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State of the Art of Earthquake Engineering in Nuclear Power Plant Design

An Essay

P. O. Schildknecht

EIR Colloquium Lecture
Given December 2, 1976

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Abstract

A brief outline of definitions based on the USNRC, Seismic and Geologic Siting Criteria for Nuclear Power Plants, and on the plate tectonics and earthquake terminology is given. An introduction into plate tectonics and the associated earthquake phenomena is then presented. Ground motion characteristics are described in connection with the selection of design earthquakes. Mathematical methods of dynamic structural analyses are discussed for linear and nonlinear systems. Response analysis techniques for nuclear power plants are explained considering soil-structure interaction effects.

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1. Introduction

In Switzerland the first seismic analysis for nuclear power plants was required with the accomplished design of the first commercial plant. This prerequisite for the licensed operation of a nuclear reactor has eventually introduced earthquake engineering in Switzerland. Living in a very low seismic activity zone the structural engineers never faced the problems of the aseismic design of any of our conventional structures. People living in less secure places, however, dealt with these problems for quite a while. Primarily in California and Japan both seismologists and structural engineers developed considerable skill to be able to build earthquake resistant structures. This was not only necessary for conventional civil engineering design but also an absolute requirement for safe nuclear power plants. Obviously lacking the background in aseismic design we started in Switzerland to adopt the seismic design criteria for nuclear power plants set forth by the USNRC. However, scientists and engineers working in the nuclear business have to have a basic knowledge of earthquake analysis as one of the toughest safety measures in nuclear power plant design. In a research project the author was heading on "Underground Nuclear Power Plants" we have decided to clearly specify the state of the art in seismic research before attacking the seismic problems associated with underground structures. This essay is a first part of this state of the art submitting the essentials in earthquake engineering. An outline of this report was presented as a colloquium lecture at the Swiss Federal Institute for Reactor Research on December 2, 1976.

2. Background Information

2.1 USNRC Definitions

The USNRC seismic and geologic siting criteria for nuclear power plants require the design of a nuclear plant for two levels of earthquake ground motions: the safe shutdown earthquake (SSE) and the operating basis earthquake (OBE). These criterion earthquakes are typically obtained by correlating past earthquakes with known geologic structures or - in the absence of such features - with tectonic provinces. Definitions of magnitude, intensity, SSE, OBE, faults, capable faults and tectonic provinces are shown below (1), (2).

Magnitude

The magnitude of an earthquake is a measure of the size of an earthquake and is related to the energy released in the form of seismic waves. Magnitude means the numerical value on the Richter Scale.

Intensity

The intensity of an earthquake is a measure of its effects on man, on man-built structures and on the earth's surface at a particular location. Intensity means the numerical value on the Modified Mercalli Scale.

Safe Shutdown Earthquake

The SSE is that earthquake which is based upon an evaluation of the maximum earthquake potential considering the regional and local geology and seismology and specific characteristics of local subsurface material. It is that earthquake which produces the maximum vibratory ground motion for which certain structures, systems and components are designed to remain functional. These structures, systems and components are those necessary to assure:

a) the integrity of the reactor coolant pressure boundary,

b) the capability to shut down the reactor and maintain it in a safe shutdown condition, or

c) the capability to prevent or mitigate the consequences of accidents which could result in potential offsite exposures comparable to the guideline exposures of this part.

Operating Basis Earthquake

The OBE is that earthquake which, considering the regional and local geology and seismology and specific characteristics of local subsurface material, could reasonably be expected to affect the plant site during the operating life of the plant; it is that earthquake 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.

Fault

A fault is a tectonic structure along which differential slippage of the adjacent earth materials has occurred parallel to the fracture plane. It is distinct from other types of ground disruptions such as landslides, fissures and craters.

Capable Fault

A capable fault is a fault which has exhibited one or more of the following characteristics:

a) Movement at or near the ground surface at least once within the past 35,000 years or movement of a recurring nature within the past 500,000 years.

b) Macro-seismicity instrumentally determined with records of sufficient precision to demonstrate a direct relationship with the fault.

c) A structural relationship to a capable fault according to characteristics (a) or (b) of this paragraph such that movement on one could be reasonably expected to be accompanied by movement on the other.

Tectonic Province

A tectonic province is defined as a region having relatively consistent geologic structural features. The boundaries of the province are determined by a regional analysis of the geologic and tectonic environments, and the seismic history of the area (3). The concept of a tectonic province is particularly useful where earthquake epicenters cannot be related to known faults or to other specific major tectonic structures.

2.2 Plate Tectonics and Earthquake Terminology

Guyots

flat-topped submarine volcanoes

Darwin Rise

ancient now dead oceanic rise

Hot Spots

places where plumes of molten magma rise from the mantle

Fracture Zones

long thin bands of submarine mountains often perpendicular to oceanic rises

Plates

the lithosphere, defined as the rigid outer shell of the earth (roughly 100 km thick), is divided by a network of boundaries into separate blocks which are termed "plates"

Boundaries

are lines separating plates; they are of three types:

Ridges or Rises
where two plates are diverging, permitting the upwelling of magma that creates new lithosphere

Trenches or Sinks
where two plates are converging, with one plate moving beneath the other eventually to be absorbed into the mantle, or "destroyed"

Transform Faults
where two plates are moving tangential to each other; lithosphere is neither created nor destroyed; the direction of relative motion of the two plates is exactly parallel to the fault

Pole of Relative Motion

is the unique point between two plates on the globe that does not move relative to either of the two plates; the pole may be visualized as a pivot point about which the two plates rotate relative to each other

Focus, Center

point in the earth's crust where calculations indicate that the first seismic waves originated

Epicenter, Epifocus

vertical projection of the focus on the earth's surface

Epicentral Distance

distances from the epicenter to a given point of interest

Earthquake Swarms

earthquake swarms are a distinctive sequence of shocks highly grouped in space and time with no one outstanding principal event (often occurring with volcanic eruptions)

3. Seismological Background

3.1 History of Plate Tectonics and Sea-Floor Spreading

The observations made by H. H. Hess back in 1946 are considered to be fundamental in the development of modern plate tectonics (4). Hess's discovery of flat-topped submarine volcanoes termed guyots together with his interpretation were exciting to geologists. He stated that the guyots (a) form as volcanic islands near rise crests, (b) are truncated by wave erosion, and (c) then are submerged to depths of several kilometers as they migrate down the flanks of the rises (5). This stimulating idea demonstrated that the sea-floor is very mobile and is still a convincing piece of evidence that sea-floor spreading has occurred.

The research then of the 1950's served to focus attention on two important features: fracture zones and oceanic rises. Ewing and Heezen (6) discovered in 1956 that at the very crest of the ridges there is usually a central rift valley which Carey (7) interpreted in 1958 to be a narrow block sinking under tension as the sea-floor on either side of the valley moves apart, that is, as the sea-floor spreads. Contrary to the ridges of the Atlantic, Indian and Antarctic Oceans no central rift valleys were found on the Pacific mountain ranges. Even though both types of ridges are produced by sea-floor spreading, for reasons still not clear, central rift valleys do not tend to form on these rises where, as in the South Pacific, the spreading rate is high.

There is also evidence for the existance of ancient oceanic rises that are now "dead" in the form of a zone of guyots extending northwestward across the Pacific. Menard's (8) explanation in 1964 was that (a) the ocean floor bulged upward due to rising mantle convection which did not, however, produce ocean-floor spreading; (b) volcanic islands formed and were truncated; and (c) the bulge subsided as convection stopped. Menard termed this ancient ridge system the Darwin rise. Hess's (9) explanation in 1962 was similar but included sea-floor spreading in a direction perpendicular to the northwest trend of the rise. A new interpretation by Morgan (10) in 1971, and one that is stimulating much contemporary research, is that the band of guyots marks "hot spots" in the floor of the Pacific Ocean, that is, where plumes of molten magma rise from the mantle.

Another most important result was the discovery of fracture zones in the Pacific in 1952 by Menard and Dietz (11). Fracture zones are long thin bands of submarine mountains parallel to the general zone trend and often perpendicular to oceanic rises. Fracture zones usually separating regions of different depth, trace paths that are segments of circles. Plate tectonic theory uses fracture zones to determine the direction of movement of plates.

In his classic paper Hess (9) suggested in 1962 that the continents do not plow through the oceanic crust, as was believed in the continental drift theory, but are carried passively on a mantle that is overturning due to thermal convection. The mid-oceanic rises are places where hot mantle material rises to form new crust, the topographic elevation of the rises being due to the lower density of the hotter rocks. Evidence for the so-called deep structure of continents such as the seismic zone beneath the west coast of South America actually points more toward a process than a structure. The process is downward movement on the descending limb of the convection cell. Dietz (12) coined the term "sea-floor spreading" to describe the overall process.

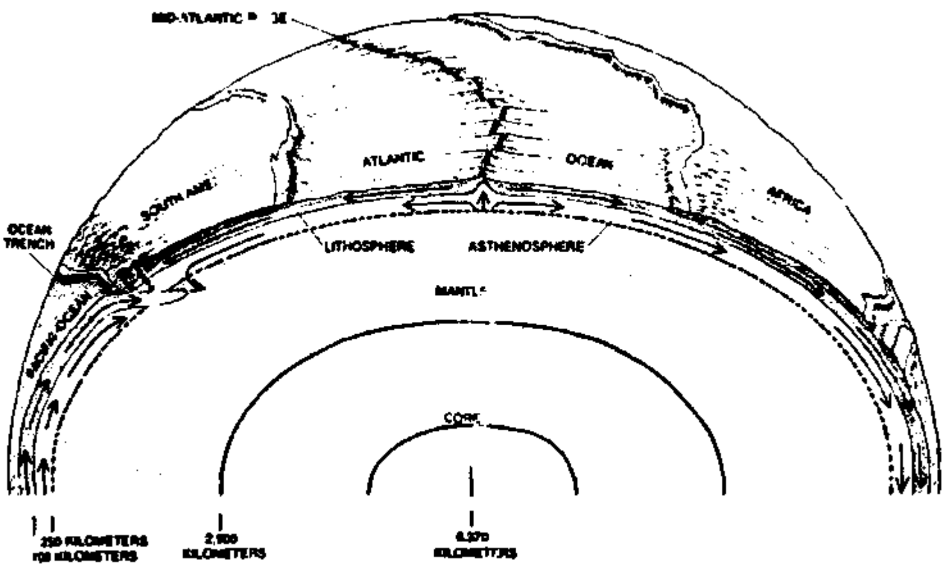

Fig. 1 Dynamic of the earth based on the theory of plate tectonics (after Wyllie, Ref. (22))

By late 1964 the stage was set at four institutions for the birth of the new theory of plate tectonics. The four institutions were Cambridge University in England, Princeton University in New Jersey, Lamont Geological Observatory in Palisades, New York, and Scripps Institution of Oceanography in La Jolla, California (4). With the theory of plate tectonics the additional step was taken of proceeding beyond the qualitative concept of ocean floor spreading to the quantitative calculation of the direction and velocity of plate movements. This was done by making two idealized geometrical postulates about crustal deformation (4).

Postulate 1 The plates are internally rigid but are uncoupled from each other. At their boundaries two plates may pull apart or slip one beneath the other, but within the plates there is no deformation.

Postulate 2 The pole of relative motion between a pair of plates remains fixed relative to the two plates for long periods of time.

The statement of Postulate 2 does not hold, however, for three or more plates on a sphere and still remains a basic contradiction of plate tectonics.

The following three theorems follow from Postulate 2:

Theorem 1 Transform faults between two plates lie along segments of concentric small circles centered on the pole of relative motion of the two plates.

Theorem 2 The pole of relative motion for two plates may be found by constructing perpendiculars to local segments of transform faults. The common intersection of the perpendiculars is the pole.

Theorem 3 The width W of new lithosphere formed adjacent to a ridge in a given interval of time decreases from a maximum width W_o at an arc distance $A = 90^o$ from the pole of relative motion to zero width at the pole itself. Quantitatively, $W = W_o \sin A$, where A is the arc distance from the pole to the point of observation and W is the width of new litosphere measured parallel to the direction of relative motion between the two plates.

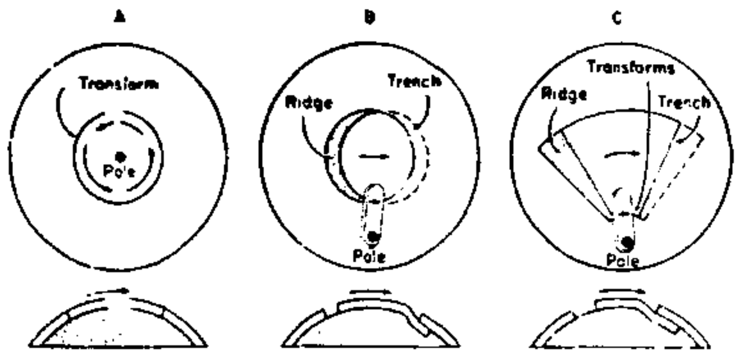

Fig. 2 The tennis ball experiment visualizing the theory of plate tectonics (after Cox, Ref. (4)). (A)A circle is cut from the tennis ball and pivoted to rotate about its center (solid dot), which is the pole of relative motion. (B) The cut-out circle is attached to a rigid arm which pivots about a new pole (solid dot), creating a ridge and trench but no transform faults. (C)A transform fault is created by cutting the upper boundary of the small plate along a circle concentric about the pivot point (i.e., pole)

3.2 Seismicity

Long before these basic postulates and theorems of plate tectonics have been formulated, seismologists recognized that a large portion of the world's earthquakes, including almost all of the deep ones, occur near the deep oceanic trenches. Furthermore it is clear from the literature of the time that seismologists were becoming increasingly aware of the planar distribution of earthquake foci beneath oceanic trenches. It was Benioff (13) who concluded in his work between 1949 and 1954 that earthquakes originated on great faults that dipped under the continents, the oceanic deeps being the result of downwarping of the oceanic blocks. All the circum-Pacific oceanic deeps are now called Benioff-Zones.

Nakano (14) and Byerly (15) developed in the 1920's a new way of analyzing earthquakes that, decades later, was to make a very important contribution to plate tectonics. Their idea was a simple one (4). Consider first an explosion taking place in a cavity. The first waves caused by it arriving at a distant station will be compressional and the first motion of the ground will be in a direction away from the source. Consider next elastic waves resulting from a collapsing cavity. These waves will be dilatational and the first motion observed would be toward the source. The two seismologists found that the elastic waves generated by an earthquake are more complicated than the two sorts of wave just described, with both compressions and dilatations being generated by a single earthquake. Imagine a sphere drawn around the point on a fault where the earthquake motion starts. Pass two perpendicular planes (called nodal planes) through the center of the sphere, dividing the sphere into quadrants. Then the motion in two diagonally opposed quadrants will be compressional and the motion in the other two will be dilatational. It was demonstrated that one of the nodal planes is parallel to the plane of the fault, while the other one is perpendicular to the slip direction (Fig. 3). Therefore the seismologist is able to determine from first-motion studies the orientation of faults and the slip directions of earthquakes, even where the actual faults are inaccessible to direct observation.

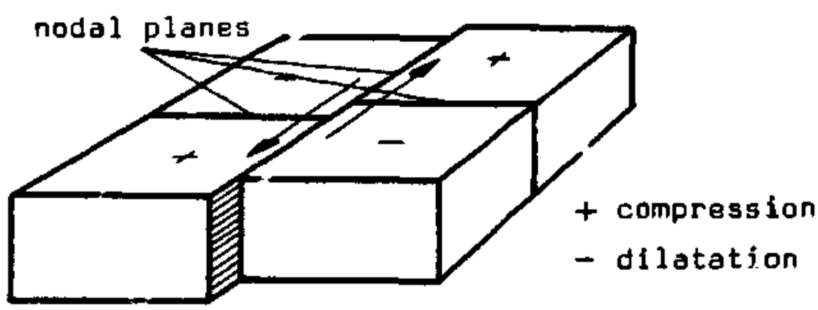

Fig. 3 Fault mechanism of earthquakes

Slippage on the other hand was also considered to be preceded by shear stress concentrations along the fault plane. This led to the elastic-rebound theory of earthquake generation investigated by Reid (20) along the San Andreas fault during the San Francisco earthquake of 1906. The large-amplitude shearing displacements which resulted from this earthquake for many miles along the fault led him to conclude that the specific source of the earthquake vibration energy is the release of accumulated strain in the earth's crust, the release itself resulting from the sudden shear-type rupture.

Earthquake occurrence maps demonstrate an agreement with these plate tectonic theories in the sense that the plate boundaries coincide with a high density of the characteristic activities (Fig. 4).

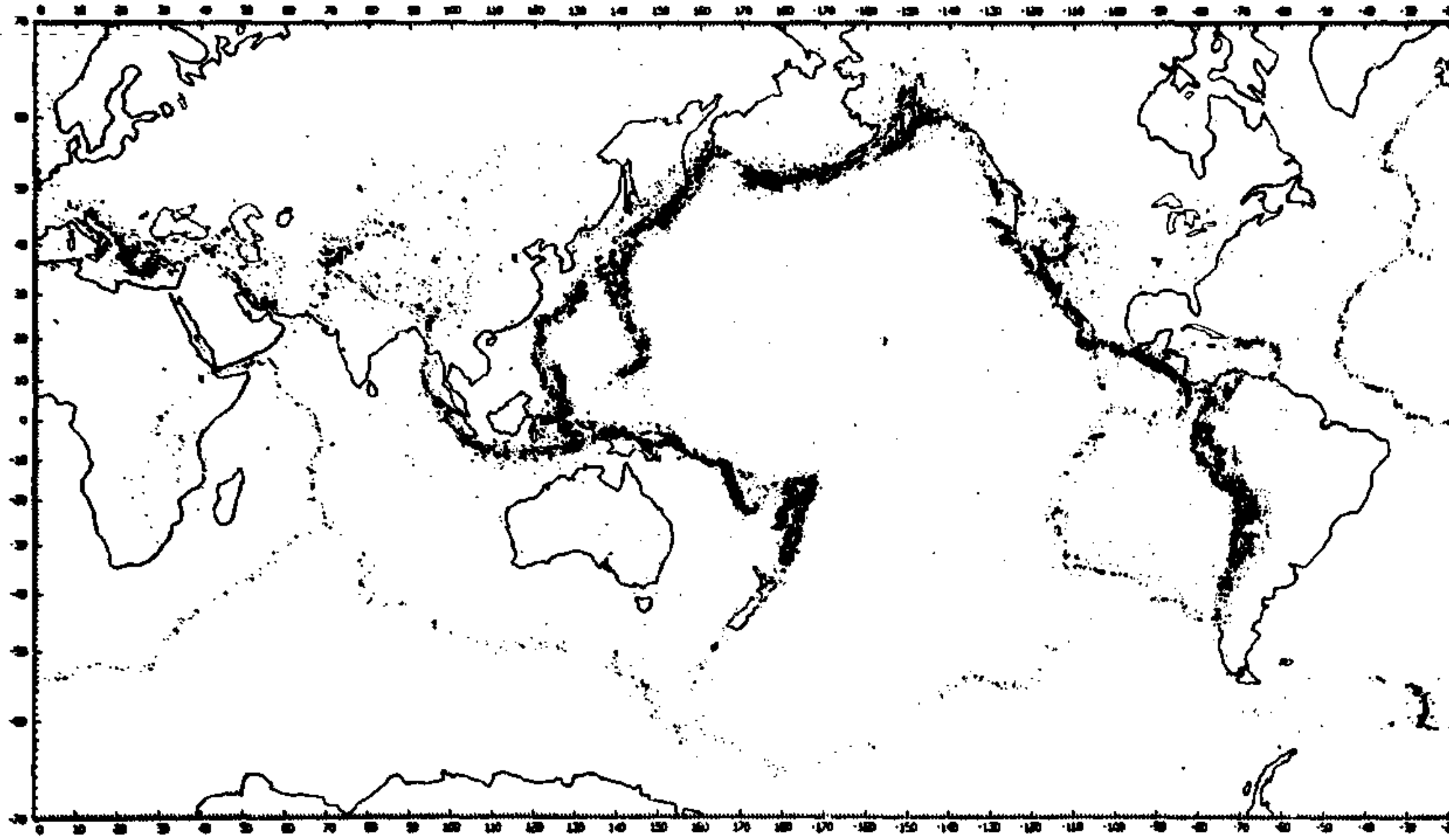

Fig. 4 Worldwide distribution of all earthquake epicenters for the period 1961 through 1967 as reported by U.S. Coast and Geodetic Survey (after Barazangi and Dorman, Ref. (23))

Three principle zones are clearly shown on this map, the circum-Pacific belt around the Pacific Ocean, which includes the great majority of earthquakes, the Alpide belt, which extends from the Himalaya mountain range through Iran, Turkey and the Mediterranean Sea, and the Atlantic belt in a north-south direction through the center of the Atlantic Ocean, being of less interest to the structural engineer due to the belts marine location.

The shown earthquakes for a period of time of only 7 years cannot be considered an adequate statistics of occurrence if a structure is to have a design life of say 30 years. This is especially true for low seismic activity areas like Switzerland, which for such a short interval might even fall into a nonseismic region.

Switzerland, being located just north of the earthquake belt through the Mediterranean, has in many regions a very scattered fault system. In most active seismic regions like California secondiary faults are associated with big faults e.g. the San Andreas fault. Consequently the source of most major California earthquakes is rather well specified while the locations for Swiss earthquakes are difficult to group due to the sparse fault pattern. The fact is that in Switzerland active faults are little known and a search is justifiable from an economical point of view only for specific sites. Fig. 5 shows a map of earthquake occurrences during the last 100 years with some historic events.

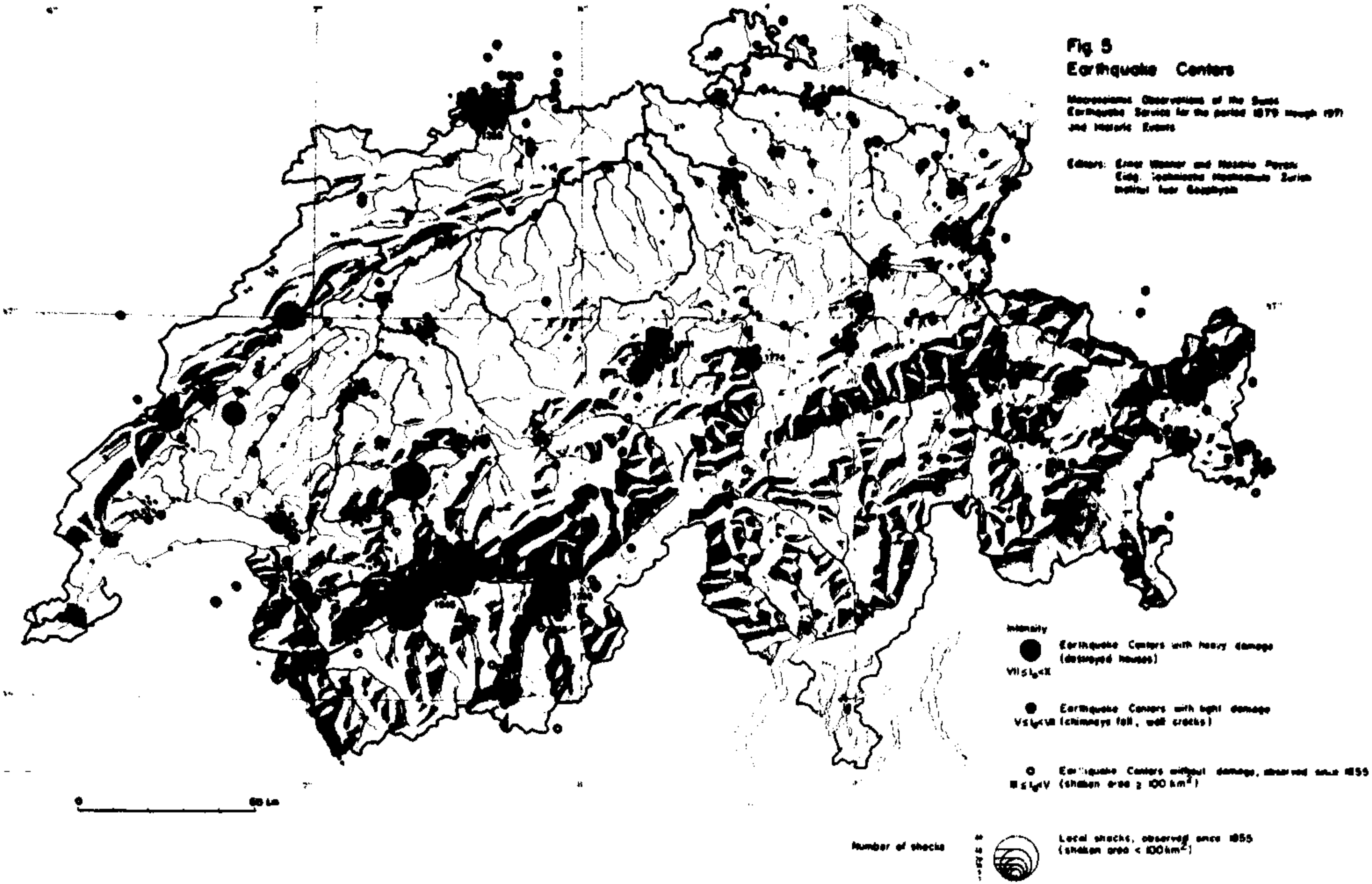

Fig 5
Earthquake Centers

Macroseismic Observations of the Swiss Earthquake Service for the period 1879 through 1971 and Historic Events

The magnitude of an earthquake is universally measured on the Richter scale (16). The Richter magnitude (denoted by M) is the logarithm of the trace amplitude, in microns (thousandth of a millimeter), of a standard Wood-Anderson seismograph having a magnification of 2800, a natural period of 0.8 sec, and a damping coefficient of 80 percent and with the seismograph located on firm ground 100 km from the epicenter. This can be written as

$$M = \log (A/A_o) \qquad (1)$$

with A being the amplitude and A_o being the amplification parameter.

Empirical charts and tables are available to correct for epicentral distances different from 100 km and for various conditions of the ground. Since the Richter magnitude scale is limited to focal depths smaller than about 30 km and epicentral distances up to about 600 km, the teleseismic scale (also denoted by M) is applied for focal distances greater than 600 km. Like the Gutenberg unified scale (denoted by m) it is determined from the amplitudes and periods of certain earthquake phases. The following empirical relation between magnitudes and energy release is at present favored by seismologists

$$\log E = 11.8 + 1.5\ M \qquad (2)$$

where M is the magnitude and E is the energy release in ergs (Gutenberg and Richter, 1956).

Almost all intensity scales are subjective and similar in structure to the modified Mercalli (MM) scale (see Appendix 1). The MM scale is widely used in North America while for other regions different scales are applied. In the Western European region the more detailed scale of Medvedev, Sponheuer, and Karnik (MSK) is favored. A comparison of the various scales is shown in Fig. 6 from Ref. (17).

Characteristics of the earth-quake (abbreviated)	European MSK scale 1964	Sowjet scale 1952	American MM scale 1931	Japanese scale 1950	Rossi Forel scale 1873	European MCS scale 1917
recorded by seismo-graphs only	1	1	1	0	1	1
felt by people at rest	2	2	2	1	2	2
felt by some people	3	3	3	2	3	3
felt indoors, dishes and windows rattle	4	4	4	2,3	4	4
objects begin to swing sleepers wakened	5	5	5	3	5,6	5
weak plaster and ma-sonry cracked	6	6	6	4	7	6
cracks in plaster, walls and chimneys	7	7	7	4,5	8	7
broken walls, roofs fall appart	8	8	8	5	9	8
on some buildings walls and roofs fall, landslides	9	9	9	6	10	9
destruction of many structures, cracks in the ground up to 1 m wide	10	10	10	6	10	10
many cracks in the ground, large land-slides in the moun-tains	11	11	11	7	10	11
large deformations on the earth's sur-face	12	12	12	7	10	12

Fig. 6 Comparison of earthquake intensity scales

For a uniform classification of all earthquakes, however, the relation between the instrumental quantity M (magnitude) and the macroseismic quantity I (intensity) must be established. This relation is necessary for the earthquake magnitude determination where no amplitude record of the propagating waves exists or where useful microseismic data is available. Therefore this relationship can be applied to all events prior to the year 1904 and to the many weak earthquakes. A summary of the many available magnitude-intensity relations is given by Karnik (18). Seismologists in Switzerland are using the formula of N.V. Shebalin in a simplified form

$$M = 0.67\ I + 0.2 \qquad (3)$$

It is worthwhile to compare our region with California where a simplified Gutenberg-Richter formula

$$M = 0.67\ I + 1.0 \qquad (4)$$

is used. Note that the second term is dependent on the depth of the earthquake. For Switzerland there are in fact no strong motion records available. However, the recent Friuli (Italy) earthquake (May 6, 1976) produced accelerograms at an epicentral distance of 12 km which can be considered a first approximation for the Swiss region. These records of course have to be modified according to the local geological effects. Fig. 7 shows according to (19) the accelerograms of the Friuli earthquake. Note the vertical acceleration component with its high frequency and a relatively small amplitude over a short period of time.

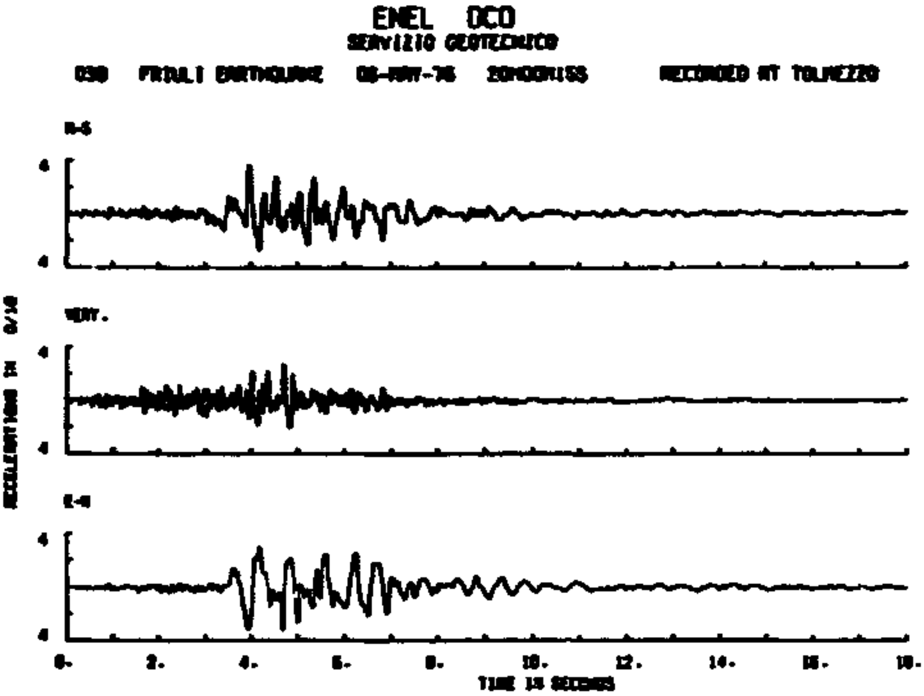

Fig. 7 Accelerogram of the May 6, 1976 Friuli Earthquake, (from Ref. (19))

The question that always arises is, what is the frequency of an earthquake event to occur? Karnik (18) has compiled statistical data for the European area. His statistics cover the period from 1901 till 1955 including data for earthquakes with an intensity I = 6 (equivalent to M = 4.1 to 4.9 for different zones) and higher. According to Karnik's terminology region 18 (which is representative for Switzerland) shows events up to the M class of 5.7 to 6.2.

Based on the original Gutenberg-Richter formula the magnitude-frequency relation can be expressed by

$$\log N = a - bM \qquad (5)$$

with M being the magnitude and N being the frequency of occurrence. The coefficient a depends on the period of observa-

tion, on the size of the investigated area, and on the level of seismic activity as well. The coefficient b is believed to depend on the depth of the shock.
With b = 1.0 for intermediate depths of the shocks we find for region 18 an approximate a = 6.5.

3.3 Earthquake Waves

The release of energy at the focus of an earthquake produces two types of body waves. The dilatation waves which are called P (primary) waves and the slower propagating shear waves which are called S (secondary) waves. P waves can be transmitted through both solids and liquids while S waves can be transmitted only through solids, materials that can sustain shear stresses that is. The excitation force of a P wave is acting in the direction of the wave propagation. The excitation force of an S wave is acting perpendicular to the direction of the wave propagation. It was found that the velocity of earthquake waves increases progressively with depth. Since S waves never propagated beyond the core-mantle boundary it was concluded that at least the outer part of the core is liquid. By study of the arrival times of waves from a common source recorded at many stations widely distributed around the earth, the position of the source and the pattern of wave reflections and refractions from internal material discontinuities can be derived. Nearly all what is presently known about the interior structure of the earth has been deduced from such seismological observations. Teleseismic records of earthquake waves like these, however, have little significance in structural engineering design, because of the infinitesimal amplitudes of the ground motions involved. It is only in some neighborhood of the focus that the motions are large enough to cause structural damage. If the earthquake results from a sequence of ruptures along a fault line, it is evident that the focus may not coincide with the center of energy release. In this case the significant factor is the distance to the nearest point along the rupture surface rather than the distance from a structure to the epicenter.

3.4 Ground Motion Characteristics

The earthquake engineer is primarily interested in the effects earthquake ground motions will have on structures. The damage potential, which is due to excessive stresses and deformations in structures, depends on the amount of strain energy released at the source of the earthquake. As was indicated earlier there is an empirical relation between magnitudes and energy release in the form of equation (2). Another important empirical observation is that only for magnitudes greater than 5 potentially damaging ground motions are produced.

Only when strong motion records from accelerographs became available the earthquake engineer was in a position to get the characteristics of earthquake motions necessary for a dynamic structural analysis. The accelerogram of Fig. 8 was one of the early recordings.

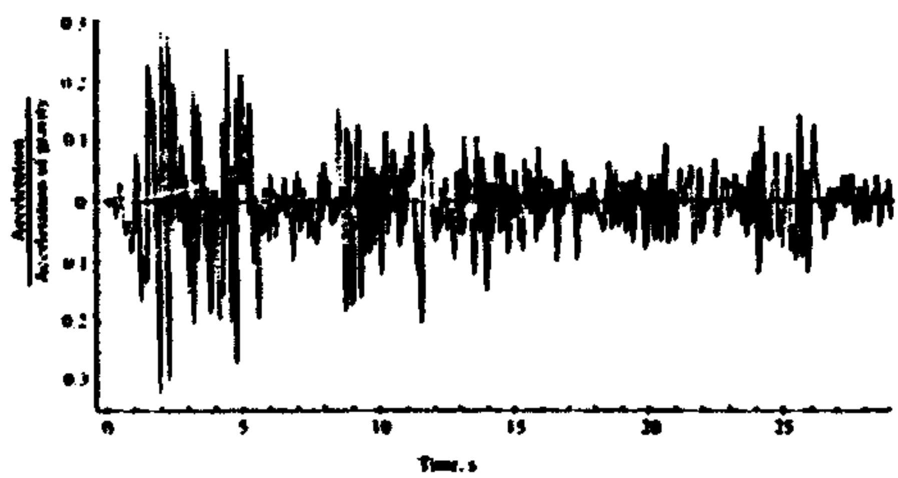

Fig. 8 Accelerogram from El Centro earthquake, May 18, 1940 (NS component), (from Ref.(21))

The most important features of the record obtained from each component (such as the one in Fig. 8) in producing structural response are the amplitude (peak values), the frequency content, and the duration. A more precise measure of the intensity of a given ground motion, than these quantitative measures, can be obtained by a simple oscillator. The response of a single degree of freedom oscillator to a specified ground acceleration $\ddot{v}_g(t)$ may be expressed by means of the Duhamel integral (neglecting the difference between the damped and the undamped frequencies, assuming ξ < 20 percent)

$$v(t) = \frac{1}{\omega} \int_0^t \ddot{v}_g\ (\tau)\ e^{-\xi\omega(t - \tau)} \sin \omega\ (t - \tau)\ d\tau \qquad (6)$$

with v = displacement

t = time

ω = natural circular frequency

$\ddot{v}_g$ = ground acceleration

τ = time t when load is applied and response starts

ξ = damping ratio

Considering the maximum response displacement relative to the ground to be the measure of the earthquake intensity it can be written as

$$v(t)_{max} = \frac{1}{\omega} S_v \qquad (7)$$

in which S_v (ξ, ω) is the integral term (max. value) of eq. (6) and is called the spectral pseudo-velocity response of the ground motion $\ddot{v}_g(t)$. S_v thus depends not only on the ground motion history but also on the frequency of vibration and the damping of the oscillator. Assuming a specific value of damping in the structure it is possible to calculate values of S_v for a full range of vibration frequencies. A graph showing these spectral velocity-response values plotted as a function of frequency (or of the reciprocal quantity, period of vibration), as shown in Fig. 9, is called a pseudo-velocity response spectrum of the earthquake motion.

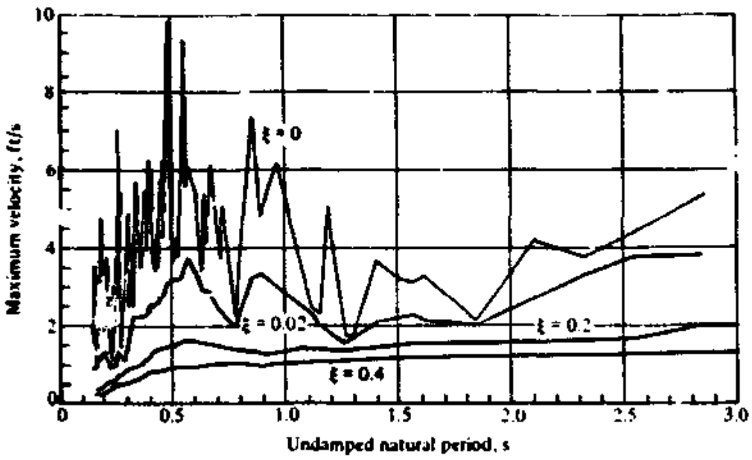

Fig. 9 Pseudovelocity response spectrum, El Centro earthquake, May 18, 1940 (NS component), (from Ref. (21))

The spectral displacement S_d can be found from eq. (7) and together with the spectral acceleration S_a yields the following relationship

$$S_a = \omega S_v = \omega^2 S_d \tag{8}$$

This simple relationship between these three quantities make it convenient to draw them in a single plot on four-way log paper (angle at 45^o) as shown in Fig. 10.

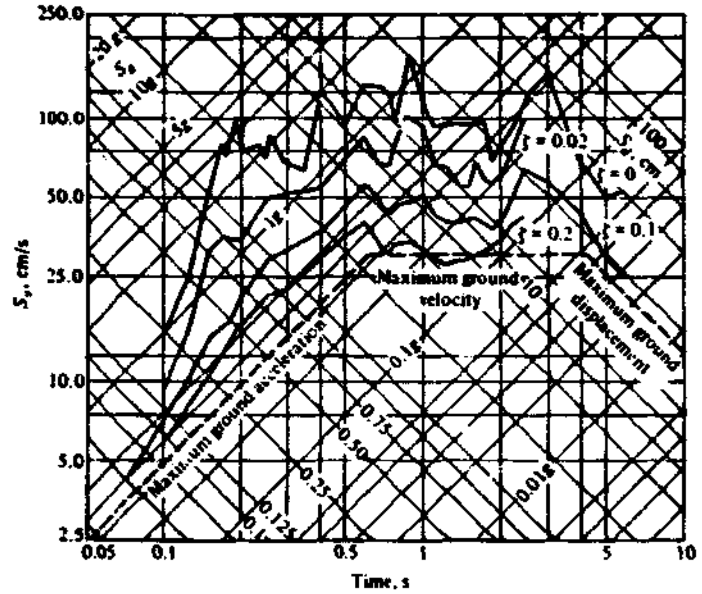

Fig. 10 Response spectra for El Centro earthquake, 1940, (from Ref.(21))

3.5 Selection of Design Earthquakes

Structural analysis requires this thorough study of seismology described in this chapter for the dynamic load of a big earthquake may cause higher stresses and larger deformations in various critical components than are expected from other critical load combinations. On the other hand the probability of such an earthquake event to occur during the life of a structure is very low. This extreme loading with low probability led in general to a dual analysis based on the following criteria:

1. Reasonably expected earthquake during the life of a structure at a specific site with the structure and its components to resist the intensity of the ground motion and to maintain its functionability; for nuclear power plants the OBE (see chapter 2).

2. Most severe earthquake which could occur at a specific site with a very low probability within the life of a structure causing significant damage, however, with the structure and certain components to maintain some functionability; for nuclear power plants the SSE (see chapter 2).

It is thus important to establish the ground-motion characteristics of a design earthquake based on the statistical data available for a specific site. The usually sparse seismological data such as magnitude and frequency of occurrence should be supplemented by geological evidence such as active faults. One way to specify the expected ground motion is to use an accelerogram of a past earthquake with a proper magnitude which was recorded at an appropriate distance.

However, big differences between the records of earthquakes with similar magnitudes and distances from the source and the even more scattered corresponding structural responses highly question the usefulness of a single record defining a design earthquake. Consequently average earthquake response spectra have been developed, for example by Housner (25), using different earthquake records and normalizing, averaging, and smoothing the resulting curves. Design response spectra can also be constructed from estimates of peak values of the maximum ground-motion acceleration ($\bar{a}$), velocity ($\bar{v}$), and displacement ($\bar{d}$). Newmark and Rosenblueth (24) have developed such a smoothed design response spectrum, shown in Fig. 11, assuming that the corresponding maximum response values for a slightly damped oscillator are about $4\bar{a}$, $3\bar{v}$, and $2\bar{d}$ and also knowing that the acceleration response tends toward $\bar{a}$ for very stiff (short-period) structures while the displacement response tends toward $\bar{d}$ for very flexible (long-period) structures.

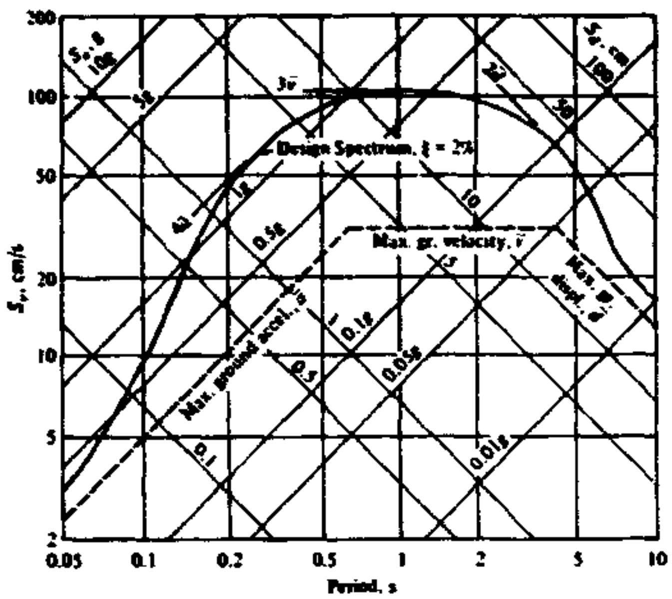

Fig. 11 Design response spectrum constructed from maximum ground-motion characteristics. (After Newmark and Rosenblueth, Ref.(24))

It is important to note that a structural response analysis based on a design response spectrum is possible only for strictly linear material behavior. Nonlinear response of a structure must be based on an actual time-history record. In order to obtain appropriate ground-motions from actual records amplitude-scaling factors, frequency content modifications by changing the time scale, and change in the duration of the earthquake were applied effectively to adjust for a given earthquake magnitude and distance. However, due to the randomness of earthquake motions it is desirable to produce a statistically significant sample of earthquake motions which can be done by either using simple mathematical wave propagation models or performing a nondeterministic response analysis by means of stochastic modeling of strong ground motions (21).

4. Dynamic Structural Response to Earthquakes

4.1 Earthquake Excitation Function

The formulation of feasible ground motions and hence establishing corresponding excitation functions at the base of a structure is the most difficult part in performing a dynamic structural analysis. The deterministic analysis of the response to a specified earthquake motion will be discussed in this chapter. Even though seismic design requirements can often be described more effectively in probabilistic terms, the nondeterministic response analysis is mentioned only briefly for its limitation to linear systems. Besides, the involved random response analysis does not give any additional insight into the dynamic structural behavior.

The dynamic excitation force in an earthquake problem is defined just like any other form of dynamic loading except that it is applied as support motion. In the most general case the discrete support point has six degrees of freedom in a Cartesian coordinate system, three translations and three rotations. Earthquake ground motions, however, are often given in terms of three translational acceleration components only. Neglecting the rotational components leads to the assumption that the structural soil or rock foundation is rigid which holds only for small dimensions of the structure relative to the vibration wave lengths in the ground. For nonlinear systems the motion components can no longer be treated independently and hence its responses cannot be subject to superposition.

Finally the ground motion introduced at the base of a structure may be influenced by the motion of the structure itself such that it differs considerably from the free-field motion. In case of a flexible structure located on firm ground (rock), the structure can transmit little energy into the soil and the free-field motion is an adequate measure of the foundation displacement. In case of a stiff structure (like a nuclear-reactor power station) located on soft soil, considerable energy will be transferred from the structure to the soil and the base motion may differ drastically from the free-field conditions. This so-called soil-structure interaction effect will also be discussed.

4.2 Modal Analysis for Linear Systems

Consider a nuclear power plant structure shown in Fig. 12 subject to an excitation force vector at its rigid base.

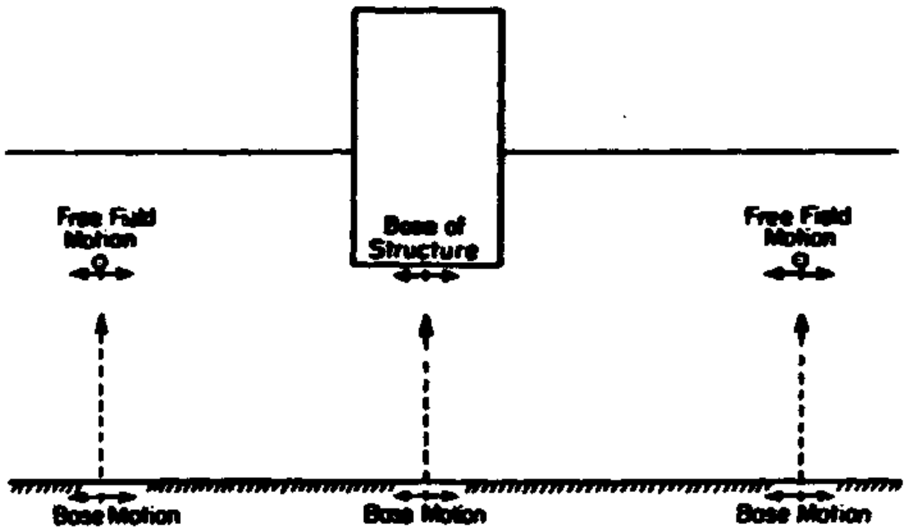

Fig. 12 Schematic nuclear power plant structure subject to an excitation force at its rigid base

A response analysis of this structure can be performed by means of a mathematical model using the finite element method. The equation of motion of a multi-degree-of-freedom system can be formulated based on Hamilton's principle. This states that the variation of the kinetic and potential energy plus the variation of the work done by the nonconservative forces considered during any given time interval must equal zero. From this the equilibrium equation of motion can be written in matrix notation as

$$[M]\{\ddot{v}\} + [C]\{\dot{v}\} + [K]\{v\} = \{p(t)\} \qquad (9)$$

with $$\{p(t)\} = -[M]\{1\}\, \ddot{v}_g(t) \qquad (10)$$

$[M]$=mass matrix, $[C]$=damping matrix, $[K]$=stiffness matrix, $\{v\}$=displacement Vector in system coordinates, and $\ddot{v}_g(t)$= assumed earthquake acceleration at the base.

Equation (9) could be solved directly by numerical integration of the coupled equations. However, for the response analysis of a structure with linear behavior it is much more convenient to transform the equations from system coordinates to modal coordinates. This is especially efficient if the assumed earthquake motion tends to excite strongly only the lowest modes of vibration. Thus only the first few eigenmodes are necessary for a good approximation of the structural response.

The free-vibration mode shapes are independent modal vectors that serve with their orthogonality properties the same purpose as the trigonometric functions in a Fourier series. With the mode-shape matrix $[\Phi]$ the displacement vector in system coordinates $\{v\}$ can be transformed to a generalized displacement vector in modal coordinates $\{Y\}$ such that

$$\{v\} = [\Phi]\{Y\} \qquad (11)$$

Substituting $\{v\}$ and its time derivatives of eq. (11) into eq. (9) and premultiplying this equation by the transpose of the n'th mode-shape vector $\{\phi_n\}^T$ it follows

$$\{\phi_n\}^T[M][\Phi]\{\ddot{Y}\} + \{\phi_n\}^T[C][\Phi]\{\dot{Y}\} + \{\phi_n\}^T[K][\Phi]\{Y\} = \{\phi_n\}^T\{p(t)\} \qquad (12)$$

It can be shown by expansion that all the terms except the n'th vanish because of the mode-shape orthogonality properties that is

$$\{\phi_m\}^T[M]\{\phi_n\} = 0$$
$$\{\phi_m\}^T[K]\{\phi_n\} = 0 \qquad \text{for } m \neq n \qquad (13)$$
$$\{\phi_m\}^T[C]\{\phi_n\} = 0 \text{ (assumed)}$$

In this case eq. (12) may be written

$$M_n \ddot{Y}_n + C_n\dot{Y}_n + K_n Y_n = P_n(t) \qquad (14)$$

or

$$\ddot{Y}_n + 2\xi_n\omega_n\dot{Y}_n + \omega_n^2 Y_n = \frac{P_n(t)}{M_n} \qquad (15)$$

with $M_n = \{\phi_n\}^T[M]\{\phi_n\}$, $C_n = \{\phi_n\}^T[C]\{\phi_n\} = 2\xi_n\omega_n M_n$

$K_n = \{\phi_n\}^T[K]\{\phi_n\} = \omega_n^2 M_n$, $P_n(t) = \{\phi_n\}^T\{p(t)\}$

in which M_n, C_n, and K_n are the generalized properties associated with mode n, Y_n is the amplitude of this modal response, and the generalized force resulting from the earthquake excitation (neglecting the negative sign in eq. (10)) is given by

$$P_n = \{\phi_n\}^T\{p(t)\} = \{\phi_n\}^T[M]\{1\}\ddot{v}_g(t), \qquad (16)$$

with

$$L_n \equiv \{\phi_n\}^T[M]\{1\}$$

the general modal response to equation (15) is given for each mode by the <u>Duhamel integral</u>

$$Y_n(t) = \frac{L_n}{M_n\omega_n} \int_0^t \ddot{v}_g(\tau)\, e^{-\xi_n\omega_n(t-\tau)} \sin \omega_n(t-\tau)d\tau \qquad (17)$$

The solution of equation (17) can be found by any numerical integration procedure which then yields with the transformation equation (11) the displacement vector $\{v(t)\}$ in system coordinates.

The convolution (Duhamel) integral of equation (17) for a specific frequency and damping ratio is also identical to the spectral pseudo-velocity response S_v (ξ, ω). Hence all the maximum modal displacements produced by an earthquake can be obtained, with S_v (ξ, ω) from a response spectrum as shown in Fig. 11, from

$$Y_n \text{ max.} = \frac{L_n}{M_n\omega_n} S_v \qquad (18)$$

Note that multiple-support excitation with different ground motions can be taken care of by correcting the dynamic-response stress vectors with pseudostatic stress vectors (see Ref. (21)).

The modal analysis requires the sometimes cumbersome determination of the eigenvalues of the equation of motion; if damping is not neglected it is a complex eigenvalue analysis. However, with some methods it is possible to extract only those few eigenvalues and their corresponding eigenvectors necessary for a good response approximation. Thus the transformation matrix $[\Phi]$ consists of any number of eigenvectors (modeshapes).

4.3 Direct Integration for Nonlinear Systems

Severe ground motions are expected to cause considerable structural damage. This implies that a structure is subject to significant stiffness changes and hence experiences a nonlinear earthquake response. Nonlinear elements encountered for in an analysis are

nonlinear damping
nonlinear stiffness
excitation force

The earthquake excitation force was represented in the linear system as a convolution integral. For general nonlinear systems, however, the transient excitation force is treated within the numerical integration process as the other nonlinear elements. The nonlinear (viscous) damping is assumed to be a nonlinear function of the response velocity. Virtually any damping level can be represented throughout the foundation (higher damping ratios) and the structure (lower damping ratios). The nonlinear stiffness is due to strain hardening (ductile material behavior) or strain softening (progressive brittle failure). The material is then said to undergo plastic deformation. A nonlinear hysteretic material may equally well be specified.

The direct integration or step-by-step integration can be performed by the so-called linear acceleration method. The idea is to convert the set of simultaneous differential equations of motion (in incremental form and matrix notation)

$$[m]\{\Delta\ddot{v}(t)\} + [c(t)]\{\Delta\dot{v}(t)\} + [k(t)]\{\Delta v(t)\} = \{\Delta p(t)\} \quad (19)$$

to a set of simultaneous algebraic equations with only one unknown vector remaining. A relationship, which holds for a short increment of time, can be established between the displacement, the velocity, and the acceleration by assuming the manner of variation of the acceleration vector with time. Based on linear acceleration the quadratic velocity and the cubic displacement can be formulated at the end of the interval Δt such that

$$\{\Delta\ddot{v}(t)\} = \frac{6}{\Delta t^2}\{\Delta v(t)\} - \frac{6}{\Delta t}\{\dot{v}(t)\} - 3\{\ddot{v}(t)\} \quad (20)$$

$$\{\Delta\dot{v}(t)\} = \frac{3}{\Delta t}\{\Delta v(t)\} - 3\{\dot{v}(t)\} - \frac{\Delta t}{2}\{\ddot{v}(t)\} \quad (21)$$

$$\{\Delta v(t)\} = \{\dot{v}(t)\}\Delta t + \{\ddot{v}(t)\}\frac{\Delta t^2}{2} + \{\Delta\ddot{v}(t)\}\frac{\Delta t^2}{6} \quad (22)$$

Substituting equations (20) and (21) into equation (19) leads to

$$[m]\left(\frac{6}{\Delta t^2}\{\Delta v(t)\} - \frac{6}{\Delta t}\{\dot{v}(t)\} - 3\{\ddot{v}(t)\}\right) + [C(t)]\left(\frac{3}{\Delta t}\{\Delta v(t)\} - 3\{\dot{v}(t)\} - \frac{\Delta t}{2}\{\ddot{v}(t)\}\right) + [k(t)]\{\Delta v(t)\} = \{\Delta p(t)\} \quad (23)$$

Transferring all terms with the known initial conditions to the right-hand side gives the set of simultaneous equations in $\{\Delta v(t)\}$

$$[\tilde{k}(t)]\{\Delta v(t)\} = \{\Delta\tilde{p}(t)\} \quad (24)$$

where

$$[\tilde{k}(t)] = [k(t)] + \frac{6}{\Delta t^2}[m] + \frac{3}{\Delta t}[c(t)] \quad (25)$$

$$\{\Delta\tilde{p}(t)\} = \{\Delta p(t)\} + [m]\left(\frac{6}{\Delta t}\{\dot{v}(t)\} + 3\{v(t)\}\right) + [c(t)]\left(3\{\dot{v}(t)\} + \frac{\Delta t}{2}\{\ddot{v}(t)\}\right) \quad (26)$$

With the solution of the displacement increment vector $\{\Delta v(t)\}$ of eq. (24) the velocity increments $\{\Delta\dot{v}(t)\}$ can be found from eq. (21) and the final displacement and velocity vectors at the end of the increment are given by

$$\{v(t+\Delta t)\} = \{v(t)\} + \{\Delta v(t)\}$$
$$\{\dot{v}(t+\Delta t)\} = \{\dot{v}(t)\} + \{\Delta\dot{v}(t)\} \qquad (26a)$$

Since we have approximated $[c(t)]$ and $[k(t)]$ by initial tangent values an accumulation of errors due to these values is avoided by calculating the acceleration vector at the beginning of each time increment from the total equilibrium of forces at that time by

$$\{\ddot{v}(t+\Delta t)\} = [m]^{-1}(\{p(t+\Delta t)\} - \{f_d(t+\Delta t)\} - \{f_s(t+\Delta t)\}) \qquad (27)$$

in which $\{f_d(t + \Delta t)\}$ and $\{f_s(t + \Delta t)\}$ represent the damping and stiffness force vectors, respectively, evaluated from the velocity and displacement conditions at time $t + \Delta t$ as well as the past history if the material properties are history-dependent.

This integration procedure is stable only for a short enough time increment, with vibration periods at least 5 to 10 times greater than the integration interval that is. The linear acceleration method is only conditionally stable and will blow up if it is applied to modal response components having periods of vibration less than about 1.8 times the integration interval. For complex mathematical models steming from finite element idealizations the shortest period of vibration may be several orders of magnitude less than the periods associated with the significant structural response. In order not to use such very short time increments unconditionally stable linear-acceleration methods have been developed such as the Wilson-θ-method (Ref. (26)). E.L. Wilson's modification is based on the assumption that the acceleration varies linearly over an extended computation interval

$$\tau = \theta \Delta t \quad \text{where } \theta > 1.37 \qquad (28)$$

An acceleration increment vector $\{\hat{\Delta}\ddot{v}(t)\}$ is calculated by the standard linear-acceleration procedure applied to the extended time step τ ; from this the acceleration increment vector $\{\Delta\ddot{v}(t)\}$ for the normal time step Δt is obtained by interpolation as

$$\{\Delta\ddot{v}(t)\} = \frac{1}{\theta}\{\hat{\Delta}\ddot{v}(t)\} \qquad (29)$$

For a value of $\theta = 1$, the procedure reverts to the standard linear-acceleration method, but for $\theta > 1.37$ it becomes unconditionally stable (Ref. (21)).

The performance of the Wilson-θ-method for a dynamic system shows a period elongation and an amplitude decay for $\theta > 1.37$. This means that with an extended computation interval τ higher mode components are truncated and therefore their periods not considered for the determination of the integration time step. This artificial damping out of the mathematically not representative mode components is analogous to the deliberate truncation in the modal analysis or the Fourier transformation.

Note that also linear systems can be treated by the linear-acceleration process; in this case the damping and stiffness properties remain constant so that the numerical analysis is somewhat simpler.

4.4 Frequency Response and Random Analysis

Frequency Response Analysis

It is sometimes convenient to rather perform a linear dynamic analysis in the frequency domain than in the time domain described above. Periodic load analysis involves expressing the load in terms of harmonic components, evaluating the response of the structure to each component, and then superposing the harmonic responses to obtain the total structural response. For nonperiodic loading, however, it is necessary to extend the well known Fourier series concept for periodic loading

$$p(t) = a_0 + \sum_{n=1}^{\infty} a_n \cos \frac{2\pi n}{T_p} t + \sum_{n=1}^{\infty} b_n \sin \frac{2\pi n}{T_p} t \qquad (30),$$

in which T_p represents the period of the load function, and the coefficients can be evaluated from

$$a_0 = \frac{1}{T_p} \int_0^{T_p} p(t)\, dt$$

$$a_n = \frac{2}{T_p} \int_0^{T_p} p(t) \cos \frac{2\pi n}{T_p} t \qquad (31)$$

$$b_n = \frac{2}{T_p} \int_0^{T_p} p(t) \sin \frac{2\pi n}{T_p} t \ ,$$

with the substitution of the trigonometric functions by the corresponding exponential terms, to the following Fourier integral

$$p(t) = \frac{1}{2\pi} \int_{\bar{\omega}=-\infty}^{\infty} c(\bar{\omega}) e^{i\bar{\omega}t}\, d\bar{\omega} \qquad (32)$$

in which the harmonic amplitude function is given by

$$c(\bar{\omega}) = \int_{t=-\infty}^{\infty} p(t)e^{-i\bar{\omega}t}dt \qquad (33)$$

and the loading period is extended to infinity ($T_p \to \infty$). The frequency increment therefore becomes an infinitesimal ($\Delta\bar{\omega} \to d\bar{\omega}$) and the discrete frequencies $\bar{\omega}_n$ of the series of harmonic loads become a continuous function $\bar{\omega}$. The steady-state solution of a single degree of freedom system subject to a unit complex forcing function $e^{i\bar{\omega}t}$ can be obtained from the equation of motion

$$m\ddot{v}(t) + c\dot{v}(t) + kv(t) = e^{i\bar{\omega}t} \qquad (34)$$

as

$$v(t) = H(\bar{\omega})e^{i\bar{\omega}t} \qquad (35)$$

in which the complex frequency response function $H(\bar{\omega})$ takes the form

$$H(\bar{\omega}) = \frac{1}{-\bar{\omega}^2 m + i\bar{\omega}c + k} \qquad (36)$$

In analogy the response through the frequency domain can be obtained by multiplying the arbitrary loading p(t) (eq. (32)), represented in equation (32) by an infinite sum of harmonic components, with the complex frequency response function $H(\bar{w})$ (eq. (36)) integrated over the entire frequency range.

$$v(t) = \frac{1}{2\pi} \int_{\bar{\omega}=-\infty}^{\infty} H(\bar{\omega}) c(\bar{\omega}) e^{i\bar{\omega}t} d\bar{\omega} \qquad (37)$$

The evaluation of the integrals is usually performed by a numerical analysis procedure in which discrete Fourier transform expressions are derived and evaluated by the so-called Fast Fourier Transform technique (Ref. (21)). Note that the frequency response analysis is restricted to linear systems due to the superposition principle applied.

Random Response Analysis

Strictly speaking, future earthquakes can only be described in terms of probability. In this case any previously described deterministic analysis can no longer be applied. The structural parameters are usually taken as deterministic quantities and hence probabilities are introduced only in the treatment of ground motions. In this case the response analysis process is random for it is impossible to predict the final earthquake response from the incomplete initial state. Usually one is interested primarily in the mean and the standard deviation of the extreme values of response. Considering the response of a simple oscillator due to a vertical compression wave of an earthquake, the response is not only random with respect to time but is also random with respect to the vertical space coordinate component, random to two independent variables that is. Obviously the larger the number of independent variables involved in a random process the more difficult it is to characterize the process.

Since the random process is usually performed in the frequency domain (frequency response analysis), again the random response analysis is restricted to linear systems (Ref. (21) and Ref. (24)). Considering nonlinear behavior of a system as being most important, satisfying probability solutions have to be sought in an even more involved nonlinear nondeterministic analysis (Ref. (27)).

5. Foundation Medium Behavior

5.1 Stability of the Soil Medium

For any earthquake response analysis taking the underlying soil medium into account it is important to deal with soil properties that sustain cyclic stresses. The medium for a response analysis will therefore be cohesive soils, which do not show any breakage of particels or any liquefaction, or rocks. Liquefaction is caused in saturated cohesionless soils during an earthquake by the build-up of excess hydrostatic pressures due to the application of cyclic shear stresses induced by the ground motions (Ref. (28)).

5.2 Soil-Structure Interaction

In all the cases of earthquake excitation discussed above, it has been assumed that the earthquake motions were introduced as specified quantities at the structure support points. However, an important consideration in evaluating the effects of earthquakes on the safety of nuclear power plants is that of assessing the relationship between the characteristics of the earthquake ground motions, the local soil and geologic conditions at the site, and the response of the structure to the ground motions. This leads after (29) to essentially two aspects of the problem:

1. The local soil and geologic conditions may affect the characteristics of the ground motions developed at the site-both in terms of the peak acceleration and the frequency content of the motions as expressed by the response spectrum, and it is necessary to evaluate the possible extent of these effects in order to establish meaningful design criteria.

2. The structures will interact with the soils below and adjacent to them during the earthquake shaking and it is necessary to evaluate the effects of this interaction on the motions developed in the structures.

For the design of nuclear power plants it is customary to specify the earthquake ground motions by means of a maximum ground acceleration and a specified shape of the response spectrum at some control point. Usually the control point is near or at the ground surface and the spectrum shape is a broad-band spectrum with a reasonably uniform spectral acceleration over frequencies ranging from about 2 to 9 Hz. A time-history of motions is then developed which produces the required spectral shape. This procedure is used for design in terms of the Safe Shutdown Earthquake and the Operating Basis Earthquake.

Earthquake motions result from a complex pattern of waves whose characteristics depend on many significant factors including the source mechanism, the geologic conditions along the travel path between the source and the site, the distance between the source and the site, as well as the local soil conditions. However, for engineering purposes at the present time the motions in the soil deposit are often evaluated by analyses based on the assumption that the motions are due primarily to vertically propagating waves.

Soil-structure interaction is influenced by the response of the soil deposit to some depth. This response depends on the design motion criteria considered. According to (29) is in some cases excessive conservatism introduced into the design through the following inappropriate specifications of the design ground motion (Ref. (30) and (31)):

1. The assumption that the maximum acceleration in a soil deposit does not change with depth - at least to the depth of embedment of a structure. There is in fact theoretical evidence and some field data showing that this is not the case and that maximum accelerations can decrease significantly with depth in soil deposits.

2. The specification of a design maximum acceleration which could not possibly develop in certain types of deposits. There is necessarily a limit to the maximum acceleration which any soil can transmit and this should be recognized in establishing design criteria.

3. The specification of a broad-band spectrum shape at some depth in a soil deposit. In fact there are sound reasons why earthquake motions at any depth in a natural deposit would exhibit a natural suppression of some frequency component, thereby precluding the possibility of a broad-band spectrum for motions below the ground surface.

4. The specifications of high frequency components at points in a soil deposit where they would not naturally exist - for example, near the surface of relatively deep or soft soil formations.

By appropriate considerations implied above it should be possible to establish response patterns which are reasonable at all points within any soil deposit, thereby providing a meaningful basis for evaluating soil-structure interaction effects. In the light of these considerations a procedure for establishing the earthquake motions to be used for design is suggested by the Ad Hoc Group on Soil-Structure Interaction of the ASCE (29) as follows:

1. By means of the combined contributions of seismological data, and available ground motion records, especially those recorded on sites with similar characteristics and ground response analyses incorporating reasonable soil characteristics (strain-dependent properties) and a suite of realistic rock outcrop motions, establish a minimum acceleration level and a required spectral shape for the ground surface motions.

2. Using the same approaches as in item 1, establish a minimum acceleration level which should be developed in the free field soil profile within the depth of interest for the structures and a minimum acceptable acceleration response spectrum to be considered in the free field at the level of the base of any structure.

3. Establish a reasonable set of soil characteristics which are consistent with the known properties of soils at the site and which are compatible with the specified acceleration distribution.

4. Based on the above, evaluate the response of the soil-structure system for the selected earthquake motions.

5. Determine the effect of variations in soil characteristics, within the likely ranges, on the computed structural response.

6. If different possible conditions are considered or if the spectra for the computed motions in the structure show marked deficiencies in certain frequency components, draw a broadened envelope spectrum to determine the design motions.

Soil Structure Interaction Analysis

The problem accounting for soil-structure interaction may now be defined as follows: given the earthquake ground motions that would occur on the surface of the ground if the structure were not present (the control or design motions), find the dynamic response of the structure.

The basic requirements for a good analytical procedure may be summarized as follows:

1. The analysis should consider the variation of soil characteristics with depth.

2. The analysis should consider the non-linear and energy-absorbing characteristics of the soils.

3. For embedded structures, the analysis should consider the variation of ground motions with depth.

4. The analysis should be capable of taking into account the threedimensional nature of the problem.

5. The analysis should be capable of considering the effects of adjacent structures on each other.

On the previously stated assumption that the motions considered in the near-surface soils are due to vertical propagation of body waves from below there are several ways in which the ground motion distribution and the response of the structure can be found. One way is to deconvolve the control motion to some depth in the soil profile such as soil-rock interface. One dimensional amplification theory (32) can be used for this purpose. Then the motion computed at this depth can be used as input to a three-dimensional mathematical model of the overlying soil and structure as shown in Fig. 13 from Ref. (32).

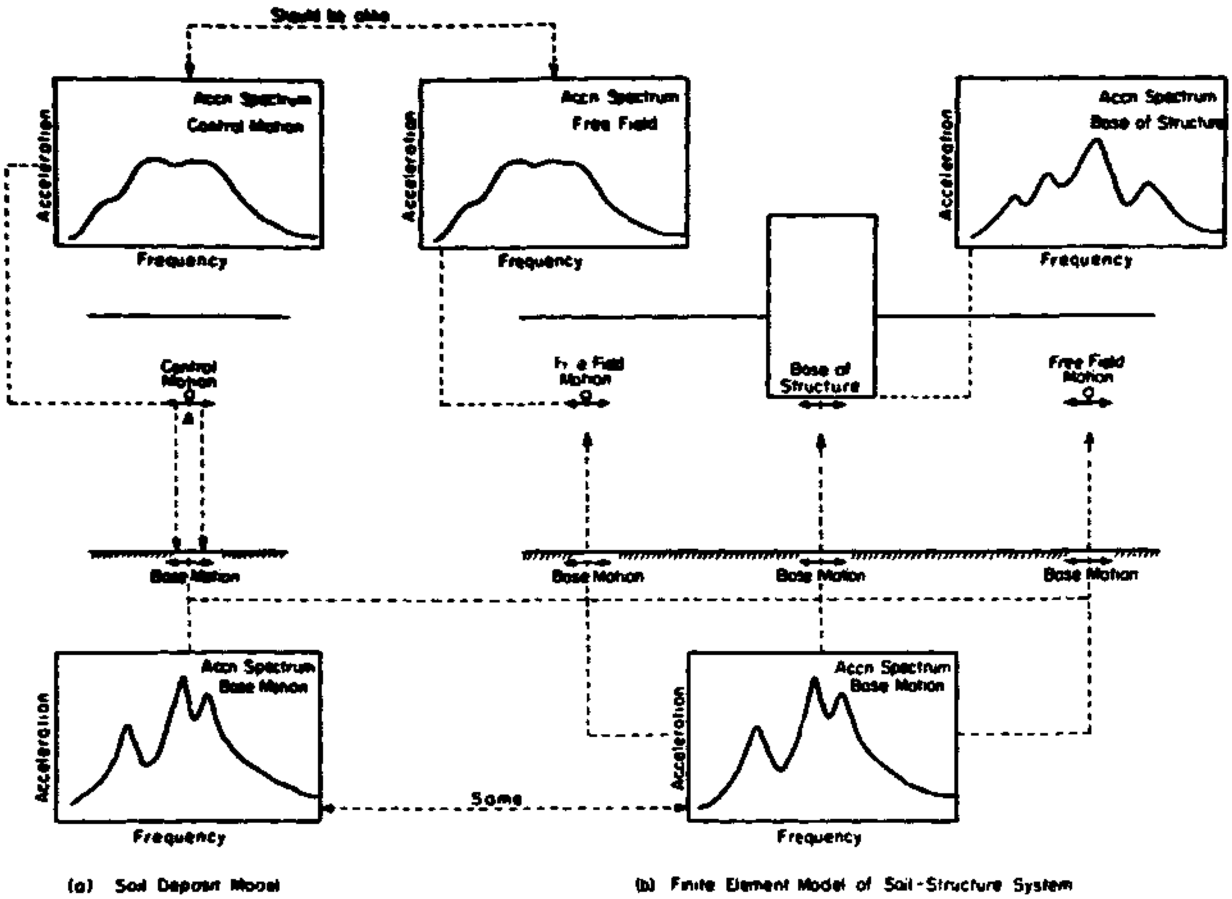

Fig. 13 Schematic representation of soil-structure interaction analysis using finite element model, from Ref. (33).

For embedded structures, as is usually the case for nuclear power plants, consideration of the variation of ground motions with depth is essential. At present time, analyses includ'ng these effects have only been developed using finite element methods. One of the most sophisticated new computer programs of this sort is the FLUSH code (33). Features and methods of analysis of FLUSH, A Computer Program for Approximate 3-D Analysis of Soil-Structure Interaction Problems, developed by Lysmer, Udaka, Tsai and Seed of UC Berkeley, will be displayed subsequently for they are representative for the very present state of the art.

Two-Dimensional Finite Element Analyses

Based on the deconvolution procedure described above (feature included in the FLUSH program) or on the transfer function approach, relating the motions at desired points in the soil or structure to the control motion applied at a point in the soil well away from the surface, the analysis is performed iteratively to allow for the strain dependent nature of the nonlinear soil characteristics. In each iteration the analysis is linear but the soil properties are adjusted from iteration to iteration until the computed strains are compatible with the soil properties used. The approach used thus allows for different soil properties for every element taking the variation of soil characteristics with depth into account, while the iteration procedure permits for the nonlinear stress-strain and damping characteristics of the soil. A two stage approach has been used for the analysis with the original LUSH program: (a) a two-dimensional analysis of the soil-structure system to determine the base motions and (b) a three-dimensional analysis of the structure to determine its response to this base motions. This two-dimensional representation of the soil-structure system has provided good evaluations of the response at the base of the structure only for this approach fails to satisfy the requirements 4 and 5 listed under the preceding section.

Three-Dimensional Effects

To take the three-dimensional nature of the soil-structure system into account, finite element analysis procedures for axisymmetric structures (Fig. 14) have been developed by several investigators (see Ref. (33)).

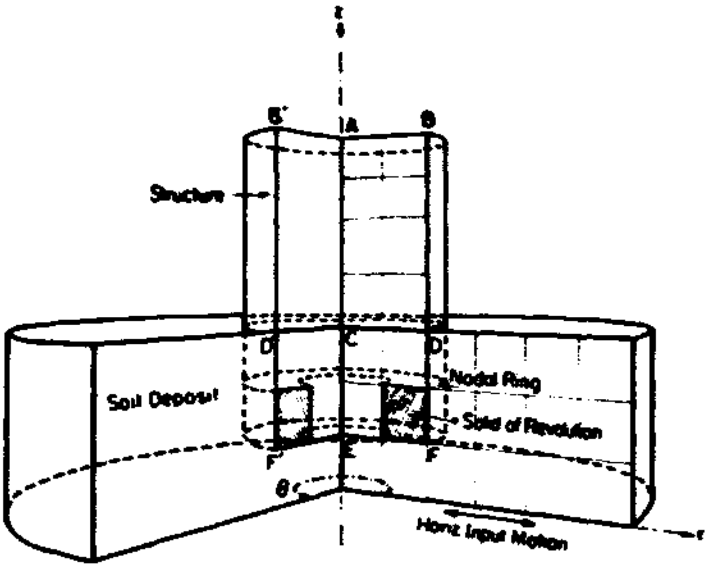

Fig. 14 Schematic view of an axi-
symmetric model, from Ref.(33)

This analytical model may be provided with transmitting boundaries which absorb any wave effects emanating from the structure and thus simulate the effects of an extensive soil deposit. However, any combined system of two or more nuclear power plant structures is no longer axisymmetric. An alternative approach for including three-dimensional effects is the method suggested by Hwang (Appendix C, Ref. (33)) involving the use of viscous boundaries along the planar surfaces of a slice of soil on which one or more structures are located. While energy radiating along the axis of the slice will be absorbed by material damping and transmitting boundaries (additionally introduced at the ends to drastically reduce the number of finite elements), the energy radiating in directions normal to the axis of the slice will be absorbed by these viscous boundaries (simulating three-dimensional effects), (see Fig. 15).

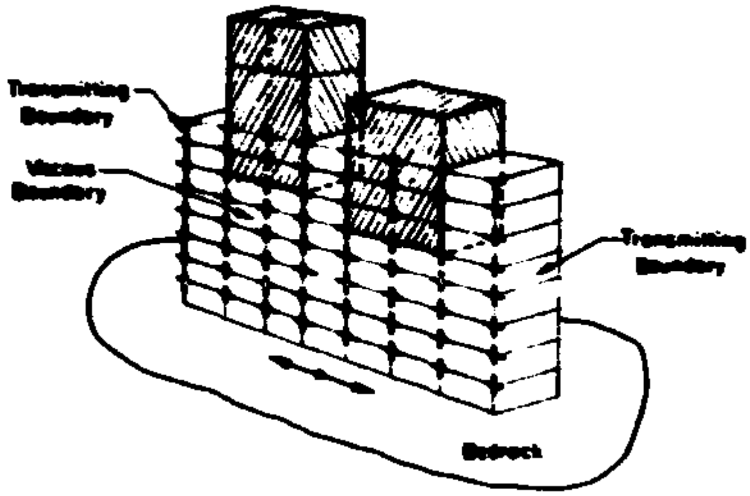

Fig. 15 Schematic view of a simplified 3-dimensional model with multiple structures, from Ref. (33).

The necessity for taking in a structural response analysis the interaction between adjacent structures, the so-called <u>building-building interaction</u>, into account, is shown in Fig. 16. It may be seen from the three curves that the presence of the adjacent structures increase the maximum response of the containment building by about 60 percent using a three-dimensional analysis shown in Fig. 15, a substantial effect, but still considerably less than that indicated by a two-dimensional analysis.

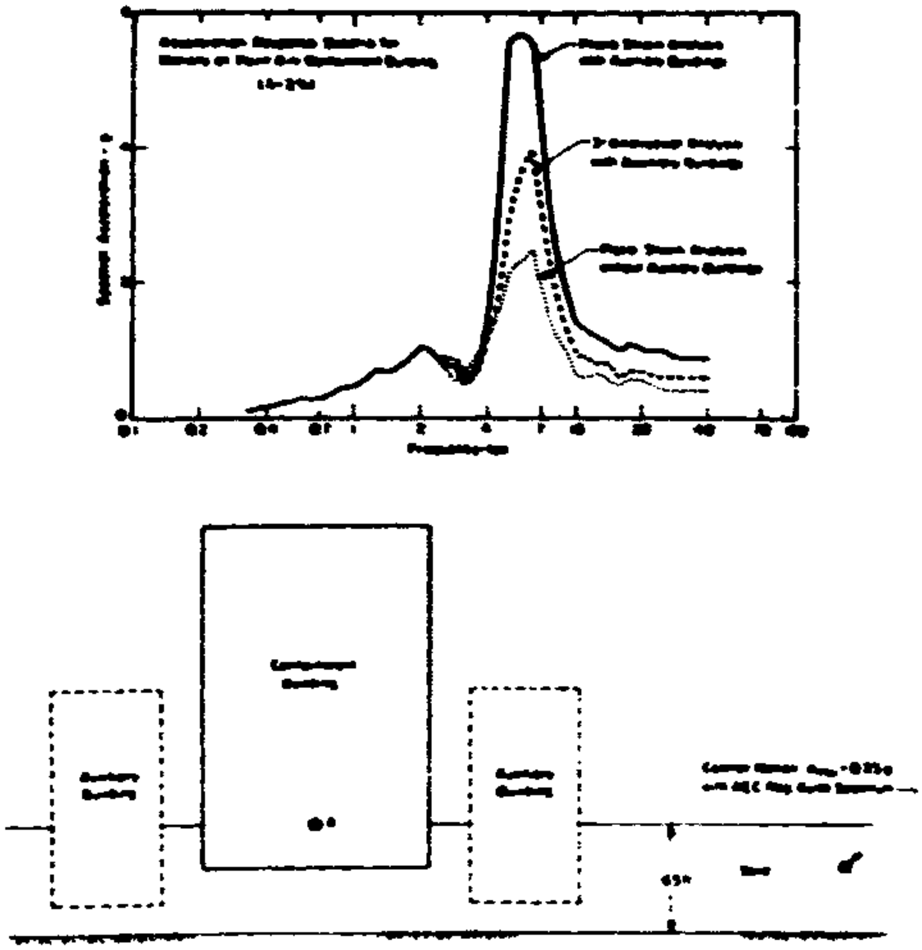

Fig. 16 Analyses of soil-structure system with multiple structures, from Ref. (33).

Computational Method Used in FLUSH

The mathematical model described above appears to satisfy the major requirements for high quality analysis of a large family of three-dimensional seismic interaction problems. The analytical procedure is essentially two-dimensional. The equation of motion for a finite element representation of the system can be written according to the FLUSH manual and in analogy with equation (9) as

$$[M]\{\ddot{v}\} + [K_c]\{v\} = -\{m\}\ddot{v}_g(t) - \{V\} + \{F\} - \{T\} \qquad (38)$$

where $\{v\}$ are the displacements of the nodal points relative to the rigid base, $[M]$ is the plain strain mass matrix and $[K_c]$ the plain strain stiffness matrix (material damping included in form of complex moduli) of the slice unit thickness, and $\{m\}$ is a vector related to $[M]$ in the direction of the rigid base acceleration $\ddot{v}_g(t)$.

The force vector $\{V\}$ originates from the viscous boundary acting in the plane of the slice sides given by

$$\{V\} = \frac{1}{d}\,[C_f](\{\dot{v}\} - \{\dot{v}_f\}) \tag{39}$$

with d being the slice thickness, $[C_f]$ a diagonal matrix representing the free field properties, and $\{\dot{v}_f\}$ the known free field velocities.

The force vector $\{F\}$ represents the forces acting on the vertical planes in the free field at the ends of the slice not involving any horizontal transmission of wave energy. It is

$$\{F\} = [G]\{v_f\} \tag{40}$$

where $[G]$ is a simple frequency independent stiffness matrix formed from the complex moduli in the free field.

The force vector related to the energy transmission is

$$\{T\} = ([R] + [L])(\{v\} - \{v_f\}) \tag{41}$$

where $[R]$ and $[L]$ are frequency dependent boundary stiffness matrices which represent the exact dynamic effect of the semi-infinite viscoelastic soil system cut off at both ends of the model.

The equation of motion can be solved by the complex frequency response method shown in chapter 4.4. Assuming that the loading is periodic the infinite time integral can be replaced by a finite sum of harmonics, i.e. a truncated Fourier series

$$\ddot{v}_g(t) = \text{Re} \sum_{n=0}^{N/2} \ddot{v}_{gn} \cdot e^{i\omega_n t} \tag{42}$$

where N is the number of digitized points in the input motion. Writing the response also as a Fourier series expansion

$$\{v\} = \text{Re} \sum_{n=0}^{N/2} \{v_n\}\, e^{i\omega_n t} \tag{43}$$

and

$$\{v_f\} = \text{Re} \sum_{n=0}^{N/2} \{v_{fn}\}\, e^{i\omega_n t} \tag{44}$$

The amplitudes $\ddot{v}_{gn}$ and $\{v_{fn}\}$ can be found by the Fast Fourier Transform algorithm (Ref. (21)). Substitution of equations (39) to (41) and corresponding terms of equations (42) and (44) into equation (38) yields

$$([K_c] + [R_n] + [L_n] + \frac{i\omega_n}{1}[C_f] - \omega_n^2[M])\{v_n\}$$

$$= -\{m\}\ddot{v}_{gn} + ([G] + [R_n] + [L_n] + \frac{i\omega_n}{1}[C_f])\{v_{fn}\} \tag{45}$$

which is a set of linear equations which determines the displacement amplitudes $\{v_n\}$ at each frequency ω_n, n=0, 1, ..., N/2. The equations can be solved by Gaussian elimination and the displacements in the time domain {v} follow from equation (43) by the inverse Fast Fourier Transform. Note that the non-linear effects in soil due to large shear deformations is taken care of by an equivalent linear method, estimating in an iterative procedure the actual shear modulus and damping value for each soil finite element (Ref. (33)). Also linear bending elements are provided by the code for a better modeling of basement walls and structural frames.

Separation of Base Mat from Soil

Consider a heavy reactor shield building being placed on a separate base mat. This typically heavy structure, usually stemming from an airplane crash design, has drawn the attention of structural engineers for the soil-structure interaction response analysis requires to consider a separation of the base mat from the soil due to the large overturning moments produced by severe earthquake excitations. Since tension that would occur in part of the area of contact is incompatible with the constitutive law of soils, the base mat will become partially separated from the underlying soil. Wolf (34) has developed a numerical method, based on the standard linear elastic half-space theory used to represent the soil-structure interaction model, to determine this partial lifting-off.

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Appendix 1

The Modified Mercalli Intensity Scale

Mercalli's (1902) improved intensity scale served as a basis for the scale advanced by Wood and Neumann (1931), known as the modified Mercalli scale (MM-Scale). The modified version is described below with some improvements by Richter (1958). The following remarks are taken almost verbatim from Elementary Seismology. Charles F. Richter (W.H. Freeman and Company, San Francisco, copyright 1958).

To eliminate many verbal repetitions in the original scale, the following convention has been adopted. Each effect is named at the level of intensity at which it first appears frequently and characteristically. Each effect may be found less strongly, or in fewer instances, at the next lower grade of intensity; more strongly or more often at the next higher grade. A few effects are named at two successive levels to indicate a more gradual increase.

Masonry A, B, C, D. To avoid ambiguity of language, the quality of masonry, brick or otherwise, is specified by the following lettering (which has no connection with the conventional Class A, B, C construction).

Masonry A. Good workmanship, mortar, and design; reinforced, especially laterally, and bound together by using steel, concrete, etc.; designed to resist lateral forces.

Masory B. Good workmanship and mortar; reinforced, but not designed in detail to resist lateral forces.

Mansory C. Ordinary workmanship and mortar; no extreme weakness like failing to tie in at corners, but neither reinforced nor designed against horizontal forces.

Masonry D. Weak materials such as adobe; poor mortar; low standards of workmanship: weak horizontally.

Modified Mercalli Intensity Scale of 1931 (Abridged and Rewritten by C.F. Richter).

1. Not felt. Marginal and long-period of large earthquakes.

2. Felt by persons at rest, on upper floors, or favorably placed.

3. Felt indoors. Hanging objects swing. Vibration like passing of light trucks. Duration estimated. May not be recognized as an earthquake.

4. Hanging objects swing. Vibration like passing of heavy trucks; or sensation of a jolt like a heavy ball striking the walls. Standing motor cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. In the uppper range of 4, wooden walls and frames crack.

5. Felt outdoors; direction estimated. Sleepers wakened. Liquids disturbed, some spilled. Small unstable objects displaced or upset. Doors swing, close, open. Shutters, pictures move. Pendulum clocks stop, start, change rate.

6. Felt by all. Many frightened and run outdoors. Persons walk unsteadily. Windows, dishes, glassware broken. Knickknacks, books, and so on, off shelves. Pictures off walls. Furniture moved overturned. Weak plaster and masonry D cracked. Small bells ring (church, school). Trees, bushes shaken visibly, or heard to rustle.

7. Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to masonry D including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices, unbraced parapets, and architectural ornaments. Some cracks in masonry C. Waves on ponds; water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged.

8. Steering of motor cars affected. Damage to masonry C; partial collapse. Some damage to masonry B; none to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; loose panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on step slopes.

9. General panic. Masonry D destroyed; masonry C heavily damaged, sometimes with complete collapse; masonry B seriously damaged. General damage to foundations. Frame structures, if not bolted, shifted off foundations. Frames racked. Conspicuous cracks in ground. In alluviated areas sand and mud ejected, earthquake fountains, sand craters.

10. Most masonry and frame structures destroyed with their foundations. Some well built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Large landslides. Water thrown on banks of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beaches and flat land. Rails bent slightly.

11. Rails bent greatly. Underground pipelines completely out of service.

12. Damage nearly total. Large rock masses displaced. Lines of sight and level distorted. Objects thrown into the air.