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THE ANALYSIS OF NUCLEAR POWER
PLANT STRUCTURES, EQUIPMENT, AND
SYSTEMS FOR SEISMIC LOADS

An Assessment of the State of the Art

by

G. A. Young

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AGBABIAN-JACOBSEN ASSOCIATES
Los Angeles, California

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SECTION 1

INTRODUCTION

This report provides an assessment of present analysis procedures used in the design of nuclear power plants for seismic loads. The objective is to define the most reliable analysis procedures currently available, and to indicate where additional studies can develop improved procedures at a modest cost and within a reasonable period of time. It is based upon our analytical, design and research experience on problems associated with the design of nuclear power plant systems for seismic loads.

The firm of Agbabian-Jacobsen Associates (AJA) has provided consulting services to the Atomic Energy Commission (AEC) for the review of seismic design criteria for the design of nuclear power plants. Research studies have been performed under AEC sponsorship on seismic input motions and soil-structure interaction analyses, and under a joint venture with Shannon and Wilson, Inc. (SW-AJA), studies are in progress on problems associated with site investigation and evaluation. The firm has also provided consulting services to nuclear reactor manufacturers and to architect-engineer firms.

SECTION 2

STATE OF THE ART

The state of the art for the analysis of nuclear power plant systems for seismic loads, as currently applied by most engineering firms, is defined in this section by considering typical criteria for the analysis of a containment structure and the internal equipment and piping. The subjects covered are the definition of seismic input motion, analysis of structure, soil-structure interaction assumptions and equipment and piping analysis.

SEISMIC INPUT MOTION

At the present time, the seismic input motion used in the analysis is normally specified by response spectra. Although based upon careful geological and seismological investigations, the spectra are derived in an almost subjective manner. This results from the very limited quantitative data available on strong motion earthquakes. Thus, while it is generally accepted that the strength of the bedrock motion is a function of the magnitude of the earthquake and the distance from the epicenter to the proposed site, only a crude correlation between these variables has been accomplished with existing records. Records used in correlation studies have been recorded at the ground surface rather than at the bedrock level. Figure 1, for example, provides a correlation by Housner. Part of the discrepancy must result from variations in soil profiles at the different sites.

Response spectra are normally estimated by methods which were initially proposed by Housner (Reference 1), and Newmark and Hall (Reference 2). The influence of site geology and soil conditions is inferred by judgment in either method. Peak earthquake motions, such as ground acceleration, are first established and response spectra are then scaled linearly from basic spectrum relationships derived from observed earthquake records. The basic difference in

Strong Motion Records	Magnitude	Distance to Fault, Miles	Peak Acceleration, Percent, g
1940 El Centro (NS)	7.0	10-15	0.33
1949 Olympia (N10W)	7.1	25	0.18
1962 Taft (N69W)	7.7	40	0.18
1934 El Centro (EW)	6.5	35	0.16

NOTE: COMPARISON OF PEAK ACCELERATIONS OF RECORDS IN TABLE TO THOSE INDICATED BY CURVES IN FIGURE INDICATES SCATTER INVOLVED IN USING CURVES OF THIS TYPE.

FIGURE 1. RELATIONSHIP BETWEEN PEAK GROUND ACCELERATION, EARTHQUAKE MAGNITUDE AND FAULT DISTANCE BY HOUSNER

the two procedures is that the Housner procedure provides an *average* spectra while the Newmark and Hall procedure provides envelope, or *maximum*, spectra. Average velocity and acceleration spectra by Housner are shown in Figure 2. These curves represent the averages of the response spectra for two components of ground motion for each of four strong motion earthquakes experienced in California. These spectra can be scaled linearly by multiplying the ordinates of the curves by the ratio of the predicted ground acceleration at a proposed site to the acceleration on the Housner curves for a period of zero seconds (4 ft/sec^2). Actually, Housner related his spectra curves to seismic regions for which maximum ground accelerations were recommended.

Newmark and Hall suggest a procedure which is based upon a standard earthquake. The standard earthquake has a maximum ground acceleration, velocity, and displacement 50 percent greater than recorded for the El Centro 1940 earthquake. The details of this procedure are provided in Reference 2. A comparison of the spectra provided by the two procedures is given in Figure 3 for peak ground accelerations of 0.08 g and 0.15 g.

Response spectra are usually provided for two different strength levels of earthquake motion. These are designated the Design Basis Earthquake and the Operating Basis Earthquake which are defined as follows:

The predicted earthquake for a reactor site which produces the vibratory ground motion for which those features of the nuclear power plant necessary to shut down the reactor and maintain the plant in a safe condition without undue risk to the health and safety of the public are designed to remain functional is designated the Design Basis Earthquake (DBE). This is the earthquake which could cause the maximum possible vibratory motion at the nuclear power plant site.

The predicted earthquake for a reactor site which produces the vibratory ground motion for which those features of the nuclear power plant necessary for continued operation without undue risk to the health and safety of the public are designed to remain functional is designated the Operating Basis Earthquake (OBE). The peak ground acceleration of the OBE is normally assumed to be equal to at least one-half that of the DBE.

(a) AVERAGE VELOCITY SPECTRUM CURVES

(b) AVERAGE ACCELERATION SPECTRUM CURVES

FIGURE 2. AVERAGE RESPONSE SPECTRA BY HOUSNER

FIGURE 3. COMPARISON OF RESPONSE SPECTRA BY HOUSNER AND BY NEWMARK AND HALL

The response spectra for the OBE and DBE are usually plotted for different percents of critical damping to facilitate easy use in design. Figure 4 provides a typical example of the manner in which the input motion is defined. It is significant to note that at this point no attempt is usually made to indicate whether the spectra are applicable to structures founded upon soil or rock. It will be shown in the next section that, as used in the analysis, the curves normally apply to bedrock.

ANALYSIS OF STRUCTURE

Since the seismic input is in the form of response spectra, the dynamic analysis of the containment structure is normally provided by modal analysis procedures. The analysis can be considered in four steps. In the first step the structure is reduced to a mathematical model in terms of lumped masses and stiffness coefficients. The locations of the lumped masses are chosen at floor levels and points of critical interest such as equipment support points. The structure properties are reduced to uniform segments of cross-sectional area, effective shear area, and moments of inertia between mass points. A cross section through a typical containment structure showing the internal equipment and adjacent structure is shown in Figure 5. The elevation of the mass points assumed in the model formulation and the equivalent vertical cantilever mass model are shown in the figure. This particular structure was founded upon rock. A rigid connection between the structure and the foundation was assumed with no translational or rotational springs.

The second step in the analysis is to compute the undamped natural frequencies and mode shapes of the model. This is done by a computerized procedure. The interaction of the structure with the soil and the interaction effects of the internal equipment is usually not considered in the frequency analysis. Normally, at least five modes are computed.

The third step is to establish proper values of damping for the different modes. Typical values for different materials and types of structural behavior are indicated in Table 1. The basic values for damping are established after an evaluation of the response. If a specific mode shows

(a) OPERATING BASIS EARTHQUAKE

(b) DESIGN BASIS EARTHQUAKE

FIGURE 4. RESPONSE SPECTRA FOR DESIGN OF NUCLEAR POWER PLANT STRUCTURES

FIGURE 5. SECTION OF TYPICAL CONTAINMENT STRUCTURE SHOWING MASS MODEL

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TABLE 1. DAMPING VALUES
(Reference 2)

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

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

predominant rocking, then the damping for that mode utilizes primarily soils damping. If the response indicates major activity of a specific material, the damping value for that material is employed. Damping values are computed by a weighting process when the response indicates that several materials or types of behavior are involved. A damping value is assigned to each mode.

The fourth step establishes the resulting inertial forces on the containment structures, using spectra response curves such as those given in Figure 4. Acceleration values are selected from the curves for each mode, based upon the damping and frequency. Effective force, shear and moment diagrams are computed for each mode. The shears and moments of the individual modes are then combined on a root-mean-square basis.

SOIL-STRUCTURE INTERACTION

In the example above, a rigid connection was indicated between the structure and the rock, and the acceleration was applied directly to the foundation. When the structure is founded upon soil, rotational and translational springs representing the soil are incorporated into the model. The earthquake acceleration is then applied at the base of the springs, i.e., at the rock level. It is important to note that the response spectra curves, such as those in Figure 4, are therefore assumed applicable to the bedrock level and not the ground surface. This is not often considered in the initial definition of the response spectra.

It is evident from Figure 5 that a containment structure is typically backfilled with compacted soil for approximately one-third of its height. It was pointed out above that the influence of soil-structure interaction is not considered in the frequency analysis of the containment structure. However, static and "dynamic" horizontal earth pressure loads are considered in the design of the structure but in a rather arbitrary manner. The rigidity of the walls are normally assumed to be such that active static earth pressures will not develop. An at-rest condition is therefore assumed. An equivalent fluid pressure assumption is then frequently used to compute the static pressure.

A typical procedure for computing the dynamic earth pressure is to assume that the dynamic earth pressure is equal to the static pressure plus $2ag$ times the static pressure, where "a" is the ratio between acceleration produced by an earthquake shock and the acceleration of gravity, and "g" is the acceleration of gravity. Based upon this assumption, the dynamic earth pressure would be represented by an equivalent static pressure equal to 1.30 times the static pressure for a DBE having a peak ground acceleration of 0.15 g. Figure 6 indicates the static and equivalent static (dynamic) load distribution frequently assumed in the design of the containment structure. These forces are not incorporated into the dynamic response analysis as previously indicated.

EQUIPMENT AND PIPING ANALYSIS

Equipment and piping mounted inside the containment structure must also be designed for seismic loads. The criteria spectra, which have been indicated above to be applicable to the bedrock, is obviously modified by the structure. In order to obtain a modified spectra, a frequent procedure is to generate response spectra for equipment support points by subjecting the structure mass model, Figure 5, to an earthquake time history. The earthquake time history in this case is normalized to agree with the OBE and DBE peak ground accelerations. The mass points in the structure model must therefore be selected to represent equipment support points. For example, the response spectra for mass point 8 in Figure 5 would be used as input for the safety injection tank shown in the figure.

FIGURE 6. STATIC AND EQUIVALENT STATIC (DYNAMIC) EARTH PRESSURE ASSUMPTIONS

SECTION 3

ASSESSMENT

This section provides an assessment of the state of the art for seismic analysis of structures, equipment and systems of nuclear power plant structures. The objective is to indicate where improved procedures are, or soon will be, available, and to indicate the problem areas where additional analytical and experimental studies are required to improve the reliability of the design. It is based upon the assumption that the previous section, which briefly presented the criteria for the analysis of a containment structure and internal equipment is a reasonable representation of the present state of the art.

SEISMIC INPUT MOTIONS

The previous section indicates that it is conventional to specify the seismic input motions in the form of response spectra curves. These curves are then used to provide a modal analysis of the structure. But in the analysis of the equipment, a time-motion record is used as input to the structure model, and response spectra are generated for different equipment support points on the structure. It follows that if the structure model must be subjected to a time-motion record to provide design information for the equipment, the results of this analysis should also be used for the design of the structure. The modal analysis of the structure based upon an input spectra is therefore superfluous, assuming the time motion record is compatible with the input spectra which is a fundamental requirement. It is reasonable to assume that in the future, seismic input will be specified as a time-motion record. It is also evident that the input motion should be specified at the bedrock level. Additional comments on these two items follow.

There is concern whether earthquake motions can be predicted reliably for a given site at either the ground surface or at the bedrock level. It has been previously noted that prediction procedures by Housner and Newmark have been derived from relatively few strong motion records measured at the ground surface. The efforts of Housner (Reference 3) to relate peak ground acceleration to earthquake magnitude and distance from the epicenter were presented in Figure 1. A wide scatter in data was noted and it was also indicated that variations in soil profiles were an uncontrolled variable in the formulation of the Housner curves. Using the limited information available, a recent assessment has been made by Seed and Idriss (Reference 4) of the influence of overlying soils at various sites where surface records have been measured. Surface records were adjusted to provide estimated peak accelerations at bedrock level. The results are shown in Figure 7. While the procedures for making the adjustments were approximate, it was a systematic attempt to eliminate one uncontrolled variable. Thus, it cannot be concluded that the bedrock relationships in Figure 7 are any less reliable than the surface relationships postulated by Housner in Figure 1. It is reasonable to conclude that peak accelerations can be predicted as reliably at bedrock level as at the ground surface.

There is a strong need to improve the reliability of the scaling relationships. The improvement must obviously be provided for bedrock motions. Since most records are for the ground surface, records in the future should be recorded at both the ground surface and at the bedrock level. This information is needed to help assess the influence of different soil profiles and will help in converting existing surface motions to bedrock motions. Soils investigations are also required at sites where surface records have been measured. Additional bedrock and surface motion instrumentation is required to record future strong motion earthquakes, and field simulation tests may also be required. The need for these studies is indicated in a recent study by AJA (Reference 5). A feasibility study to define this program has been incorporated in the program plan of the SW-AJA Joint Venture Contract (Reference 6) with Union Carbide.

AJA2111

FIGURE 7. RELATIONSHIP BETWEEN PEAK BEDROCK ACCELERATION, EARTHQUAKE MAGNITUDE AND FAULT DISTANCE BY SEED

Additional comment is required on the form of the bedrock input. It was indicated above that the input should be specified as a time-motion record rather than as response spectra. There are two general procedures for doing this. One procedure (Reference 4) is to scale strong motion records of previous earthquakes in both amplitude and frequency to provide an equivalent bedrock motion. Unfortunately, each earthquake record is quite different and may give strong response for some frequencies and practically no response for others. A recent study by AJA (Reference 5) recommends that an ensemble of records be used to make certain that excitation is provided for all significant response frequencies. This report also recommends that band limited white noise can be used to provide an equivalent bedrock input motion which will not exhibit frequency bias. Additional study of these scaling procedures is justified and is not currently funded.

There have been several very important studies (References 4, 5, and 7) which clearly demonstrate the importance of using a mathematical model to consider the effect of overlying soils on earthquake motions. This analysis can be done with one-dimensional finite element, or discrete mass-spring-dashpot models for simple soil profile geometries and horizontal bedrock input motions. A two-dimensional finite element model is required for more complex profile geometries, and when the input has both horizontal and vertical components which should be the normal case. Figure 8 provides the results of analyses of two soil profiles (from Reference 5) of sites which have been approved for nuclear power plant construction using a discrete mass-spring-dashpot model. It is significant that the soil profile in one case amplifies the intensity of the bedrock motion, but in the other case, the soils have an attenuating effect. Mathematical analyses of these types are well within the capability of many consulting engineering firms today. Also, one- and two-dimensional models can adequately model the soil profile in nearly all cases. Therefore, it is reasonable to assume that mathematical models of these types will be used in the near future to introduce the influence of the soil profile on the bedrock motions.

FIGURE 8. PARAMETRIC STUDY OF INFLUENCE OF SOIL PROFILES ON GROUND MOTION WITH DISCRETE MASS-SPRING-DASHPOT MODEL

The overall assessment of seismic input motion is that time motion records at bedrock level should be used to specify the input motion for dynamic analysis of structures. At this time, bedrock motions can be predicted for seismic design as reliably as surface motions, but a program for improving the reliability of both predictions is needed. A program has been submitted to Union Carbide under an existing research contract. Bedrock motion can be specified as an ensemble of scaled strong motion records, or bandlimited white noise can be used to provide an equivalent bedrock motion which will not exhibit a frequency bias. Additional study of these scaling procedures is justified and is not currently funded. One- and two-dimensional models should be used to introduce the influence of the soil profile on bedrock motions. Present mathematical models are adequate for this purpose.

SOIL STRUCTURE INTERACTION

Soil-structure interaction is frequently the most misunderstood phenomenon in the overall seismic analysis. It is, therefore, the most frequently mishandled and provides a very gross area of uncertainty in the results of many analyses. Before attempting to explain the complexity of the problem, it is important to point out some of the very obvious misconceptions.

It was indicated in the state-of-the-art section that when the structure is founded upon rock, the mathematical model of the structure (Figure 5) should be assumed fixed at the bedrock level, but when founded on soil, translational and rotational springs should be introduced between the bedrock and the structure. The same earthquake input motion is then used for both structure models. Overlooking the three-dimensional aspects of soil-structure interaction, and considering this to be only a two-dimensional problem, it can be easily demonstrated that this procedure will seldom provide a reasonable model. The soil mass is a continuum which has distributed mass and distributed stiffness. The translational or rotational springs (interactors) must when used integrate the overall behavior of the soil column between the structure and the rock, and concentrate it in single interactors. This is a

very difficult modeling task. The distributed mass of the soil, of which some amount moves with the structure, is then either neglected in the analysis or added arbitrarily to the mass of the structure. This mass should vary with time, and it should vary for the different modes of response, but this is not possible with this model. The stiffness of the interactors should also vary with time and with modes of response, but this is also not possible. Obviously, this model does not adequately describe the dynamic behavior of the structure and soil mass. This is demonstrated by the following example taken from nuclear weapons effects calculations. Similar comparisons for earthquake inputs can also be made but are not as yet available.

Figures 9 through 11 provide the results of a comparison of four spring-dashpot interactor models with a finite element model of a structure embedded in soil and subjected to an impulse load. Figure 11, which provides a comparison of time-motion response records for the models, clearly demonstrates the frequency distortion and the variations that result with the interactor models. Even when soil properties are poorly defined, the need for having properly distributed mass and stiffness dictates the need for the finite element model.

The state-of-the-art section indicates that the frequencies of the structure were computed without considering the presence of soil on the base or sides of the structure. This is analogous to inferring that a bell will ring with the same frequency when struck, whether suspended in air or buried in soil, an obviously incorrect statement. It follows from the above examples that frequencies computed neglecting the presence of the soil, or using arbitrary interactor assumptions, have questionable validity. A finite element procedure is needed to introduce the effect of the soil. This procedure provides proper distribution of mass and stiffness of both structure and soil, and automatically couples translational and rotational modes. There are, however, three-dimensional aspects to the soil-structure interaction problem which should be considered. A definition of soil-structure interaction is required before this problem can be defined.

STRUCTURE ASSUMED SYMMETRICAL

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FIGURE 9. SOIL-STRUCTURE INTERACTION MODELS
SPRING DASHPOT TYPE - BARBS CASES

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(a) FREE FIELD FINITE ELEMENT ANALYSIS

(a) SOIL/STRUCTURE INTERACTION FINITE ELEMENT ANALYSIS
(WITH COMPATIBLE FREE FIELD INPUT)

AJA1523

FIGURE 10. FINITE ELEMENT SOIL/STRUCTURE INTERACTION MODEL

FIGURE 11. COMPARISON OF HORIZONTAL ACCELERATIONS (POINT 1) WITH FINITE ELEMENT MODEL WITH BARBS CASES

Soil-structure interaction is frequently defined as the modification in soil stress and soil motion caused by the presence of a structure. Modifications induced by the dynamic effect of earthquake motions result from differences in soil and structure stiffness and mass, and from structure dissymmetries. The modifications are three-dimensional, and the impact on mathematical modeling can be demonstrated by referring to Figure 12. Figure 12 provides vertical and horizontal sections of a prismatic containment structure embedded in soil. Under static loads, the pressure distribution across the base in the plane of the vertical section could be that represented by either curves (a) or (b) in Figure 12(a), depending upon the flexibility of the base of the structure, the type of soil and the magnitude of the base pressure (Reference 8). Under a rocking motion, the base pressure diagram will vary with time in a fashion that is influenced by the same variables. If a vertical section is now passed normal to the vertical section in Figure 12, at the centerline of the structure, the base pressure diagram under static loads will again be similar to curves (a) or (b). This base pressure will also be modified in some manner by the rocking motion of the structure. The static earth pressure usually assumed on the vertical face of the structure is represented by curve (c), but under a rocking motion, the maximum pressure could easily be that represented by curve (d) which is significantly different from the equivalent static load indicated in the state-of-the-art criteria. More important, this pressure diagram will vary across the vertical face in some manner such as represented in Figure 12(b). Shear stresses also vary in some undetermined manner on the base and vertical faces of the structure. Thus, the three-dimensional characteristics of soil-structure interaction are strongly evident.

Two-dimensional plane strain finite element models are frequently used to model the soil-structure interaction problem described above. In the absence of experimental test results on real or model structures, and in the absence of three-dimensional mathematical model analytical results, the reliability of two-dimensional analyses becomes suspect. This can be demonstrated

(a) VERTICAL SECTION

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(b) HORIZONTAL SECTION

FIGURE 12. SOIL PRESSURE DISTRIBUTION CURVES FOR AN EMBEDDED PRISMATIC STRUCTURE

by considering the two-dimensional finite element model in Figure 13 of the structure given in Figure 12. There are two procedures that could be followed in modeling this structure. In one, the end wall elements designated by (A), which are normal to the horizontal motion, could be given an artificial flexural stiffness that approximates the support provided by both the base and roof slabs and the side walls (B). The shear stiffness of the side walls (B) could then be added to the model as a separate element. Thus, an attempt could be made to force the model to give an average response. Exactly what an average response means relative to the real response of the wall (A), which is out of plane and is primarily in flexure, and the real response of wall (B), which is primarily acting in shear, is difficult to establish. Also, it is important to recognize that *no account would be given to variations in soil stress in the third dimension on either the base slab or the end walls (A)*. A second approach would use two, two-dimensional plane strain finite element models similar to Figure 13. In one, the end walls (A) are omitted, and in the other, the side walls (B) are omitted. One model is then representative of a section through a side wall and the other of a centerline section through Walls A. The response of these two models may then approach upper and lower bound responses of sections through the structure, *but the out-of-plane variations in soil pressure will not be modeled*, and it is difficult to integrate the results into a reliable quantitative answer.

Analytical and experimental studies are needed to improve the state of the art in soil-structure interaction studies. Three-dimensional dynamic linear elastic finite element computer analyses are currently possible and analyses should be performed on representative structures. These results should then be compared with two-dimensional dynamic linear elastic finite element analyses to establish the importance of the three-dimensional aspects of the problem. Also, since it is possible to compare the results from two-dimensional dynamic *linear elastic* finite element solutions with results from two-dimensional dynamic *nonlinear* models, it should be possible to infer the importance of nonlinear soil behavior in the third dimension. Capability with the latter two models has been demonstrated in soil-structure interaction studies (Reference 9). More reliable analysis

FIGURE 13. TWO-DIMENSIONAL FINITE ELEMENT MODEL OF AN EMBEDDED PRISMATIC STRUCTURE

procedures for design would result from this overall analytical study, Until these studies have been performed, three-dimensional analyses have the greatest reliability and a quantitative estimate of the error in two-dimensional analyses cannot be made.

The reliability of design procedures derived from the above analytical studies would be greatly enhanced if supported by experimental tests on dynamically loaded structures embedded in soils. Tests on large-scale structures require a very high energy input and extensive instrumentation. Also, it is a very expensive risk to attempt full-scale field tests without first proving the experimental and analytical techniques on smaller laboratory structures. Thus, while both laboratory and field tests may prove essential, analytical verification of laboratory experimental tests should be obtained before expensive field tests on large structures are attempted.

In summary, present analytical procedures which use spring-dashpot interactors do not adequately handle soil-structure interaction. Two-dimensional finite element procedures are a decided improvement, but the reliability of the analysis is uncertain because the three-dimensional characteristics of the problem have not been adequately considered. Two- and three-dimensional dynamic linear elastic and two-dimensional dynamic nonlinear studies are needed to provide information which can be used to improve current design procedures. Tests on experimental structures are also needed. Laboratory tests under carefully controlled conditions should be used to demonstrate that analytical and experimental verification is possible before expensive field tests are attempted.

STRUCTURE ANALYSIS

Errors in the soil-structure interaction analysis obviously carry over into the analysis of the structure and the equipment and piping. However, there are other problems associated with modeling the structure that should also be considered. Modeling the containment structure as a vertical cantilever (Figure 5), for example, is a simplification that may neglect important structural modes of response and the interaction effects of heavy equipment. Brief consideration of these two items follows.

Modeling a cylindrical containment structure as a vertical cantilever eliminates the possibility of including an ovaling mode in the horizontal plane. This could be a significant mode depending upon the geometry of the structure. For a prismatic structure such as considered in Figure 13, a similar problem results. For this configuration, the stiffness of the cantilever would be determined largely by the side walls (B) which are parallel to the motion. The end walls (A) contribute mass but little stiffness. These walls, however, experience out-of-plane flexural deflections which are comparable to the ovaling modes of the cylindrical structure. This type of behavior cannot be introduced directly into a cantilever model. Three-dimensional dynamic models which combine shell or plate elements, beam elements, and elements representing the soil continuum provide an ideal tool for modeling containment structures.

Reactors and steam generators are massive equipment items supported within containment structures. These items interact with the structure and can be considered in mathematical models in two steps, such as shown in Figure 14. In Figure 14(a) it will be noted that the equipment items are included in a model of the structure and soil. Very little detail is provided for the equipment in this model, as the objective is to compute the response of the equipment support points by a procedure which includes the interaction effect of the soil and equipment on the structure. More emphasis is placed upon the soil and structure representation in this model. The second model, Figure 14(b), which is a more detailed model of the equipment and piping, uses the response of the equipment support points from the first model as input. This will be discussed further under analysis of equipment and piping. The overall reliability of the analysis is obviously enhanced if three-dimensional models are used in both cases in Figure 14.

In summary, three-dimensional dynamic finite element model capability should be used to improve the reliability of mathematical models of containment structures. Two-dimensional models cannot effectively combine shell, plate, beam, and soil continuum elements.

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(a) SOIL-STRUCTURE FINITE ELEMENT MODEL

AJA1922

(b) MODEL OF REACTOR, GENERATOR, PUMP AND PIPING

FIGURE 14. MODELS USED FOR STRUCTURE AND EQUIPMENT AND PIPING ANALYSES

EQUIPMENT AND PIPING ANALYSIS

The seismic analysis of the equipment and piping requires a three-dimensional dynamic analysis which uses the response of equipment and pipe support points from the structure analysis as input. This was noted above relative to Figure 14. This type of analysis requires a time-motion input and cannot be adequately treated with a spectrum approach. There are available three-dimensional dynamic analysis procedures which represent the piping and equipment as structural beam elements. Several items of equipment, such as the reactor and steam generators, are shell-type structures, for which a finite element shell representation would be more realistic. This type of model representation is indicated in Figure 14(b) and has been previously noted. A dynamic three-dimensional model procedure which incorporates both shell, plate, and beam elements should become a part of the state-of-the-art procedures.

SECTION 4

RECOMMENDED ANALYSIS PROCEDURES

The previous sections indicate that present analysis procedures used in the design of nuclear power plant structures, equipment, and systems for seismic loads follow a systematic procedure, and in general utilize the best information and analytical techniques available. There is, however, a strong need for better input motion, a better understanding of soil-structure interaction, and improved analytical methods for predicting structural and equipment response. The procedures which will provide the most reliable seismic analysis at the present time are summarized below.

- a. Input motion should consist of an ensemble of earthquake records or band-limited white noise records scaled to represent bedrock motion. Both horizontal and vertical components of bedrock motion are required.
- b. The finite element three-dimensional dynamic elastic model is the best model currently available to model the overlying soil profile and nuclear power plant structures, and should be used to compute the response of the structures, and equipment and pipe support points to bedrock motion. This model is similar to that represented in Figure 14(a) and should include the mass of major equipment items. The results of this model analysis will provide information for the design of the structure, and input to the model used to analyze the response of the equipment and piping described in (c) below.
- c. A three-dimensional dynamic elastic model which incorporates beam (pipe), plate and shell elements, such as shown in Figure 14(b), should be used to determine the primary response of the equipment and piping.

- d. Secondary two-dimensional dynamic nonlinear models should be used to study nonlinear behavior indicated by the results of the primary analysis of the equipment and piping. More detailed two- and three-dimensional dynamic elastic models may also be required to provide detailed analyses of equipment supports and internal elements.

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