Course 22.314 "Structural Mechanics in Nuclear Technology"

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Main Course SMiNPT. Lecture Note No. M –32

Survey Of Problems of Seismic Analysis and Aseismic Design of Nuclear Power Plant Structures and Equipment

1. Seismic Design Basis

The possibility of earthquake damage to a nuclear power facility, sited in a seismically active area, constitutes a serious safety problem because of the consequent possibility of a release of fission products. Therefore nuclear power plants which are to be erected in seismically disturbed areas must be designed to withstand the effects of possible earthquake events.

As a design basis, two different earthquake events are being postulated. These are usually called the Operating Basis Earthquake and the Design Basis Earthquake. In simple words the Operating Basis Earthquake is defined as the maximum level of ground motion that could be expected to occur during the service life of the nuclear power station which normally is expected to last about 30 years. The Design Basis Earthquake is the maximum level of ground motion which could be envisioned to occur at the site at any time the future. The prescription of a certain level for each of these two seismic intensities is a task of engineering seismic risk analysis and of suitable design decisions which have to be weighted in balance with the radioactive hazard of the specific nuclear power plant for its environment.

Reactor plant buildings and equipment are designed to resist the effects of the Operating Basis Earthquake at normal working stress levels. For the consideration of the Design Basis Earthquake, in principle, stresses equal to the yield stress of the material could be tolerated. However, it is normal practice to limit the deformations to those associated with the proportionality limit strain of the material. In certain cases deformations may be allowed to exceed the proportionality limit strain of the material, provided that the associated ductility factors can be verified, and provided that the corresponding deflections and displacements do not impair the functioning of the safety related components.

The seismic analysis of nuclear reactor systems can be divided into several topics. These are:

(1) Seismic site evaluation and generation of seismic input data based on engineering seismic risk analysis.
(2) Classification of the parts of the reactor system with respect to their relative importance with regard to reactor operation and reactor safety.
(3) Specification of seismic design criteria from the stress and strength point of view as

well as from the point of view of functional requirements

(4) Analysis of the dynamic interaction effects between the underlying soil and the heavy reactor plant structures.
(5) Dynamic analysis of the reactor building structures which include the equipment support structures as well as the containment building structures.
(6) Dynamic analysis of the equipment and piping systems. The main parts of the equipment are the reactor vessels with the internal core structures and its supports as well as the control system, and the heat exchangers.
(7) Design considerations to bring about optimum aseismic features of the plant with minimum investment.

This lecture throws a few spotlights on some of these points. Figure 1 illustrates the interrelationships between the various design tasks.

2. Seismic Input

Earthquake ground motions can either be prescribed for a site by suitable adaption of strong motion records (Figure 2) or by the generation of simulated earthquakes on a stochastic basis (see Lecture Note M-32A).

The simulation process can be summarized as follows: A stationary random process --for example a Gaussian process -- is generated on a digital computer. The white noise has to fulfil a prescribed power spectral density. This white noise is then passed through a filter whose properties are chosen to yield the desired frequency content. Finally, by multiplying the resulting sections of the filtered stationary Gaussian process by a suitably chosen envelope, the desired nonstationary properties are given to the record (Figure 3, Figure 4).

3. Classification of the Parts of the Reactor System

The method of structural analysis to be utilized to determine the seismic loadings on reactor structures and equipment depends on the importance or the particular reactor structures and equipment with regard to reactor operation and reactor safety. For this reason it is customary in the design of nuclear reactor facilities to classify the parts of the structures and equipment into three categories. These three categories can be defined as follows:

Class I: Those structures, equipment systems and components whose failure clearly might cause a reactor accident or contribute to it. Further, those structures, equipment systems and components that are required to maintain the reactor facility in a safe shutdown condition, and finally those which have to prevent the release of radioactivity to the environment.

The Class I parts thus include the primary reactor coolant pressure boundary, reactor controls and control rod drive, the emergency core coolant system, heat removal systems for spent fuel storage, as well as the containment building.

Class II: Those structures, equipment systems and components which are essential to permit power plant operation but whose individual failure would not cause or contribute to a clear reactor incident and also would not impair the capability for safe shutdown and for containment.

Class III: Those structures, equipment systems and components that are not directly required for the operation of the nuclear reactor system but are essential or convenient for maintaining support for normal plant operations.

This classification is customary in the United States. Slightly different classifications are in use in other countries, but generally the same philosophy is being followed. In Japan, for example, containment structures are categorized in a special upgraded class for which the most stringent requirements apply. In Germany the Classes II and III are lumped together.

It should be pointed out that a nuclear power facility constitutes a tremendously complicated system and that for this reason it is impossible to clearly separate all of the structures and equipment into each of the various categories. In cases when a clear categorization is not possible, it is, of course, necessary to upgrade certain elements of the facility to ensure that a conservative and safe design will result.

As far as the methods of structural analysis to be utilized are concerned, it is required that the seismic response of Class I structures and equipment be determined by a suitable truly dynamic analysis. A suitable dynamic analysis can be defined generally as one which adequately accounts for the postulated seismic ground motions, for the dynamic response of the structures and equipment to these ground motions, taking into consideration the dynamic interaction effects between underlying soil and structure as well as between structure and mounted equipments, and for the appropriate degree of energy dissipation in structures and equipment.

Structures which are categorized in Classes II and III may be analyzed using equivalent static techniques as, for example, they are presented in the United States in the Uniform Building Code.

For practical reasons it is convenient to classify the parts of a reactor system also according to design groups, which include: Reactor internal structures, pressure vessels, piping, heat exchangers and other coolant system components, and building structures.

Such a type of classification according to design groups is usually desirable because the design work for each class is generally done by different design groups or even by different companies. A further reason is that codes and standards are established for similar classifications. Each of these classifications may also have specific seismic design requirements.

4. The Equations of motion and Solution procedures

4.1 The Equations of Motion

The equations of motion of a multi-degree of freedom damped system, subjected to an arbitrary ground motion may be written as (see Lecture Note M-31):

$$[M]\{\ddot{u}(t)\}+[C]\{\dot{u}(t)\}+[K]\{u(t)\}=\{F(t)\} ,$$

where

[M], [C], [K] are the mass, damping, and stiffness matrices, respectively;

$\{u(t)\}, \{\dot{u}(t)\}, \{\ddot{u}(t)\mathring{a}\}$ are the displacement, velocity, and acceleration vectors respectively; and

$\{F(t)\} = \square[M]\bar{P}\ddot{u}_g(t)$ is the forcing function vector, where $\ddot{u}_g(t)$ is the time-dependent prescribed support acceleration, $\bar{P}$ is the vector with components of unity in directions parallel to support movement and zero otherwise.

Using the orthogonality relations and expressing the displacements, velocities, and accelerations into normal coordinates, the system of coupled equations of motion may be transformed to a set of uncoupled normal equations of motion.

There are three methods of dynamic analysis that can be used to solve multi-degree-of-freedom structural problems for specified input acceleration time histories:

- the direct integration time history method,
- the time history modal analysis method,
- the so-called response-spectrum modal analysis method.

4.2 The Time History Methods

The direct integration time history method is a numerical step-by-step integration method which is applied to the system of coupled equations of motion.

The time-history modal analysis procedure is based on the uncoupled form of the equations of motion. In this case the set of uncoupled equations is numerically integrated and the results are thereafter combined.

Both time and history methods lead to the time-histories of the displacements and effective forces in the structural system. From these data the time-histories of shears and moments can be determined by conventional structural analysis procedures. The maximum values of the shears and moments are determined and then used for design. Figure 5 shows a simplified block diagram of the computation process.

It is apparent that if the time history procedure is adopted, the response of any structural system must be determined for several earthquake input records and the maximum response parameters for all those records have to be summarized. This is essentially a computer operation; the number of seismic input records will increase the computer time required. In order to minimize this, the least number of time-histories, consistent with the required accuracy should be used. Parametric studies seem to indicate that satisfactory results may be obtained with as few as 8 to 15 artificial earthquakes.

In contrast to this computational effort, the great computational advantage of the so-called response spectrum analysis method is that all the seismic input is represented by a single function, so-called smoothed design response spectra. Smoothed response spectra represent an envelope to all conceivable earthquakes with different time-histories which might occur at a given site.

4.3. Response Spectrum Modal Analysis Method

The decoupled system of differential equations of motion which constitutes the basis for the time history modal analysis technique, also forms the basis of the response spectrum modal analysis method.

The response spectrum modal analysis method is very popular among structural engineers and mechanical engineers engaged in seismic analysis and aseismic design of nuclear power plant facilities.

In the following, the concept of response spectra is briefly defined by us of an illustrative sketch given by R.V. Whitman (Figure 6) in his introductory lecture to the MIT Seminar on Seismic Design of Nuclear Power Plants, 1969.

The basic information to be processed is a specific accelerogram of seismic induced ground motion. A single-degree-of-freedom linear system with a specified value of damping is taken and the vibrational response of this simple oscillator to the specified group motion is calculated.

In the illustration, for example, the relative displacement between the ground and the mass is plotted as a function of time. From this diagram we take the maximum value of the relative displacement and plot it on a graph versus the fundamental natural period of vibration of the one-degree-of-freedom spring-mass system. One such mathematical operation gives one point on the diagram.

If we hold the damping of our simple linear-elastic oscillator constant but vary its natural period and carry out the calculation for many systems with different eigen periods, we obtain an assembly of such points in the above diagram. These points define a curve which is called the elastic response spectrum for the given seismic input motion and for the specific damping coefficient. The repetition of the whole process for another specified damping ratio results in another response spectrum curve.

The peaks in a response spectrum occur when a natural frequency of the structure meets with a corresponding frequency content of the seismically induced ground motion. The response spectra tend to become smoother with increased damping.

By using a tripartite logarithmic plot it is possible to represent three response quantities of interest in a single diagram. These quantities are:

D = maximum relative displacement between the mass of the oscillator and its base,
$V = \omega D$ = maximum pseudo relative velocity,
$A = \omega^2 D$ = maximum pseudo acceleration of the mass of the oscillator.

In these relations, ω is the circular natural frequency of the oscillator. The maximum pseudo relative velocity is a measure for the energy absorbed in the spring and the maximum pseudo acceleration is proportional to the maximum force in the spring.

As an example for such a tripartite plot of response spectra, Figure 7 shows response spectra computed for the strong-motion earthquake which occurred at El Centro, California in 1940. Similar diagrams can be obtained for single-degree-of-freedom structures in which energy is absorbed inelastically in the spring.

As a basis for design purposes, so-called smoothed response spectra are used which can be considered as envelope curves to the response spectra due to all possible seismic ground motions due to earthquakes of a prescribed intensity level at a given site.

Figure 8, taken from a paper by Newmark and W. J. Hall, shows such a smoothed response spectrum for earthquakes having a maximum ground acceleration of 0.33 g. The maximum values of ground acceleration, ground velocity, ground displacement are represented by the lower polygonal curve. For this earthquake-set a smoothed response spectrum curve has been developed for the case of 2 per cent of critical damping. In the table the authors suggest amplification factors for various degrees of damping from which a set of smoothed response spectra for elastic responses can be constructed.

The response spectrum modal analysis, however, yields only the maximum response values of a seismically loaded structure. The method does not give information at what times the maximum values of the various vibration modes of a particular component occur; the phase relationships between the individual modal maxima remain unknown. Various rules have been proposed which suggest how we might combine the individual maxima to obtain approximate but conservative results, however, there is inherently some uncertainty involved.

The most commonly suggested method of combination considers it reasonable to combine the effects of the several components of seismically induced motion on a probabilistic basis. In this approximation, the probable total response -- stress, deflection, or some other specified quality -- is obtained by computing the square root of the sum of the squares the modal maxima.

There are situations when the utilization of the response spectrum modal analysis yields highly uncertain results. For example this is the case with complex structures which may have several significant contributing modal responses with nearly equal maximum values which may occur at different times and with opposite signs. One could, of course, remain on the conservative side for all conceivable cases by using an upper bound approach to modal response combination, however, this may result in a highly uneconomic design.

Another troublesome case for the response spectrum modal analysis is when equipment systems are mounted at several locations and elevations in one supporting structure or even interconnected between two or more separate supporting structures.

5. Procedure of Seismic Analysis

After this short look at the equations of motion and the solution procedures, several practical problems in the seismic analysis of nuclear power plant structures and equipment are to be discussed. Figure 9 gives a summarizing view of the different parts of the task and the general procedure.

6. Soil-Structure Interaction Effects

6.1 General

If a nuclear power plant is not founded on firm rock but on soil, the seismic input to the base of nuclear power plant structure will be considerably influenced by soil-structure interaction effects. Seismic motions applied to the base of a massive structure founded on soil may be severely modified by the induced motion of the structure itself. The feedback of structural oscillation to the underlying soil may significantly affect the motion at the soil-structure interface which in turn may result either in amplification or in reduction of the response of the structure.

Figure 10 schematically shows the deformation of the soil-foundation interface caused by feedback from the horizontal, vertical, and rotational oscillatory motion of a symmetrical structure. A structure which is not symmetrical about the axis normal to the earthquake excitation will also experience torsional motion.

To account for the soil-structure interaction effect, equivalent foundation springs are usually introduced at the soil-structure interface. For a homogeneous soil body, determination of the characteristics of these soil springs can be done by analytical methods; in case of a heterogeneous soil continuum, the finite element method is in place (Figure 11).

For determination of the interaction effects, and thus the lumped characteristics of the soil springs, a rather simple model of the structure is used in combination with the detailed finite element representation of the soil. The application of the finite element approach to analysis of the dynamic behavior of soil bodies allows a general flexibility in treating problems of rather complex geometry, namely non-homogeneous soil conditions, complex boundaries, structural inclusions, etc. Consideration of non-linear soil properties requires the application of the direct integration technique.

Figure 12, for comparison, shows the horizontal responses of a typical pressurized water reactor building and of a typical boiling water reactor building for the cases of foundation on rock and foundation on a gravel layer. The general results in this particular study are that the layer of gravel tends to amplify horizontal motions below approximately 8 Hz and to attenuate motions above 8 Hz.

6,2 Through-Soil Coupling Effect of Adjacent Structures

In the past, theoretical investigations of soil-structure interaction during seismic disturbances had been confined to single-structure configurations. The coupling through the soil with other massive structures had not been considered. Thus, adjacent structures had been treated as separate systems which do not influence one another, whereas in reality they are systems coupled through the underlying soil.

In recent time there is an increasing popularity of twin-plant nuclear power installations, involving two or more independent large and very heavy building structures, which are built on separate base slabs. This made it necessary for engineers to assess the effects of coupling between adjacent structures on the dynamic response of nuclear reactor systems during seismic loading.

Interesting parametric studies have been made recently for a seismic model which consists of two identical structures simulating the twin reactor buildings and a third structure representing either the turbine building or the reactor service building (Figure 13).

The theoretical formulation is based on a dynamic model in which these structures are bonded in close proximity to an elastic half-space and are simultaneously excited by the surface motion of this half-space. In the parametric study the separation distances between the structures were varied. The numerical computations also covered different foundation media and different distribution of superstructure natural frequencies.

The solution scheme essentially involves the combination of the equations of motion of linear three-dimensional elastic multi-mode structure with the dynamic response solution of an elastic half-space.

The important findings of those parametric studies are that, indeed, the seismic loads on reactor structures can be significantly altered by the coupling through-soil of adjacent structures when the plants are built on soil media. The magnitude of the coupling is increased with decreasing soil stiffness. The seismic response as modified by the coupling effects may be influenced advantageously by proper design and layout of the twin nuclear plant complex. Reductions of dynamic loads achievable by proper plant layout may be quite considerable.

Conversely, improper plant layout can result in dynamic response in excess of those predicted by considering the individual structures of independent systems

7. Mathematical Modeling of Structures

7.1 General Principles

An essential feature of the dynamic analysis of structures is the conception of simplified mathematical-mechanical representations of complex systems. The mathematical modeling has to fulfil the requirement that the analysis retains all-important features of the dynamic behavior of the actual structures.

A commonly used mathematical mechanical-model is the so-called Discrete Lumped Mass System. The idealization accomplished by this model consists of representing the structures, vessels, piping and ether components by a network composed of lumped masses at a finite number of characteristic points, so-called nodes, which are connected by elastic springs representing the system stiffness, and by viscous dashpots, representing the damping properties of the structure. (The connecting elements are considered as weightless.)

Some examples for lumped mass idealization for various types of structures are shown in Figure 14. For pin jointed trusses it is evident that the masses should be assumed to be concentrated at the physical joints of the structure. For other structural systems such as frames, plates, and shells, the suitable choice of mass points is not as simple.

Each mass, concentrated at a node, can have six degrees of freedom; namely, three translations in the three principal orthogonal directions and three rotations about the three principal orthogonal axes. There are then as many modes of vibration of the model as there are independent degrees of freedom. Some of the degrees of freedom may be eliminated by kinematic considerations or may be ignored because they are relatively unimportant.

The degree of complexity of such a mathematical-mechanical model will depend on the extent of information to be obtained from the analysis. In any case, there are two conflicting requirements. On the one hand the model should be able to closely simulate the expected dynamic behavior of the real structure; on the other hand, the model should be simple enough to allow economic computation and easy interpretation of the results.

With the complex structural system of nuclear power plants a suitable selection of the nodes and of the significant degrees of freedom for these nodes is a task which requires considerable engineering judgment. If sufficient experience is lacking, it is advisable to vary the mathematical model in order to minimize the possibility of omitting significant effects

Normally, characteristic points or nodes are selected so that they coincide with concentrations of mass; this means, at floors or at other locations important for stiffness. The characteristic points for lumping of the masses of a vertical axisymmetric structure are best selected at the centroids of horizontal cross-sections through individual parts of the structure; in such a case the centroids lie on the vertical center line. Figure 15 shows an example of such simple modeling of a nuclear power plant structure.

The stiffness coefficients for framed structures and pipework systems can be obtained by conventional frame analysis. Plate and shell structures can be treated by equivalent frames, but they are better modeled by use of the finite element method. Figure 16 shows the structural system of a BWR with pressure suppression containment, and Figure 17 shows the kind of modeling used for the seismic analysis of this structure.

If the mass of the physical structure is assumed to be concentrated at a finite number of locations on the structure, this results in a diagonal mass matrix with an effective mass for each degree of freedom. Finite element discretization leads to much more populated matrices. This, of course, increases the required computation considerably, but it gives a more realistic picture of the behavior of the system.

One way of reducing the computation efforts is keeping the number of carefully chosen mass points as low as possible. Another way of reducing the computation costs lies in the already mentioned elimination of degrees of freedom which appear to be relatively unimportant at specific nodes. The computational and interpretative effort can be further reduced by treating some degrees of freedom that are uncoupled from the others in an independent analysis.

The translational motions of symmetric structures, such as axisymmetrical shells in the vertical directions, for instance, can be uncoupled from lateral motion. Inertia forces due to vertical motion can, in this case, be neglected in computing lateral motion, and vice versa. If structures and substructures are symmetric with resect to two orthogonal horizontal axes, lateral motion in the direction of the two axes can also be uncoupled, since in such a case it is not accompanied by rotation about the vertical axis.

In real nuclear power plant structures however, the center of gravity and the center of shear lie a distance apart. In these cases, torsional modes can be excited and the lateral motions are coupled through the torsional stiffness of the structure (Figure l8).

7.2 Substructuring

Idealization of a nuclear power plant as a single mathematical model -- to be analyzed in one run -- may be prohibitive with regard to calculational efforts. The complexity of nuclear plants requires that, for analysis purposes, the system be subdivided into two or more interlinked mathematical models.

There will be at least one primary supporting structure model, and there may be one or more major supported secondary models. The support motions of the secondary models are being furnished by the motions of the primary model. However, unless the mass of the secondary system is very small in comparison with the mass of the primary system, there will be interactions at the interfaces of the mathematical models which must be included in the analysis. However, the interaction effects at the interfaces can be predetermined by a coarse-structured analysis done separately for each of the partial systems. Reactor pressure vessels and heat exchanger vessels will normally have considerable dynamic interaction with their supporting structures. Relative light piping systems will normally influence supporting concrete structures in a negligible way. The pre-estimated magnitude of the dynamic interactions at the interfaces may determine how the facility may be subdivided into separate mathematical models.

It should be noted that there are situations when light secondary systems may have disproportionate dynamic back-coupling influence on the supporting primary system. Such a case, for example, can arise with a relatively complex primary system, where a single spring and mass is connected to one of the masses in the primary system (Figure 19); (the case has been discussed by N.M. Newmark).

In the following discussion we will neglect the consideration of interaction effects. In the seismic analysis of a nuclear power plant, the analysis of the primary supporting structure must furnish input information for the supported equipment, namely the piping systems, vessels and substructures. In order to obtain this information, a dynamic analysis with a time-history input function will be required for the primary supporting structure. The analysis can either be performed by direct numerical integration of the couple system of equations of motion or it can be done by the time history modal analysis method which is based on the uncoupled set of equations of motion. The calculation renders the time histories of the points attachment of the supported subsystem.

Once the time-histories of the instructure support motions are obtained, the next step may either be to perform a time history analysis for the subsystem or to analyze the secondary system by use of the response spectrum modal analysis technique.

In the latter case, single-degree-of-freedom systems with the natural frequency range of interest and various damping ratios are subjected to the time history of the structure support motion. According to the earlier explanation, the natural periods of the single oscillators are plotted on the abscissa and the corresponding maximum acceleration responses obtained are plotted as ordinates. Then, an upper bound envelope of the plots is drawn. This then gives the smoothed structure response spectra for the analysis of the secondary system.

It should be noted that there are also attempts to develop reliable shortcut methods for developing floor response spectra for equipment design in nuclear power plants based on the ground response spectrum and the results of a response spectrum analysis of the supporting structure. But whatever the official opinion of the licensing authorities is, simplified analysis procedures for seismic analysis have great value to the design engineer for design optimization purposes, even if a more elaborate analysis is required for the final design.

In general, the following can be said about structures and substructures. As the effects of vibratory motion are transmitted through a structural system, each part acts as a filter and predominantly transmits selected frequencies to subsequent parts of the system. Thus, the seismic input to a supported subsystem may be considerably different from the input to the supporting system.

A problem which deserves special care is the matter of subsystems having supports on different supporting structures. There may be transient relative motions which produce strain in the secondary structure, in addition to the strains produced by the dynamic effects of the overall motion (Figure 20). This is particularly important for piping systems.

8. Reactor Piping Systems

As far as reactor piping systems (Figure 21) are concerned their masses are relatively small so that the dynamic feedback effect to the supporting building structure is not a significant effect. Nevertheless the dynamic analysis of complex three-dimensional piping systems with connected equipments involves considerable problems.

Figure 22 demonstrates the complexity of a piping configuration. Such piping systems are usually idealized as discrete mass systems with three translational degrees of freedom at each mass point. The establishment of the three-dimensional stiffness matrix must take into account the effects of flexural, torsional, shear, and axial deformation, as well at the flexibility effects of curved elbows.

A primary piping systems is at one end anchored to the reactor pressure vessel and the other end it is anchored to the steam generators. In between, such a piping system is restrained by numerous anchors, guides and so-called snubbers.

A piping system has to be designed to provide sufficient flexibility to keep the stresses caused by thermal expansion at low values. Therefore it is necessary to place hydraulic snubbers, shock absorbers, or limit stops at various locations on the piping system to reduce excess motions and stresses caused by earthquake excitation. The hydraulic snubber is a device which permits free, slow thermal movements but acts as a rigid support during earthquake excitation. The shock absorber is used to absorb part of the induced energy, consequently it reduces the pipe motion.

The determination of the dynamic behavior of reactor piping with all its supports, supporting springs, hinges and branches and with its appended masses is difficult for the following reasons:

- The various parts of a nuclear power plant which support a piping system, or are otherwise connected to it, oscillate with different amplitudes and directions. Therefore, a piping system is essentially subjected to different excitations at each anchor and restraint location.

- The flexibility of the pipe connections to the equipment is not easy to assess.

- Thermal conditions can affect material properties, restraint locator and forces as well as clearances. Since these conditions influence the system response, both the cold and the hot conditions should be considered in the seismic analysis.

- A problem for the design of piping systems which operate at elevated temperatures is that the flexibility requirements to accommodate the malexpansion may not be compatible with the restraints required to accommodate seismic effects.

9. Damping

Determination of the damping coefficients to be used in the dynamic analysis of the various structures and equipment components of nuclear reactor facilities is one of the most important steps in the seismic analysis. The actual damping in the system is a complex phenomenon that includes inelastic action, hysteresis damping, viscous damping and frictional forces. The magnitude of the damping in components is not only a function of the stress- and temperature-dependent materials properties, but it also depends on the amplitude of vibration, on displacement velocities and on the vibration frequency.

Presently the choice of appropriate damping coefficients is still more of an art than a science. There is relatively little test data available to support an estimate of true damping characteristics of the components of a nuclear reactor plant under strong motion earthquake effects.

Most of the available damping data have been obtained by dynamic tests of structures and components under conditions of small amplitude distortion. However, there are indications that damping in complex structures may be strongly displacement dependent. This casts doubt on the utilization of results of the low amplitude vibration tests as far as damping is concerned. The results probably do not accurately reflect the damping that might be expected for the large amplitude motions associated with severe strong-motion earthquakes.

Parametric studies, however, have shown that very small changes in the assumed damping may significantly change the response of a structural system. Therefore, there is a considerable need for experimental research that will reduce the large uncertainties in the currently used damping values.

10. Reactor Internals

A particular problem in the dynamic analysis of reactor core structures is the problem of impact forces that arise from the vibratory motion when the unrestricted relative displacement between two parts of the reactor internal structure exceeds the clearance between them. This problem of narrow gaps may exist between the pressure vessel and the core barrel of a PWR, among fuel assemblies, between fuel assemblies and baffle plates, and between control rods and their guide paths (Figure 23).

The only technique available for analyzing systems in which impacts occur is direct time integration of the equations of motion. Nonlinear analysis by the direct time history method allows determination of the magnitude of the impact forces when banging occurs. It should include not only structural damping due to the components' vibration, but also the energy losses during the impact.

It is clear, that even more than the case of linear analysis, this method presents problems of numerical stability and of convergence which are dependent on the stiffness and mass of the components and the damping as well as on the particular method of integration employed.

A particularly important problem in the seismic analysis of reactor internals is to assure that the control rods can be inserted into the reactor core both during and after an earthquake.

11. Design Considerations

In striving to achieve an aseismic design of a structure, the designer can choose to resist the seismic motion in various ways. He may choose to use a relatively flexible, energy-absorbing structure which can comply with the base motions readily; or he may choose a relatively rigid structure to limit the relative deformation within the structure itself.

As N.M. Newmark has pointed out in his various guides to aseismic design, the strains in a relatively flexible structure are determined primarily by the maximum transient base displacement, while the strains in a relatively stiff structure are determined primarily by the maximum transient base acceleration. In the intermediate range of stiffness, the energy absorbing capacity within the structure is of greatest importance, it involves both the strength and ductility. The trade-off between strength and ductility should be made in a balanced fashion.

It may be of interest to give an impression of the size of the task of a complete seismic analysis of a large-capacity power reactor plant. At the MIT seminar on Seismic Design of Nuclear Power Plants in 1969, R.V. Whitman mentioned in his introductory survey that the cost of the additional engineering services required for ascertaining an aseismic design of a nuclear power plant amounted to perhaps about $250,000.

In 1972, the President of a specialized consultant firm told me that costs for conducting a complete seismic analysis of a nuclear power plant according to the requirements of the USAEC Division of reactor licensing amounted to between 0.8 and 1.2 million dollars. He added that these numbers do only apply under the provision that an experienced staff runs the analysis with available sophisticated computer programs. No computer program development costs were included in these figures.

The utilization of less sophisticated and less expensive methods of dynamic analysis, which then must be proven to be conservative in every respect might lead to much higher capital costs for making the reactor plant system earthquake resistant. The problem is a problem of knowing how and where.

12. Hydrodynamic Effects

A special problem in the seismic analysis and aseismic design of nuclear power plants, namely the problem of hydrodynamic effects should be mentioned here.

The problem of hydrodynamics in the context of seismic analysis of nuclear power plants can be divided into two parts. The first is the effect of confined fluid on the vibration of structural systems immersed in that fluid. The second part of the problem is the effect of sloshing fluid on the vessel which contains the fluid.

13. Vibration Testing of full-scale reactor Facilities

13.1 Test Methods

During the past decade there has emerged a trend to perform dynamic testing of full-scale reactor plant structures, in order to check the "algorithm makers", or even more in order to check the "input makers" and the designers of mathematical models.

The following energy inputs are being utilized:

- ambient vibrations,
- structural vibrators,
- snapback,
- explosive blast.

Ambient vibrations are caused by low-level influences. These are natural ground vibrations or vibrations induced by heavy traffic, or vibrations induced by wind. This method suffers from the disadvantage of extremely low amplitudes and that the experimenter has no control over the input force frequency content and amplitude. Nevertheless the method is useful in some cases as a means for preliminary assessments.

Figure 24 shows the instrumentation required for a forced vibration test. In this case the excitation is provided by an eccentric mass structural vibration. The applied force can be varied by adjusting the eccentricity.

The frequency is varied by a manually adjusted feedback control system. The direction of the applied force can be varied by moving the vibrator or by adjusting the direction of the force vector.

Data acquisition is by means of accelerometers or other vibration transducers. The analog data are recorded on a strip chart recorder. The record also shows the position of the force vector as a function of time, so that the phase angle between the signal and the applied force can be obtained.

The analog data are scaled and compiled from the charts, and punched on cards for digital computer processing. This is a tedious and expensive step in this method. The computer converts the raw data to values of acceleration and displacement as a function of frequency, and then plots response curves from which damping values and mode shapes can be obtained.

The principal advantages of the forced vibration technique are the capability to excite specific frequencies so that individual modes can be emphasized, and the fact that the applied force is a known quantity. Also, the forced vibration technique permits levels of response which are 10^2 or 10^4 times greater than ambient vibrations, although generally still 10^1 or 10^2 times less than strong motion earthquakes.

In snap-back tests, an initial displacement or the structure must be provided. Cables with tensioning devices or a hydraulic ram are used to produce large static displacement of equipment such as a steam generator or primary coolant pipe. When the displacement forces is suddenly removed the equipment undergoes relatively large amplitude free vibrations.

The explosive blast technique makes use of explosive charges placed in the soil adjacent to the structure to be tested. Vibrations of rather amplitude can be induced in large structural complexes.

13.2 Parameter Identification

When experimental data on the dynamic behavior of a structure have been acquired, the next task is to identify the structural parameters from experimental data. The parameter identification approach may be schematized as shown in Figure 25. In addition to the experimentally studied system, there are three basic units: the model, the criterion function, and the parameter adjusted algorithm.

The process of parameter identification can be divided into three

1) The first part consists in the determination of the mathematical-mechanical model, this means in the appropriate determination of the governing differential equations, and further in the isolation of the unknown parameters.

2) The second part of the parameter identification process is the selection of a criterion function by means of which the suitability of the fit of the response of the theoretical model to that of the actual system can be evaluated, when both the theoretical model and actual system are subjected to comparable inputs. The criterion function is constructed in such a way that a good fit is obtained when the criterion function is minimized as a function of the unknown parameters of the theoretical model. For most applications the criterion function is too complex to minimize analytically.

3) The third part of the parameter identification process is the selection of an algorithm or a strategy for adjustment of parameters in such a way that the difference between the model and system responses as measured by the criterion function, is step-wise minimized.

14. Bibliography

see e.g.:
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T.W. Pickel, Jr.: Evaluation of Nuclear System Requirements for Accomodating Seismic Effects. Nuclear Engineering and Design 20 (1972) p. 323-337

Proceedings of the 1st SMIRT Conference (1971), Vol. 5, Part K.

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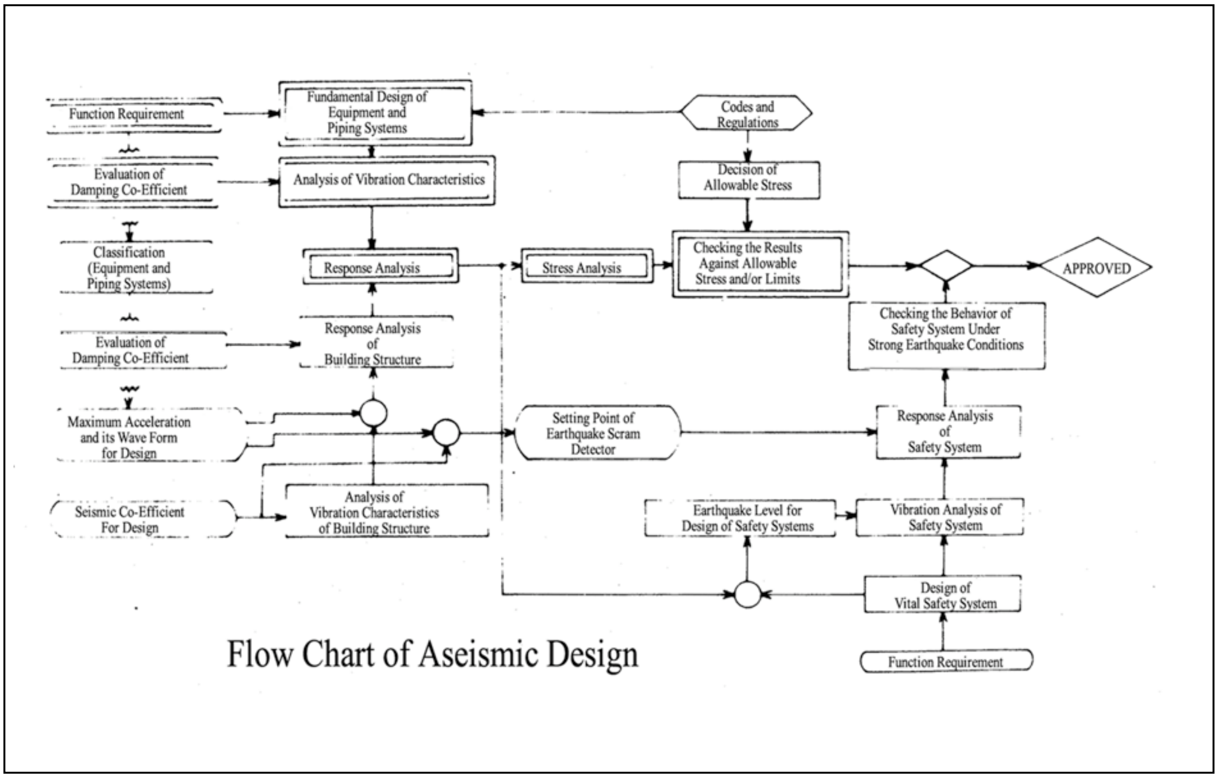
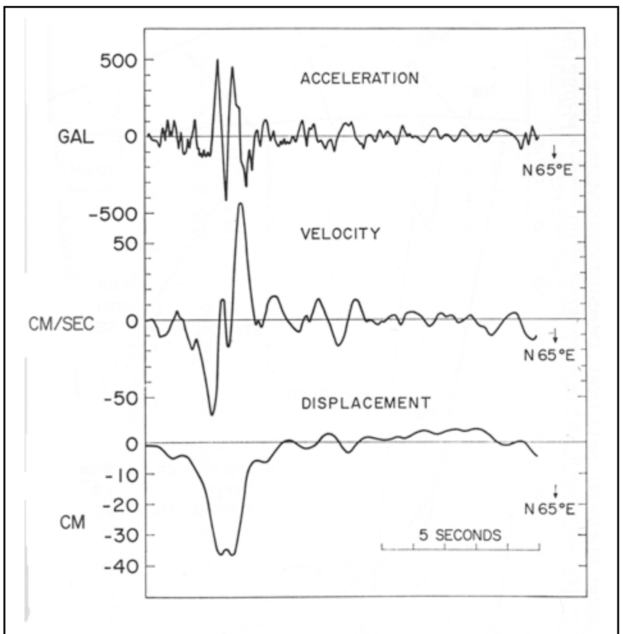


Fig. M-32.1



Ground velocity and displacement obtained by integrating the perpendicular component accelerogram (reproduced from Aki [1968]).

Fig. M-32.2

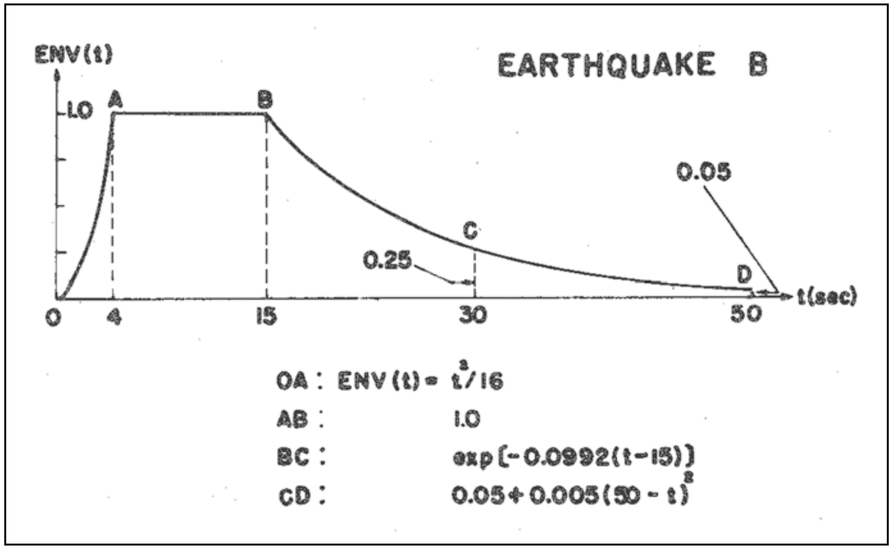


Fig. M-32.3

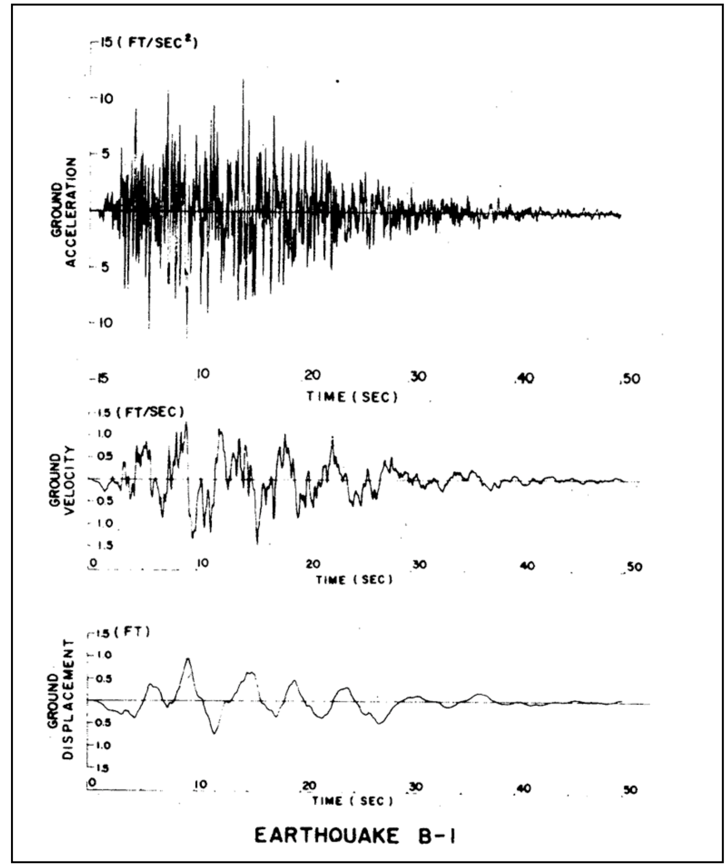


Fig. M-32.4

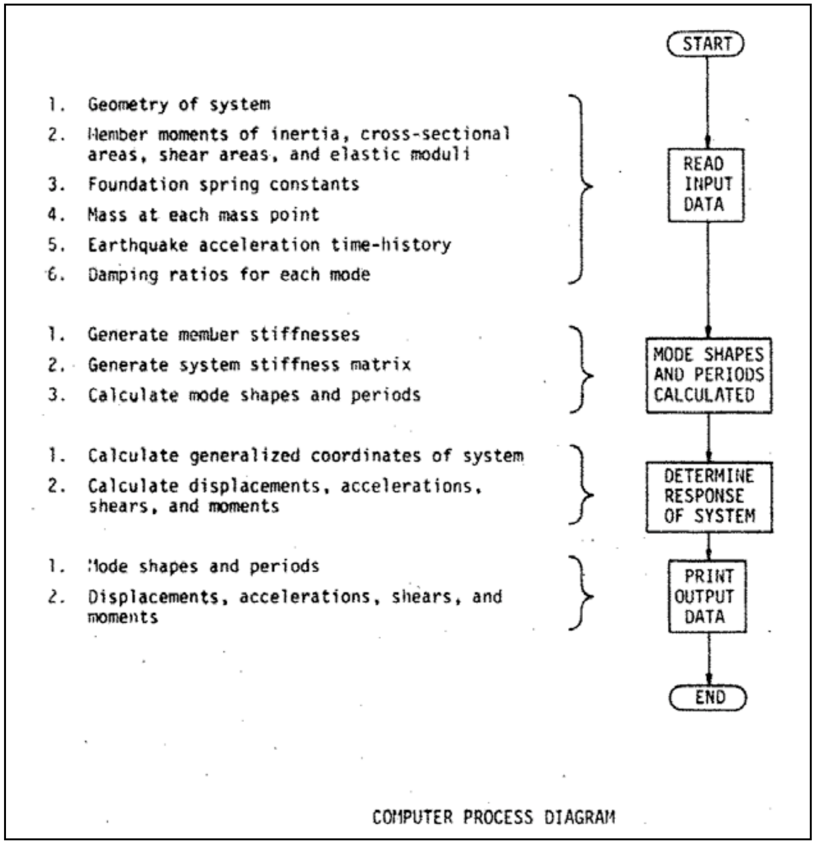


Fig. M-32.5

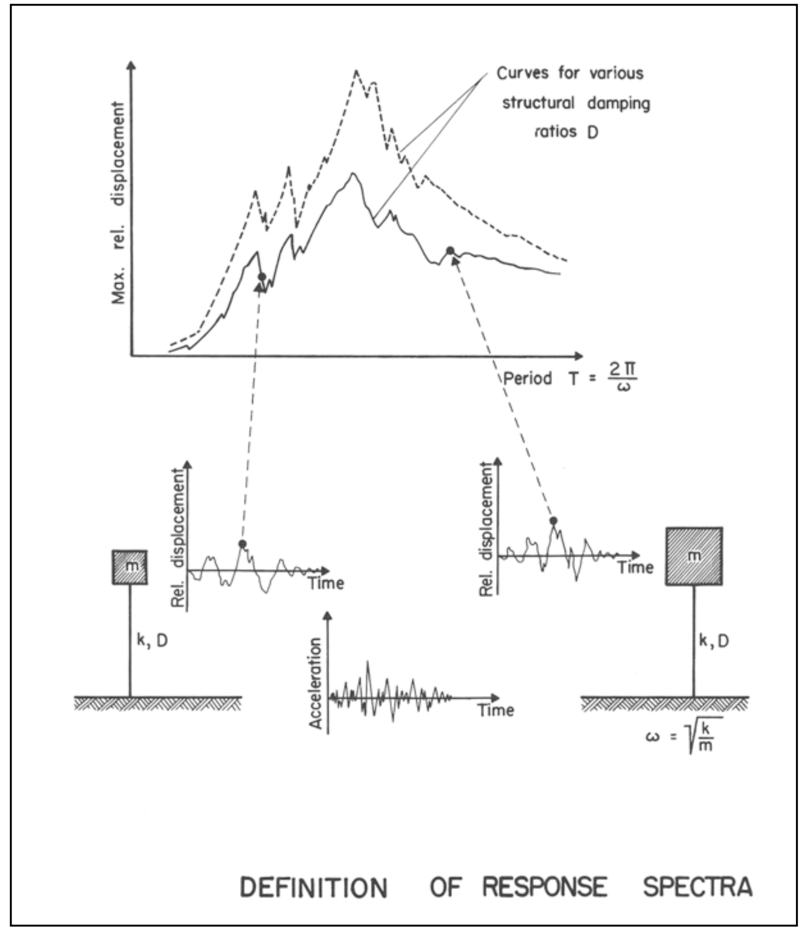
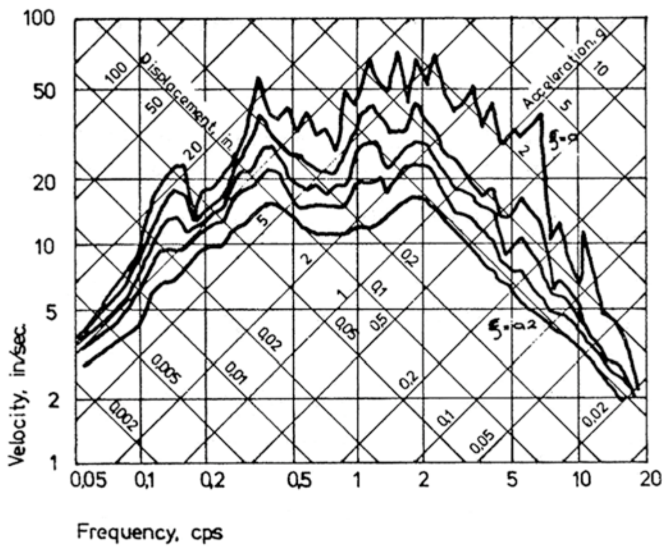


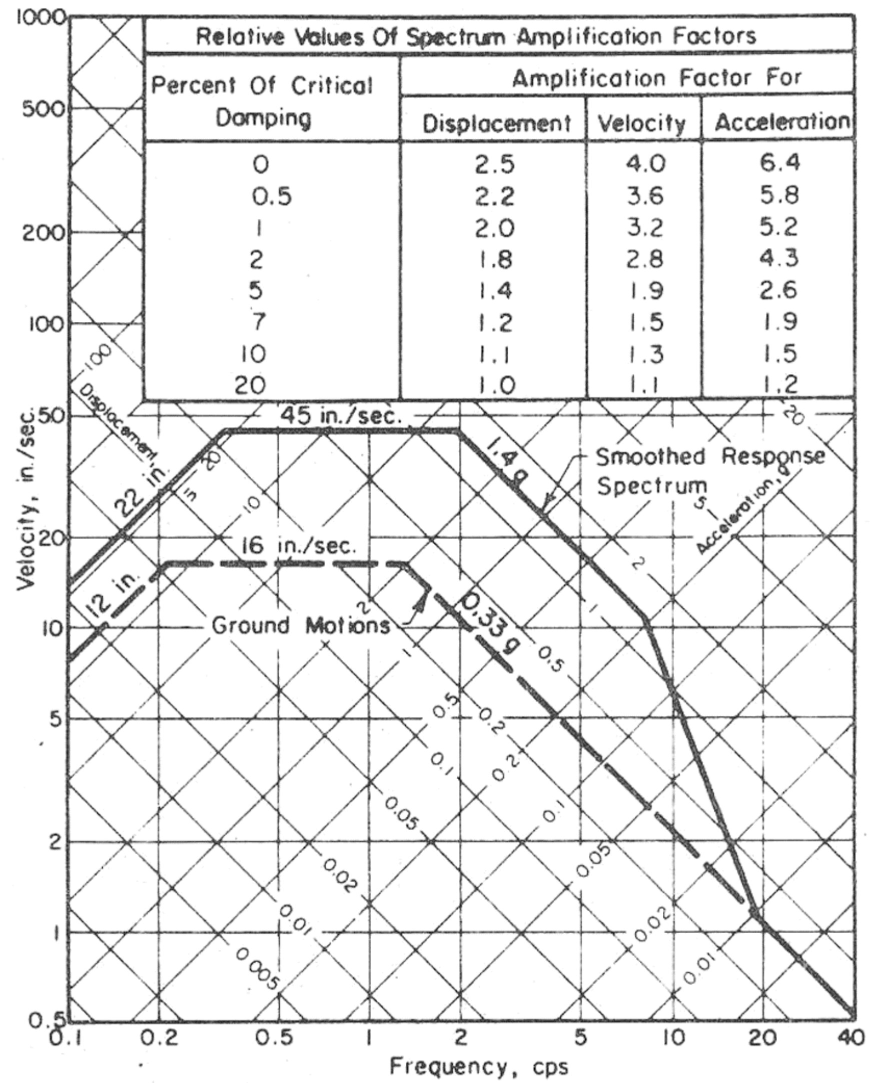
Fig. M-32.6



Response spectrum
El Centro earthquake 1940

from [10]

Fig. M-32.7



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

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

Fig. M-32.8

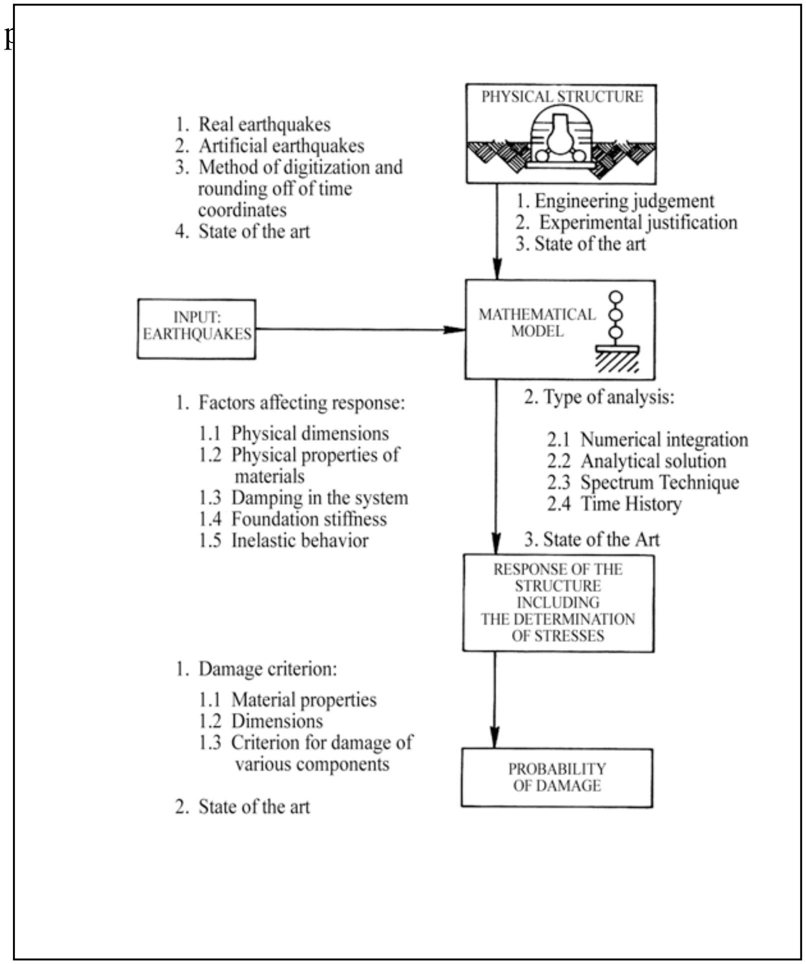


Fig. M-32.9

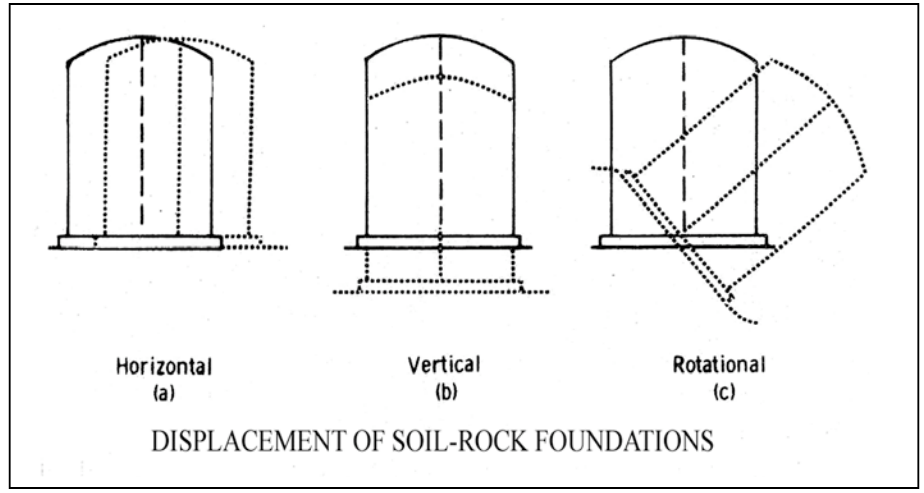


Fig. M-32.10

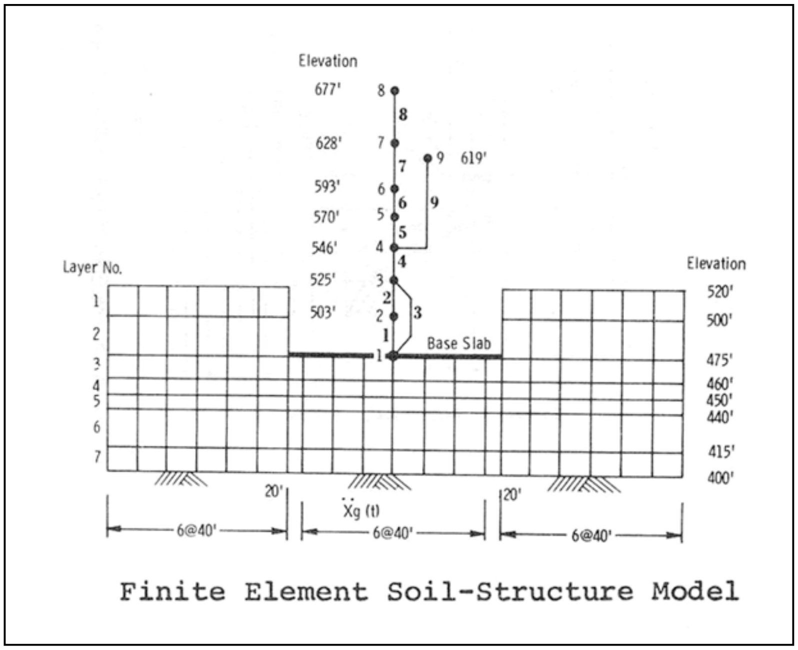


Fig. M-32.11

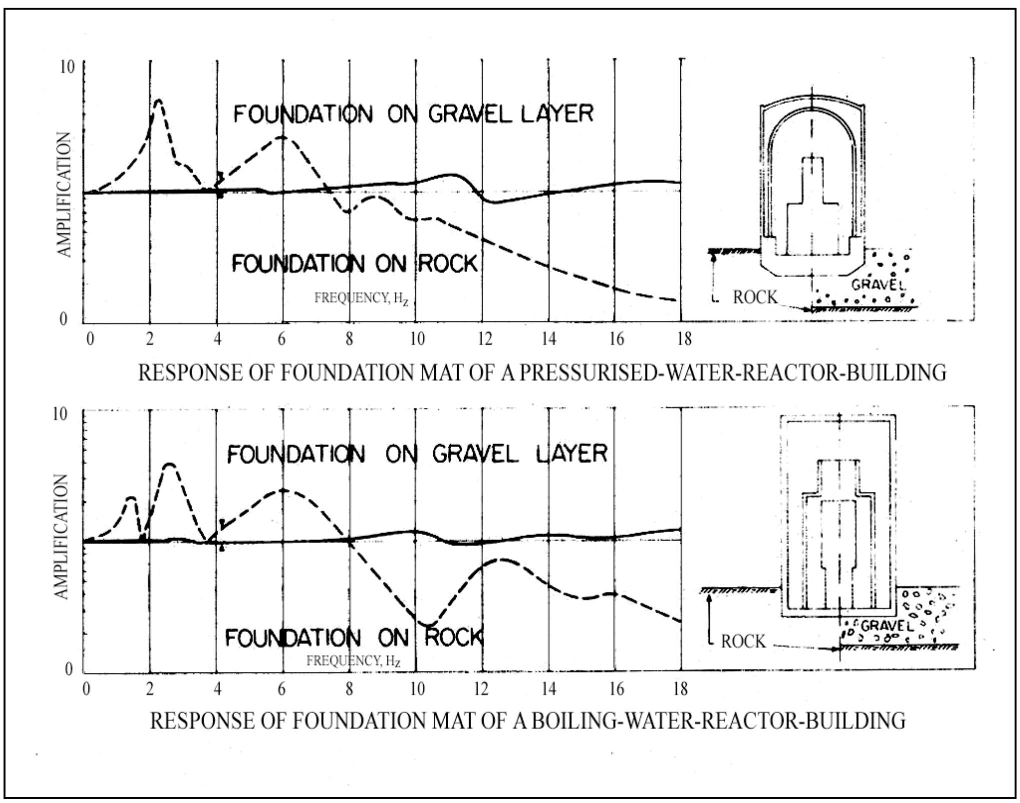
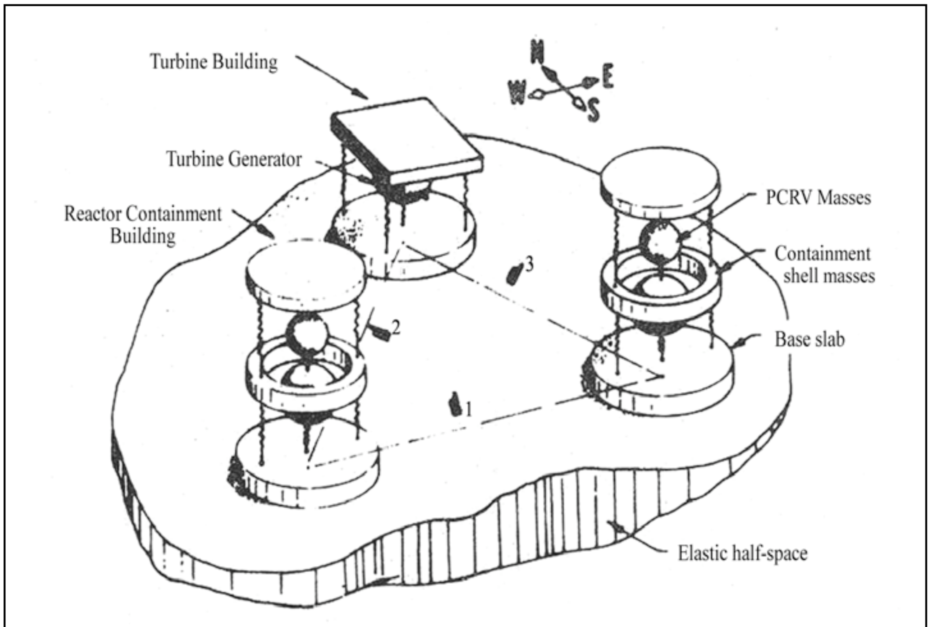
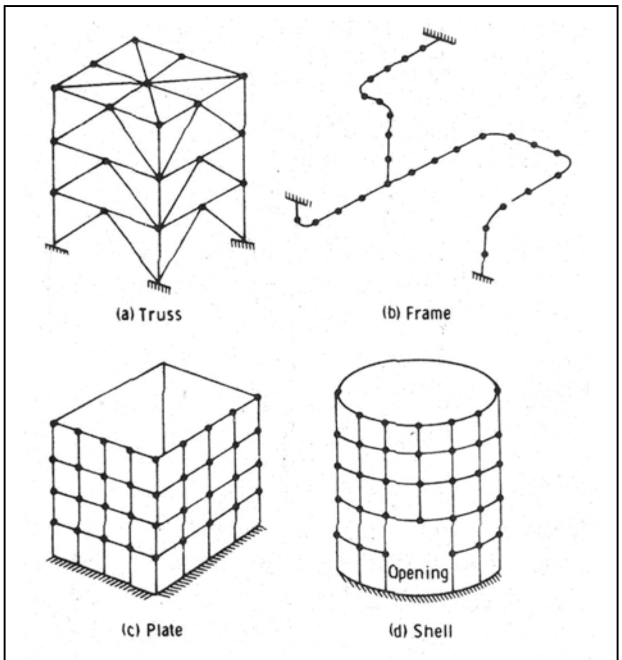


Fig. M-32.12



Three-Structure Model I Containing Two Reactor Containment Buildings and One Turbine Building

Fig. M-32.13



Discrete Lumped Mass Systems

Fig. M-32.14

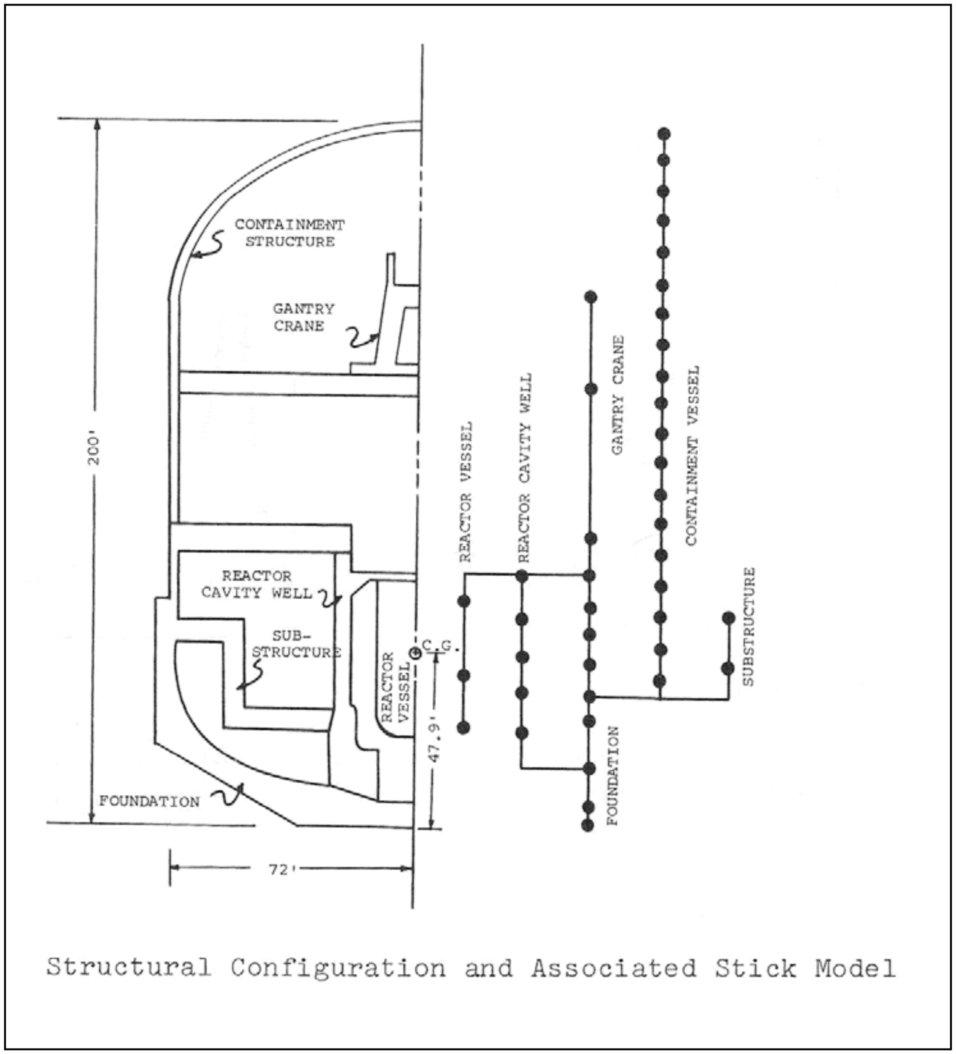


Fig. M-32.15

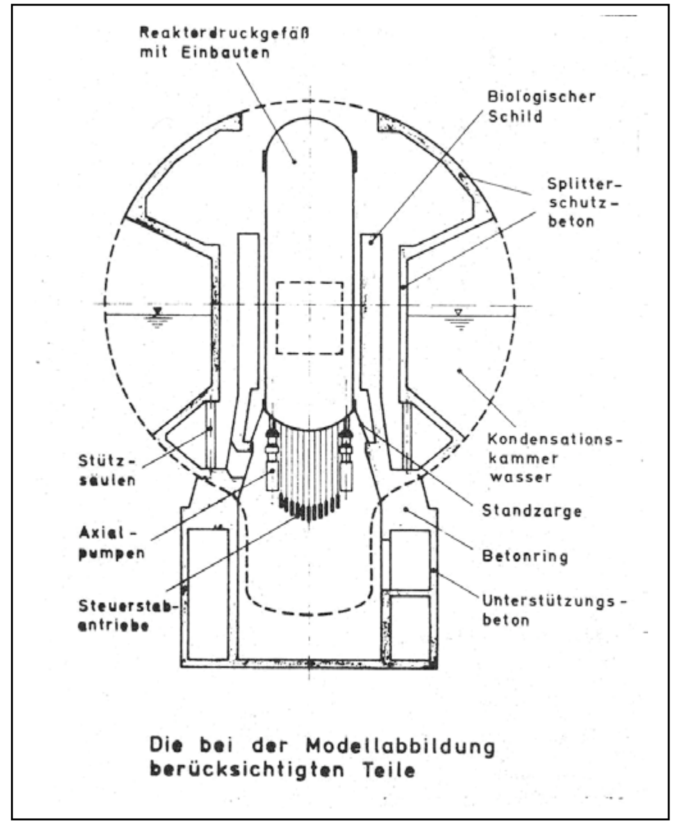


Fig. M-32.16

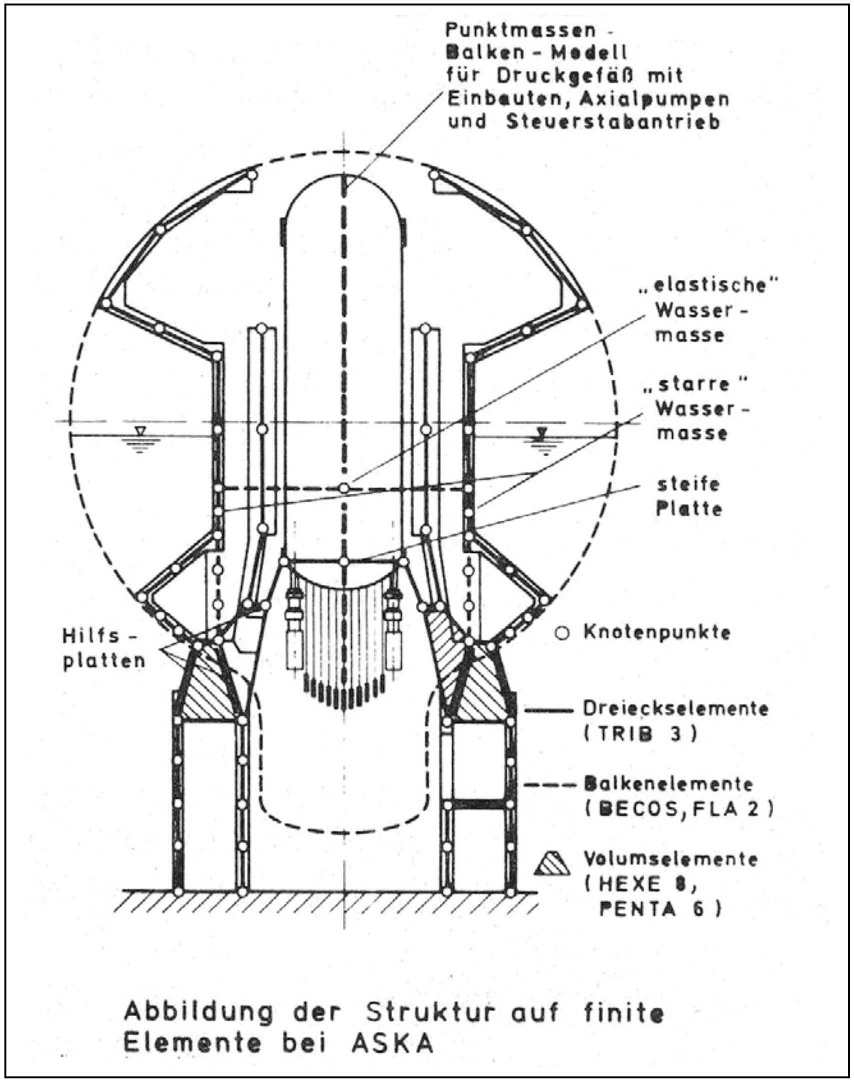


Fig. M-32.17

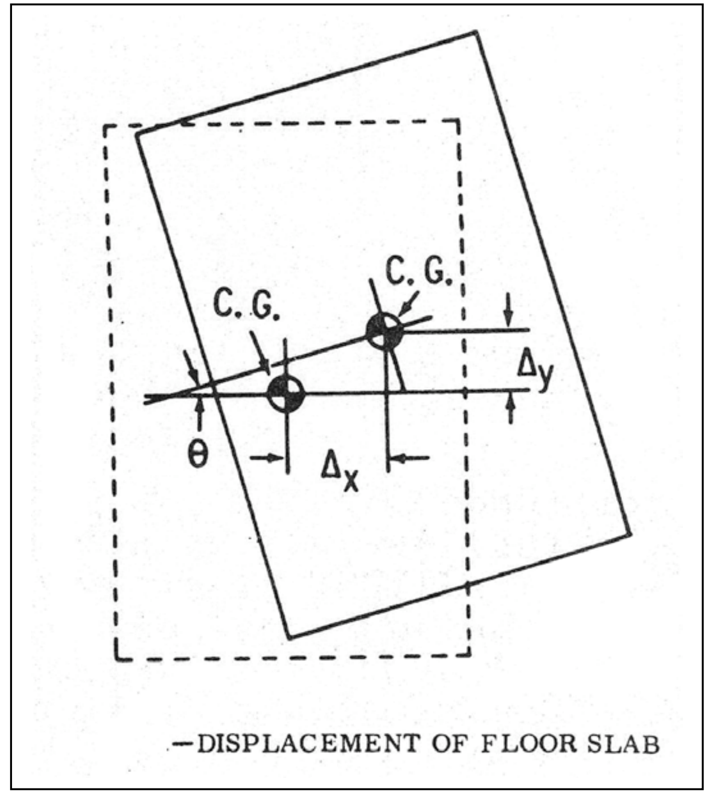
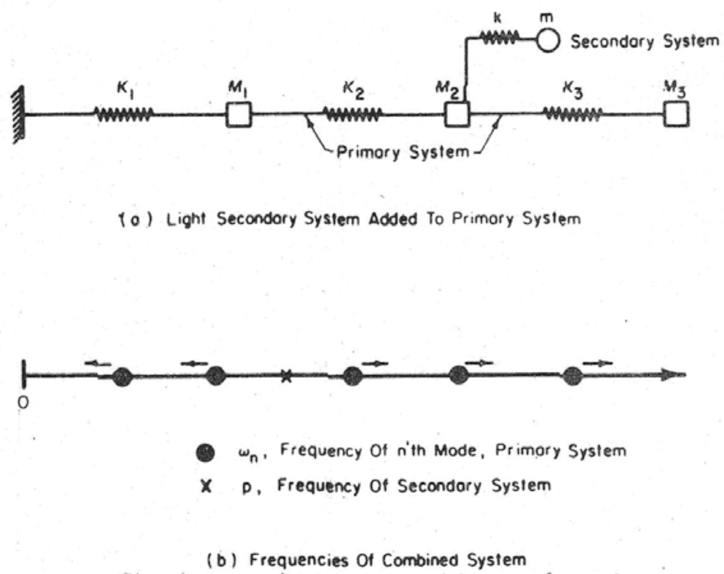
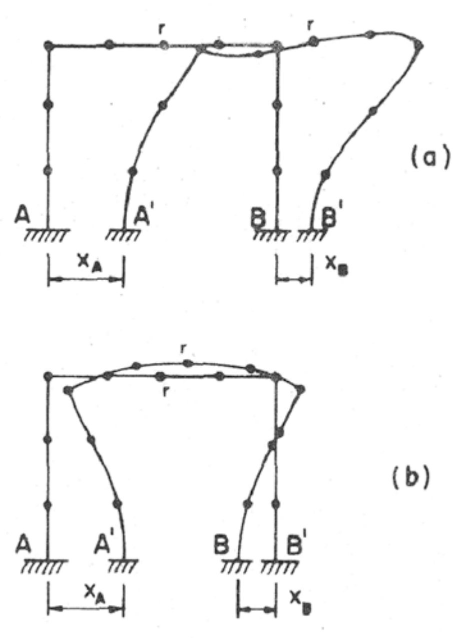


Fig. M-32.18



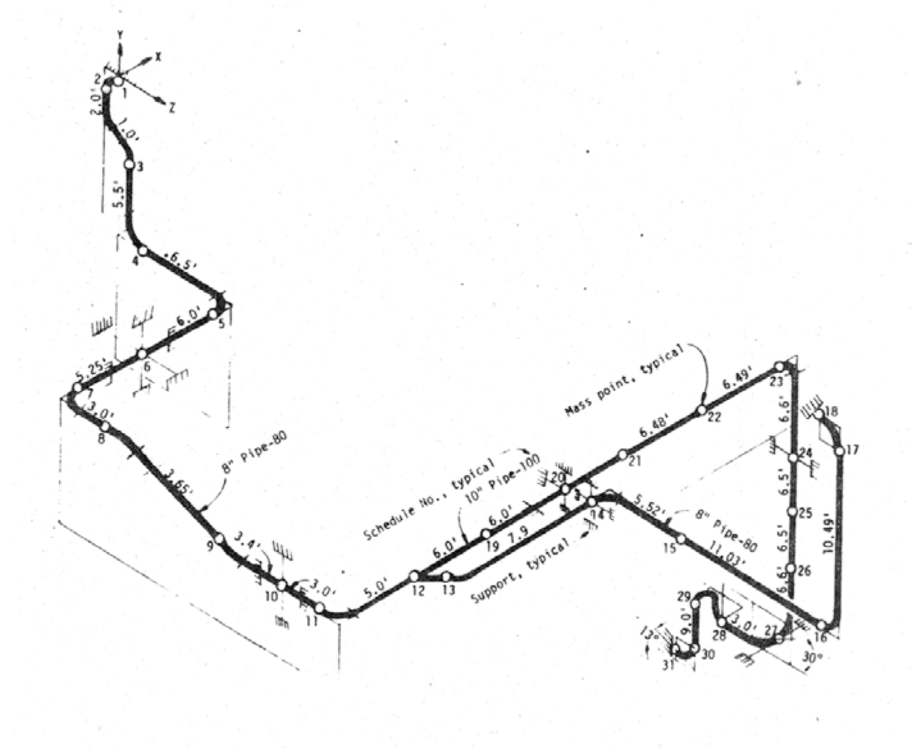
Simple secondary system with complex primary system.

Fig. M-32.19



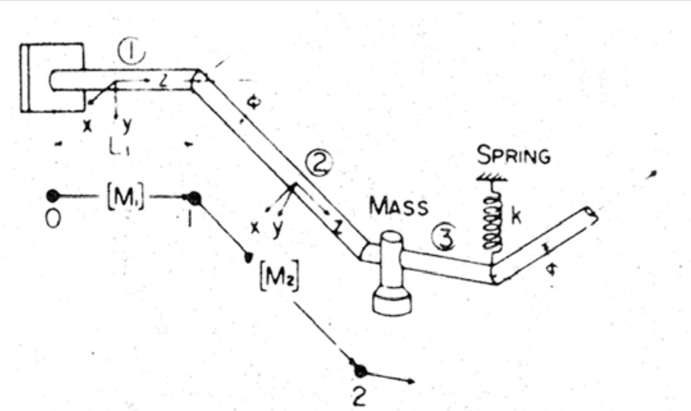
– SECONDARY SYSTEM SUPPORTED ON TWO PRIMARY SYSTEMS

Fig. M-32.20



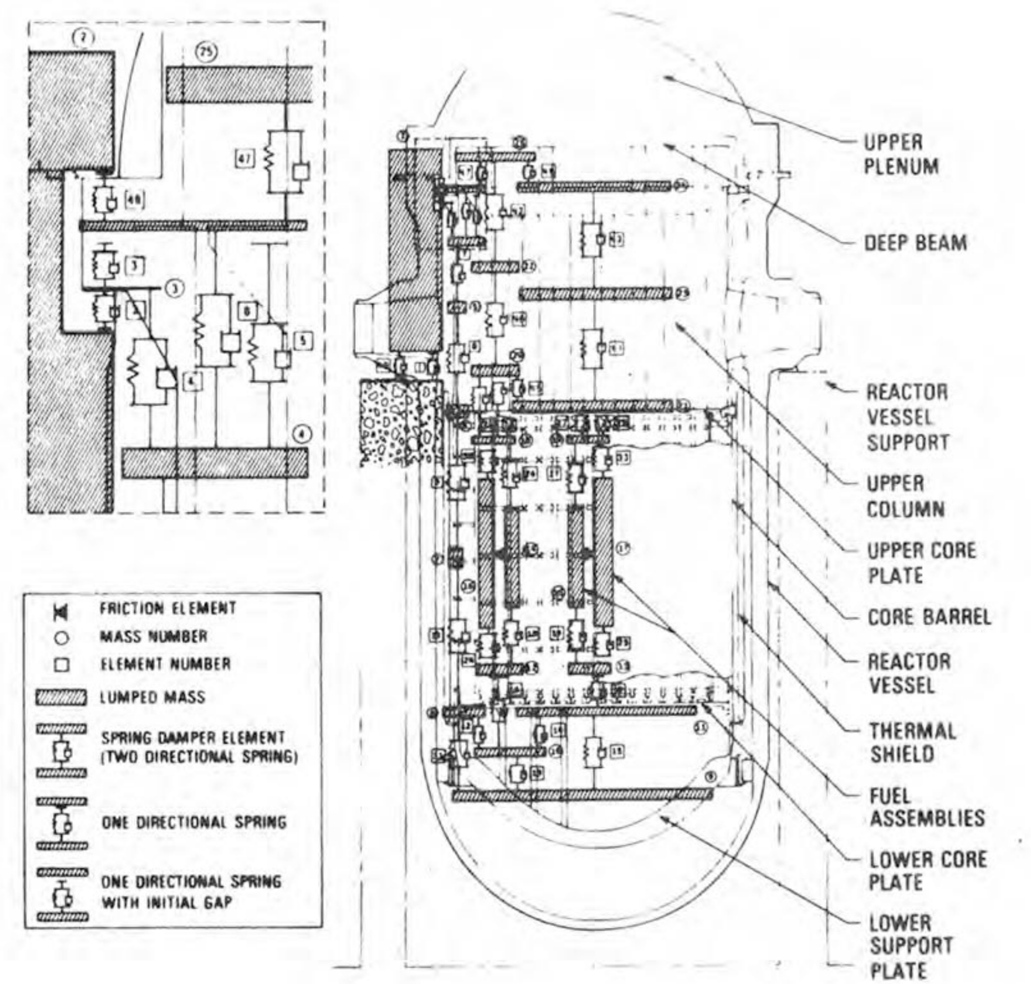
PIPING ANALYSIS EXAMPLE VII-3, MATHEMATICAL MODEL

Fig. M-32.21



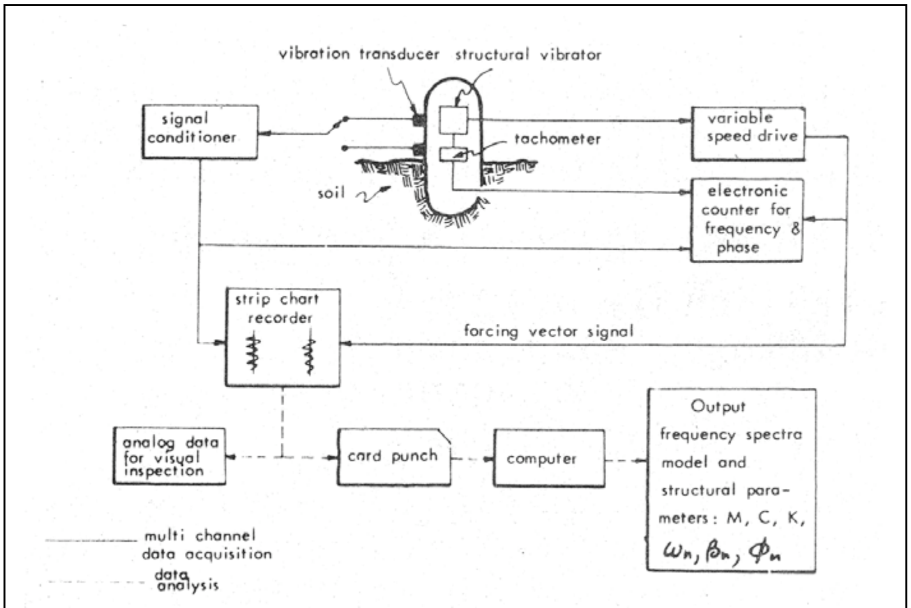
Schematic piping system and nomenclature.

Fig. M-32.22



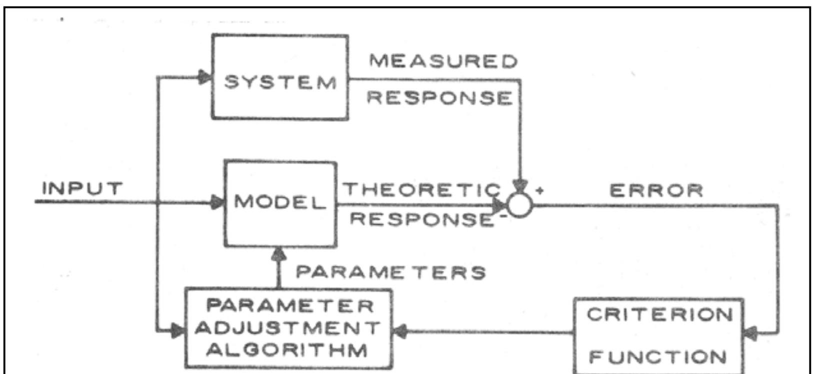
Reactor Internals Modeling

Fig. M-32.23



Current forced vibration test technique.

Fig. M-32.24



Parameter identification philosophy.

Fig. M-32.25