A Numerical Study of Seismic Isolation

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ABSTRACT

The seismic isolation of nuclear structures is an attractive concept and has stimulated a great deal of interest both in practical engineering and research and development. The arguments in its favour suggest a considerable advantage and justify a close examination of the potential and practicability of such systems. This paper presents a summary of isolation systems that have been proposed over the years in the published literature. Three of the most feasible of these systems are selected on the basis of practical considerations of implementation, reliability and other constraints within the nuclear industry. The selected systems are analysed to assess their potential in isolating a simple structure whose masses and springs are chosen to represent the predominant modes of vibration of a reactor building. The results demonstrate some of the dangers of extrapolating results from very simple models. They also show the necessity for considering other sources of dynamic excitation as isolating for one source may make the structure more vulnerable to other sources.

1. Introduction

The design of structures to sustain forces induced by earthquakes is a subject of major concern and a great deal of technical effort has been dedicated to ensuring the integrity of important structures under seismic loading. Two general approaches have been adopted by civil engineers and designers to achieve this aim. The first method employs the conventional procedure of stiffening the structure to cope with the dynamic induced forces. The second method is directed at reducing the forces by special structural elements which may include energy absorbers and isolators that limit the magnitude and frequency of the forces experienced by the structures. This paper is concerned with the second approach with special emphasis for nuclear power plants.

Civil engineers have for many decades considered the isolation or partial isolation of structures from unusual environments. Typical applications have been concerned with vibrations induced by railways and heavy machines. However, it was not until the last decade or so that vibration isolation found practical application in the field of earthquake engineering. Seismic isolation is an attractive concept and it has stimulated a great deal of interest both in practical engineering and research and development. The major arguments that have been put forward for encouraging the consideration of isolation as a practical solution to the problem of aseismic design of structures include:

(a) Isolators reduce the forces induced in the structures and equipment

(b) The use of certain isolators reduces the importance of the uncertainties generally associated with the magnitude and frequency content of the design ground motion.

(c) If the structural forces are limited by the use of the isolators then a standard structural design can be used independent of the seismic environment.

(d) Isolation permits construction of standardised conventional structures in regions of high seismicity where this could not have been previously considered.

(e) The use of isolation devices could have economic benefits in the design of structures in areas of high seismicity.

These arguments suggest a considerable advantage in using seismic isolation devices and justify a closer examination of the potential and practicability of such systems.

The development and implementation of seismic isolation devices is presently an area of active scientific research and development. A number of schemes have been presented in technical publications and some of these have been tested and applied to conventional structures like bridges, schools and high-rise buildings in Europe, Japan and New Zealand. Recent innovative work by a French consortium has broken new ground in suggesting and building two nuclear power plants on isolation bearings.

In this paper isolation devices that have been proposed in the published literature are identified and summarised. Of these, a few systems are selected on the basis of practical consideration, as feasible for use in the nuclear industry. These systems are assessed by analysis of a simple structure comprising lumped masses and springs. The numerical values assigned to the masses and springs are chosen to obtain predominant frequencies of vibration similar to those of a typical nuclear reactor building. The results demonstrate the effectiveness of the selected isolation devices and highlights some of the problems that must be considered when designing such systems.

2. Isolation Systems

A complete review of seismic isolation was performed by Kumar and Maini [1]. From this study five categories of isolation systems were identified:

(1) period lengthening devices

(2) energy absorbing devices

(3) decoupling devices

(4) screening devices

(5) special structural concepts

Period Lengthening Devices:

The dynamic response of structures and their foundations are crucially dependent on the ratio of their fundamental frequencies to the frequency of excitation. The maximum response (resonances) occurs when this ratio is unity. One method of achieving lower structural response (i.e. partial seismic isolation) is to reduce the fundamental frequency of the structures (by employing physical devices like soft springs) so it falls outside the range of frequencies characteristic of strong ground motion. Devices that isolate structures in this way are categorised as period lengthening devices. Such devices include:

(i) Soft springs in the form of helical steel springs, laminates of natural rubber, vulcanised India rubber and neoprene with steel, and air springs.

(ii) Pendulum devices

(iii) Combinations of rubber springs with friction plates, hysteretic dampers and mechanical fuses.

Two practical applications of this system are (i) a three storey reinforced concrete school building in Skopje, Yugoslavia (rubber isolators) (Petrovski et al [2]), and (ii) the Koeberg nuclear power plant complex (rubber isolators in conjunction with friction pads, see fig.1) (Jolivet and Richli [3]).

Energy Absorbing Devices:

The principial purpose of these devices is to reduce structural response by absorbing energy, thereby limiting a build up of vibrations and forces within the structure. Typical practical systems include:

(i) hysteretic dampers

(ii) viscous dampers

(iii) tuned mass dampers

(iv) combination of hysteretic dampers and rubber bearings

Robinson and Greenbank [4] reported that a lead extrusion energy absorber has been used for bridges in New Zealand (see fig.2).

Decoupling Devices:

One obvious way to achieve isolation is to directly limit the seismic forces that can be transmitted to the structure by decoupling the structure from its foundation. Possible schemes are to float the structure in water, use friction plates, or use rollers and rockers. The French system (Jolivet and Richli [3]) referred to previously, under period lengthening devices, can also be classified as a decoupling device for high horizontal accelerations.

Screening Devices:

The principle of isolation by screening is to place a barrier between the seismic source and the structure to be protected. Trenches and piles have been suggested in the literature.

Special Structure Designs:

This aspect of the aseismic design of structures does not consider the explicit use of special isolating devices. It concerns design features incorporated into the construction to reduce structural forces. These include:

(i) soft first storey

(ii) double basemat concept

(iii) rod mechanism flexible column system

3. Practical Consideration for Isolation of Nuclear Structures

Any seismic isolation device must satisfy certain fundamental requirements if it is to be considered practical. In the nuclear industry some of these requirements are particularly severe because of the extremely stringent safety standards that must be satisfied.

It is expected that before any seismic isolation is adopted, stringent numerical and laboratory tests or even field tests would have been performed to verify the system. Numerical calculations must take into account any eccentricities and deviations from specifications that may occur during the construction. In the event of minor deviations from expected conditions, it is important that the efficiency and behavioural characteristics of the isolation device are not impaired significantly.

Seismic isolation systems must be guaranteed to function correctly during 30 seconds or so of an earthquake which may occur anytime during the lifespan of the structure. This places a great deal of emphasis on reliability which becomes doubly important when it is recognised that adequate performance of the isolation system is crucial to the integrity of the isolated structure. For this reason, it may be preferable to choose a simple mechanical system rather than a system which relies on a triggering device or external power source. In addition to this, it is important that regular monitoring and maintenance be performed on the system. Routine maintenance should be made as simple as possible if the isolation system requires it during the life of the structure.

One of the more important design requirements for a seismic isolation system is that it must be effective for main earthquake shocks as well as aftershocks assuming no repairs are possible after the main shock. Some devices which rely on plastic deformations for absorption of energy may not meet this requirement. Also some devices that permit permanent off-set displacements after an earthquake (for example, sliding devices), must be designed with this in mind. A further requirement is that after a major earthquake and aftershocks, it must be possible to restore the full effectiveness of the isolation system by replacing devices where and if necessary.

Cost-effectiveness is always an overriding consideration in any project. For a seismic isolation system to be feasible, it must be cheaper overall to construct, monitor and maintain during the life of the structure than a conventional design constructed to withstand the seismic loads. Cost benefits will increase substantially if the isolation system can free the structure design from dependence on the site configuration and the seismic conditions. This will permit the use of a standard structure that would not incur any major modification costs for compatibility with site specific criteria.

A further consideration is related to the question of licenceability. In the nuclear industry the utmost priority is safety, and the licencing bodies are understandably very conservative. Consequently, a great deal of experimental and numerical work must be performed with the isolation system to convince authorities of its behaviour and reliability. All uncertainties must be considered and eliminated wherever possible. It is therefore in the interest of the nuclear supplier to use as simple as system as possible. This will minimise the uncertainties and improve the changes of its acceptance by the licencing authorities.

From the above considerations, the most practical of the identified isolation systems appears to be soft springs or rubber bearings. In practice, this system may be combined with other devices like friction plates or dampers. In this paper, three systems are selected for further assessment: rubber or soft springs; rubber combined with friction elements; rubber combined with hysteretic dampers.

4. Numerical Study of Some Practical Systems

The three practical systems identified above (linear springs, springs with hysteretic dampers and springs with friction elements) are analysed using a simple model of a structure as shown in fig.3. The masses and the springs of this model are selected to be representative of a nuclear reactor building. Two artificial earthquakes generated to match this USNRC horizontal and vertical spectra are applied simultaneously to the model of the structure and the isolation devices.

Linear Springs:

When soft springs or rubber bearings are used as isolators, the structural response is predominantly a rigid body mode of vibration. The frequency of this vibration is referred to as the isolation frequency. The model shown in fig.3 was analysed for a range of isolation frequencies. The resulting peak acceleration and displacement responses of mass 2 are plotted in fig.4 as a function of the isolation frequency. This example demonstrates one of the aspects of seismic isolation that require careful consideration and design. The purpose of this isolation system is to reduce the acceleration response of the structure by shifting the fundamental vibration to a frequency below the predominant earthquake frequencies. In some practical seismic designs, the isolation frequency is selected to be 1Hz or less (Jolivet and Richli [3]). As can be seen from fig.4, at these frequencies the displacement relative to the ground increases quite significantly. Thus the structure and its connections must be designed to accommodate these large displacements. In addition to this, reducing the first mode of vibration to be less than 1Hz may make the structure more vulnerable to wind loading. It is important therefore to consider other hazards which may not be considered important for conventional designs.

Springs and Hysteretic Dampers:

The same structural model used in fig.3 was isolated with a model of springs and hysteretic dampers as suggested by Skinner et al [5].

Analyses were performed for different values of stiffness k_1, k_2, k_3 and Q - the damper yield force (see fig.5). For large values of k_1 (the damper stiffness for small amplitudes) the system is not very effective compared to the spring or rubber on its own ($k_2 = k_1 = 0$). If k_1 is selected such that the isolation frequency for small amplitudes is 5Hz, the response of mass 2 is about 3 times larger than if the rubber isolation alone (tuned to 1Hz) was considered. It is also worth noting that results obtained with different values of Q indicate that if the yield of dampers with large k_1 values is exceeded then it is possible to get higher responses than when yield is not exceeded (i.e. the damper gives adverse rather than beneficial effects1. This is due to the sudden large change in stiffness from $(k_1+k_2+k_3)$ to (k_2+k_3). This in itself is a source of dynamic excitation. For small values of k_1 (comparable to k_3) this sudden change is not large and the system becomes more effective.

Springs with Friction Elements:

The model shown in fig.6 was used to represent a spring and friction isolation system typical of that described by Jolivet and Richli [3]. The linear spring was chosen to have an isolation frequency of 1Hz and the coefficient of friction was taken as 0.2. Fig.7 compares the acceleration response of mass 2 with and without the friction elements. The friction elements clearly increase the effective isolation. It should be noted that the peak accelerations exceed the value of 0.2g which would be predicted by a simple 1-DoF resonator. This shows the danger of extrapolating from a very simple model. The friction element is used to limit the horizontal force that can be transmitted to the structure. It will give little or no isolation to the structure for a mode of vibration which is predominantly a rocking of the foundation.

Such a mode of vibration can be excited by a horizontally travelling SV wave. Evidence of such waves playing an important role in damaging structures is discussed by Takada et al [4]. Consider the example of a typical model of a PWR building as shown in fig.8. The reactor building is constructed on a flexible upper raft which is isolated from the lower raft by springs and friction plates as suggested by Jolivet and Richli [3]. This model which is qualitatively similar to ref.[3], was analysed for simultaneous horizontal and vertical seismic excitations. Firstly, the motions were input as vertically propagating waves and then the same motions were phased along the base of the model to simulate an effective travelling velocity of 1000m/sec. The results obtained show that for the travelling waves the horizontal response at the top of the structure was about 3 times larger than when the waves were assumed to be vertically propagating. These results contradict the conclusions reached by Richli et al [5] but are consistent with general trends observed by Wolf and Obernhuber [6].

This last example highlights the need to consider all potentially hazardous dynamic modes of excitation. Isolating a structure for one particular mode may very well result in a vulnerable structure for other modes.

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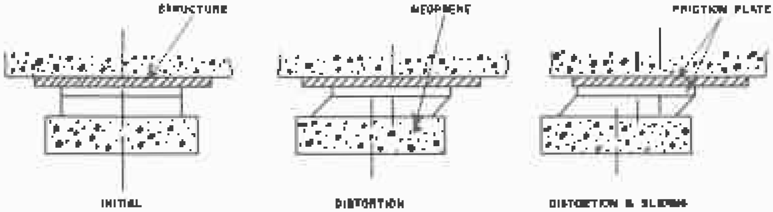

Figure 1 Rubber and friction plates isolation systems (after Jolivet and Richli [3])

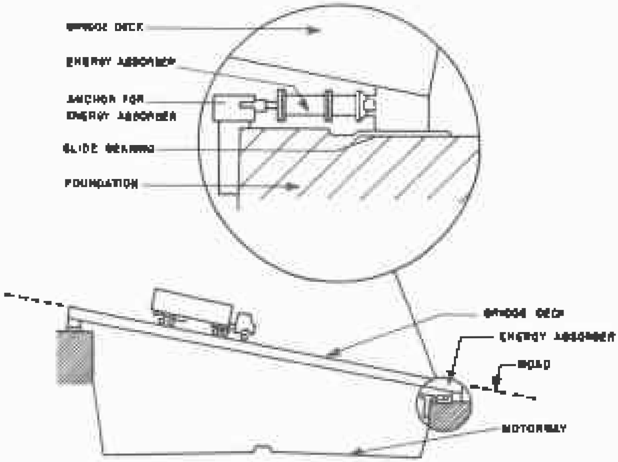

Figure 2 Application of extension energy absorbers (after Robinson and Greenback [4])

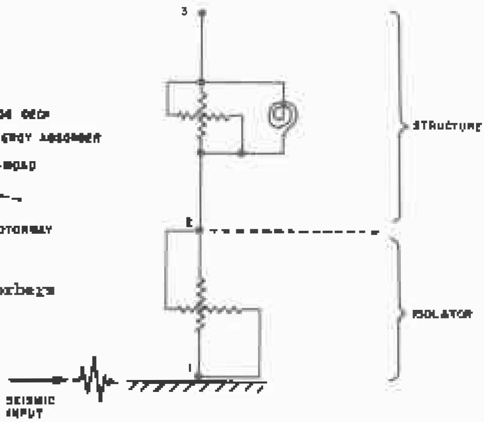

Figure 3 Simple structural models and spring isolation system

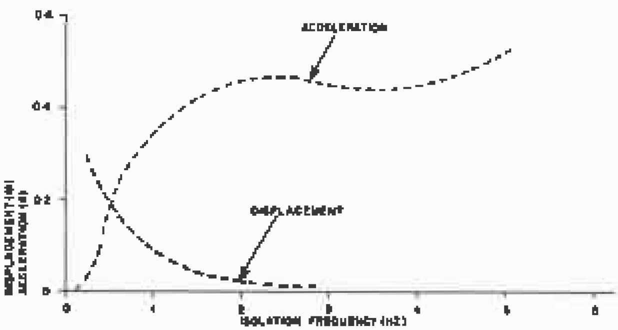

Figure 4 Peak acceleration and relative displacement vs isolation frequency for mass 2 – spring isolation

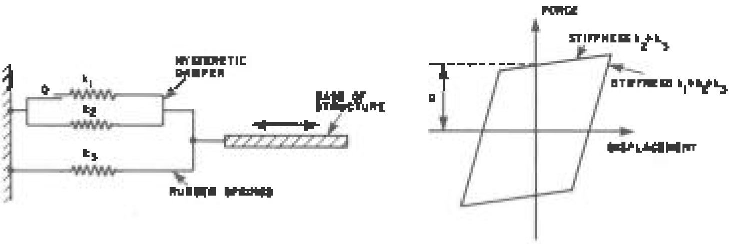

Figure 5 Model of spring and damper isolation systems

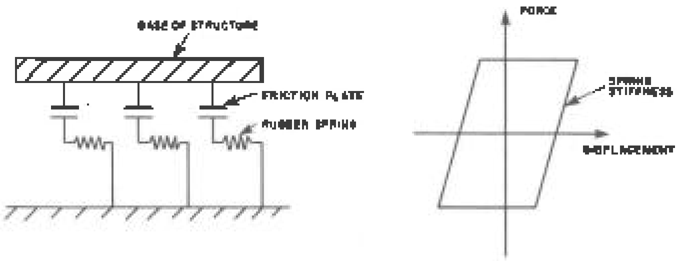

Figure 6 Model of spring and friction element isolation system

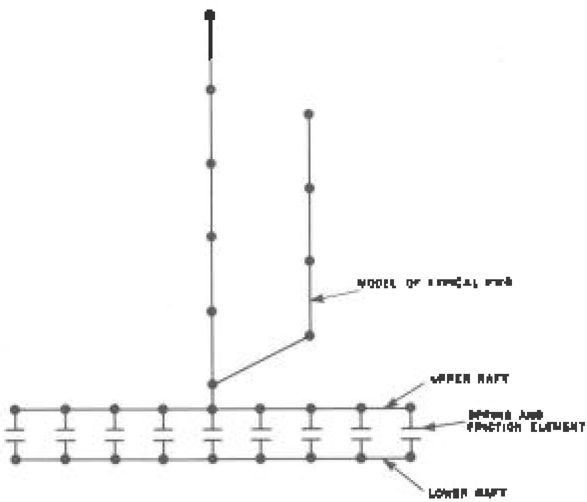

Figure 7 Structural response for spring and friction isolation system

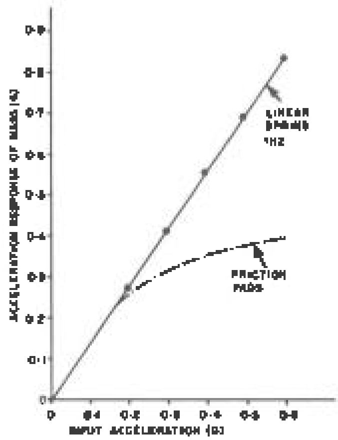

Figure 8 Model of PWR reactor building used for travelling wave study

Vibration Isolators as a Tool to Prevent Earthquake Damage

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SUMMARY

The objective of earthquake resistant design for a nuclear power plant is to be able to shut down safely during a major shock, and also to be able to restart afterwards, in such a way that no undue damage occurs to the structure and equipment. If a conventional method of design is followed, the additional cost of achieving earthquake resistance may be excessively high because, in the conventional philosophy of design, the entire structure and installations are subject to the full shaking of ground motion, causing large deformations and stresses throughout. The structure itself is expected to absorb the full earthquake energy by developing plastic hinges and cracks at critical points. Although the structure is not expected to collapse, stresses are allowed to go into the plastic range, and secondary elements like pipes, installations, etc., may undergo severe damage, endangering public safety and requiring excessive cost of repair.

When vibration isolation is used, however, all of these undesirable effects are avoided, and the earthquake safety of the nuclear power plant is ensured at a much less cost and with more reliable accuracy, because the ground motion energy is not transferred into the superstructure and is absorbed at the foundation level by means of appropriate shock absorbers. Neoprene pads and spring-dashpot systems are ideally suitable for this purpose.

The basic idea behind vibration isolation is to modify the natural period of vibration of the structure, to be markedly different from the predominant period of the ground motion and thus to reduce significantly the earthquake response of the structure. In fact, when vibration isolators are used, the entire building behaves almost like a rigid body, causing no internal stresses and displacements.

Helical springs and dashpots are proven to be an excellent arrangement for vibration isolation for all three directions of vibration. Neoprene pads, however, provide vibration isolation only in the horizontal direction. The pads are very stiff in the vertical direction and therefore the structure is exposed to severe internal stresses under vertical and rocking motions. It is shown explicitly, by means of a simple numerical example, that the underlying soil, however soft it may be, does not at all assist the neoprene pads in the vertical direction. The additional cost of earthquake resistance of a nuclear power plant may be in the order of 40% to 60% of the overall cost when conventional principles are used. This drops down to 3% to 6%, however, when vibration isolators are used.

1. INTRODUCTION

1.1 Conventional Design

The conventional principle of earthquake resistant design for reinforced concrete structures is to allow the primary structural elements to undergo large plastic deformations during a strong ground motion. The stresses are expected to go beyond the elastic range and plastic hinges and minor cracks are expected to develop at critical points such that the main load carrying system does not collapse. All nonstructural elements, such as plasters, partitions, piping system, etc., however, may be damaged, requiring extensive repair.

It is evident that the conventional principle of earthquake resistant design not only calls for additional cost due to the requirements of ductility of the load carrying elements, but also endangers the immediate and safe use of the building after a major earthquake, because of the possible extensive damages which occur in the nonstructural elements. There are other disadvantages of the conventional design principle:

a) Construction of highly deformable structural and nonstructural elements is more difficult and expensive.

b) If secondary elements, like the piping system in a nuclear power plant, fail to function properly, not only does the cost of repair become prohibitively excessive but also the safety of the public may be exposed to great danger.

c) Inelastic design, when compared with elastic design, involves greater complexity and approximations in material stresses, structural behaviour, and mathematical procedures.

1.2 Vibration Isolation

When vibration isolators are used under the foundation, however, almost all of the disadvantages of the conventional design principle are avoided. The energy of the ground motion is absorbed basically by the vibration isolators at the foundation level and the structure behaves almost like a rigid body with little or no relative storey displacements. The basic idea in vibration isolation is that the natural period of vibration of the structure is increased to a level much greater than the predominant periods of the earthquake ground motion. The dampers reduce the amplitudes of displacements. Consequently, all material stresses remain always within the elastic range.

2. TYPES OF VIBRATION ISOLATION

A variety of shock absorbing devices may be used as vibration isolators, such as mechanical (Refs. 1 and 6), neoprene pads (Refs. 7 to 10) and spring-dashpot systems (Refs. 11 to 14). The basic idea behind the vibration isolation is to modify the natural period of vibration of the structure and thus to reduce drastically the transfer of ground motion into the superstructure. The concept of vibration isolation is widely used all over the world in connection with machine foundations. Its application, however, to structures and nuclear power plants is relatively new.

The advantages of the use of helical springs and dashpots over the other systems as vibration isolators are discussed in great detail in refs. (13) and (14). Helical springs and dashpots are ideally suitable for the vibration isolation of nuclear power plants, since the isolation is efficiently achieved in all three directions of vibration. The neoprene pads provide vibration isolation only in the horizontal direction and they are ineffective in the vertical direction.

3. ISOLATION OF THE WHOLE NUCLEAR ISLAND

A nuclear power plant consists of various important structures like the reactor building, the fuel building, safeguard building, and the turbine building, etc. on the "nuclear island". These buildings adjacent to each other are interconnected by a network of electrical and mechanical installations and also by heating and other piping systems. If only one of these buildings is vibration isolated, there may be undesirable relative displacements between the adjacent buildings resulting in a possible damage of the installations. If the complete foundation raft of the whole nuclear island is vibration isolated, however, as shown in fig. 1, the whole island behaves like a rigid body during a strong ground motion and practically no relative displacements occur among the individual structures, thus the installations remain undamaged. In any case, the critical installations and piping elements, especially those extending from an isolated to a non-isolated building, should be supported and detailed by means of special energy-absorbing restrainers as described in reference (15).

4. MATHEMATICAL FORMULATION

For the purpose of determining the best suitable arrangement of helical springs and dashpots, the superstructure and the underlying soil are idealised into a planar or spatial mathematical model. It is normally sufficient to reduce the structure into as simple a mathematical model as possible, consisting of a sufficient number of lumped masses interconnected with one or two dimensional structural elements, since no complex behaviour occurs in the superstructure. Typical mathematical models are given in reference (12).

The subsoil medium may be represented by means of either an assemblage of finite elements or a series of lineal elastic springs. In this presentation, for reasons of simplicity in determining the effects of local soil conditions, the subsoil is represented by means of equivalent elastic springs. After the mathematical model is decided, the time history response to any selected ground motion may be determined by means of a numerical integration technique.

A typical one-mass mathematical model and its response to the horizontal components of the 1940 EL Centro ground motion are illustrated in figures 2 and 3. Similarly, the mathematical model of a seven-story building and its acceleration response are shown in figures 4 and 5 (ref. 12). It is seen that the response is significantly reduced when an appropriate arrangement of helical springs and dashpots is used. A more sophisticated mathematical model of a reactor building is illustrated in figures 6 and 7.

5. INFLUENCE OF SOIL CONDITIONS

5.1 Idealisation Scheme

In order to demonstrate the influence of local soil conditions on the vibration isolation, a simple numerical example is studied. The mathematical model of a typical nuclear reactor building given in reference (16) is idealised into a two-mass assembly of structure and soil as shown in figure 8. A single spring is sufficient to represent the vertical action of the soil, because only the tendency of the influence of the soil condition is investigated. Similarly, the superstructure is reduced to a single mass, because the whole structure moves like a rigid body causing almost no stresses or strains in the superstructure. Further, the purpose of this particular analysis is only to investigate the changes of natural periods of vibration due to different soil conditions.

5.2 Numerical Example

The total weight of the reactor building is given as W_2 = 45 000 ton in reference (16). The total weight of the lower foundation, combined with the phase in mass of the soil, is assumed as W_1 = 10 000 ton. The total horizontal stiffness of the reinforced elastomer bearing pads is calculated from equation (1) of the reference as $K = 1.12 \times 10^5$ ton/m. Assuming the vertical stiffnesses to be 800 times greater than the horizontal stiffnesses, the total vertical stiffness becomes $k_2 = 9 \times 10^7$ ton/m. Two different soil conditions will be considered:

a) "Soft soil" representing fine dense sand with an allowable bearing stress of 25 ton/m² requiring a mat foundation area of about 1800 m². Assuming the modulus of subgrade as k = 5 kg/cm³ the total vertical stiffness becomes $k_1 = 0.9 \times 10^7$ ton/m.

b) "Hard Soil" representing dense gravel or rock-like soil with an allowable bearing stress of 35 ton/m² requiring a mat foundation area of about 1300 m². Assuming the modulus of subgrade as k = 22 kg/cm³, the total vertical stiffness becomes $k_1 = 2.84 \times 10^7$ ton/m.

5.3 Changes in the Natural Period of Vibration

The natural period of vibration of a two-spring model is

$$T = 2\omega/\pi \quad \text{and} \quad \omega = \left(\frac{B \mp \sqrt{B^2 - 4A}}{2}\right)^{\frac{1}{2}} \tag{1}$$

where

$$A = \frac{k_1 k_2}{m_1 m_2} \tag{2}$$

$$B = \frac{k_1 + k_2}{m_1} + \frac{k_2}{m_2} \tag{3}$$

Incorporating the above mentioned numerical values in these expressions, the periods of vibration are obtained for "soft" and "hard" soil conditions as, T = 0.162 sec, and T = 0.098 sec, respectively. For infinitely rigid subsoil condition, the natural period of vibration in the vertical direction is T = 0.045 sec. It is seen that the influence of soft soil condition on the change of natural period of vibration is very insignificant.

5.4 Discussion

The soil under most nuclear power plants is hard soil. Even in the case of soft soil, its contribution to the vibration isolation is practically nil. In the case of neoprene pads, no vibration isolation is supplied for vertical and rocking motions of the structure, since neoprene pads are very stiff in the vertical direction. The claim that the subsoil conditions may assist the neoprene pads in the isolation of vertical motions is thus unquestionably disproved by the above numerical example.

Earthquakes generate three dimensional motions which may contain, close to the epicentral region, vertical accelerations as high as those in the horizontal direction. The structures possess inherently greater strength in the vertical direction thus being sensitive to higher frequency motions. Since the vertical components of earthquake motions contain relatively higher dominant frequencies, in order to prevent any quasi-resonance condition, the nuclear power plants must be appropriately isolated also in the vertical direction.

Helical springs and dashpots are ideally suitable for this purpose, since they may provide any desired amount of flexibility and damping in all three directions. Usually, the horizontal stiffness of helical springs is in the range of 2 to 5 times those in the vertical direction. The dashpots may supply damping values as high as 20% to 30% of that of the critical damping ratio. The coefficient of viscosity of the dashpots in the horizontal direction is normally 60% of that in the vertical direction (11, 13).

6. CONCLUSIONS

Some of the basic advantages of vibration isolation by means of helical springs and dashpots may be summarized as follows:

1) The additional cost of incorporating earthquake resistance to a nuclear power plant by the conventional principle may increase the overall cost by as much as 40% to 60%. The double mat foundations, helical springs, and dashpots, however, may increase the cost by only 3% to 6%.

2) A greater degree of safety and assurance is incorporated into the design, when compared with the conventional principle or the neoprene pads. There is no complex nonlinear behaviour, no possibility of slippage of pads even at accelerations as high as 1.0 g to 1.5 g, no possibility of magnified response in vertical and rocking motions.

3) The number of shut-downs in nuclear power plants will be much less, since the acceleration response in all three directions is greatly reduced. Further, a prompt and immediate restarting of the facilities becomes possible after a major earthquake, since neither the vibration isolation system nor the installations are expected to be damaged. In the case of neoprene pads, a difficult and sensitive adjustment procedure is necessary after a major earthquake.

4) Helical springs and dashpots are durable and less sensitive to temperature changes and to physical conditions of the air. It is very easy to replace any defective helical spring or dashpot.

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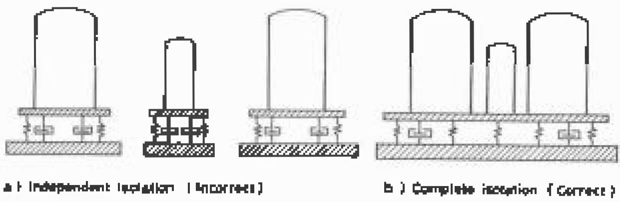

FIG 1.- VIBRATION ISOLATION OF THE NUCLEAR ISLAND

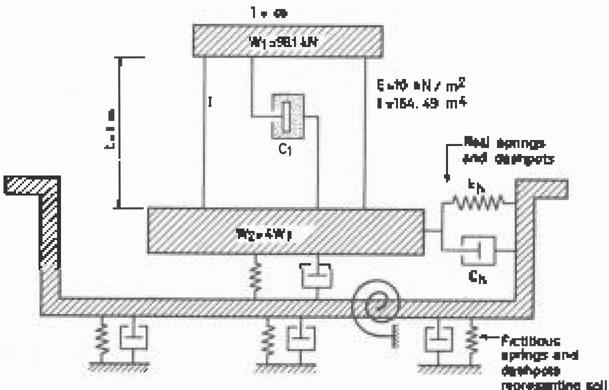

FIG 2.- A BASIC VIBRATION ISOLATION MODEL

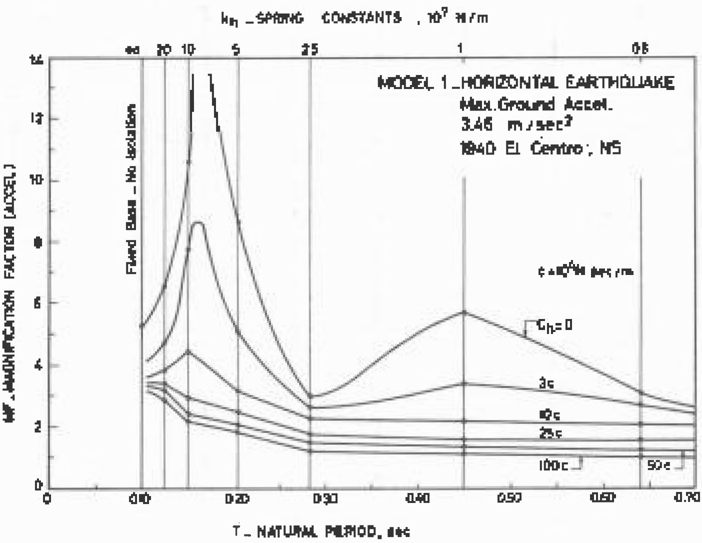

FIG 3.- EARTHQUAKE RESPONSE OF THE BASIC MODEL

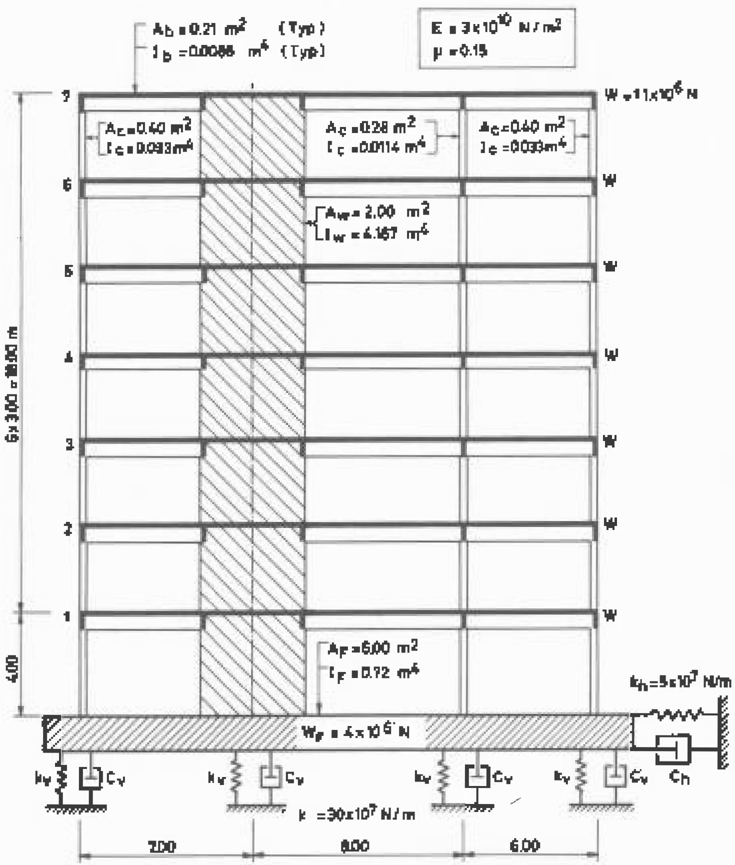

FIG 4.- SEVEN-STOREY BUILDING FRAME

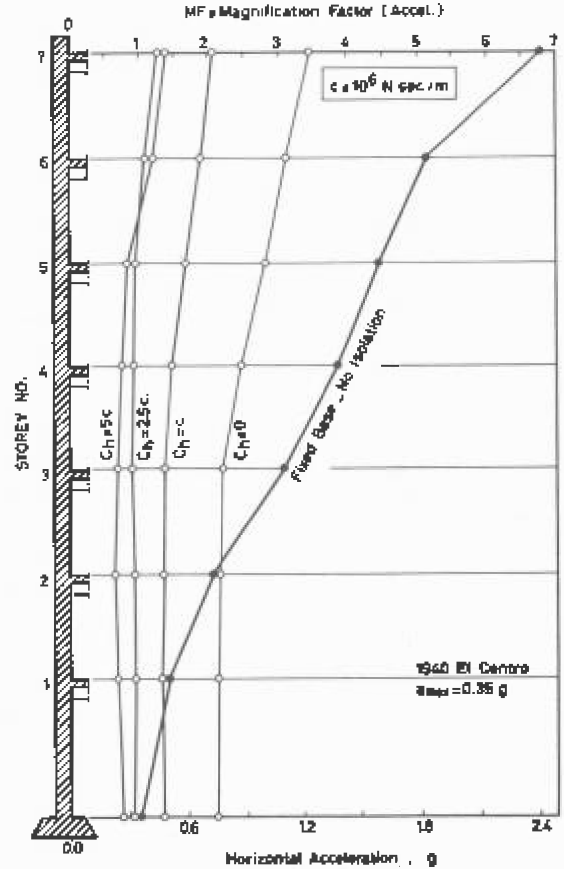

FIG 5.- ACCELERATION RESPONSE OF THE 7-STOREY BLDG

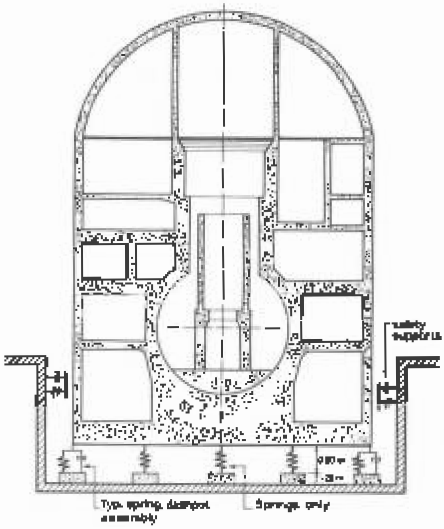

FIG 6.- A TYPICAL REACTOR BUILDING

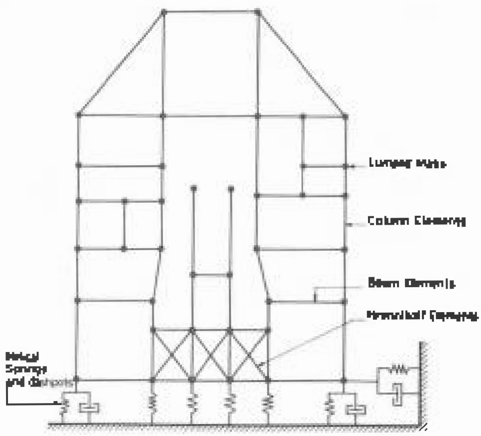

FIG 7.- MATHEMATICAL MODEL OF THE REACTOR BUILDING IN FIG 6

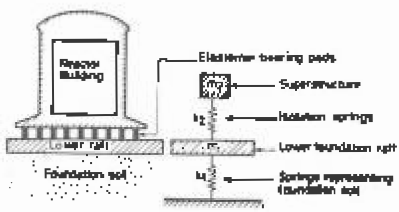

FIG 8.- TWO-MASS ASSEMBLY OF A REACTOR BUILDING

Effect of Travelling Waves on a Shock Isolated Nuclear Power Plant Put on Rock Foundation

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ABSTRACT

Conventional analyses of the earthquake response of structures are commonly performed with the hypothesis of vertically incident S and P waves (body) by applying a specified ground motion at the base of the model.

Many studies have been performed in order to determine the influence of eventual travelling surface waves. In general, the resulting stresses and displacements are no larger than those encountered in conventional analyses.

As for a Nuclear Power Plant with a conventional foundation, a shock isolated Plant is also calculated with the hypothesis of vertically incident body waves.

However differences between the designs, particularly in view of the large in-plane dimensions of the common raft associated with the shock isolated version, necessitate certain specific considerations.

The object of this study is to estimate the influence of travelling waves on a shock isolated Nuclear Power Plant and to check whether the same safety margin exists for travelling waves as for the body wave assumption.

This comparative study between travelling waves assumption and body waves assumption is performed in both cases of a shock isolated Nuclear Power Plant and of a conventional one (non-isolated) founded on rock.

The analyses give equivalent results for the shock isolated design and for the conventional design.

1. INTRODUCTION

When considering the effects of seismic loadings on the design of structures, soil-structure interaction analyses are performed on the basis that all points at the ground surface move in phase, horizontally and vertically. Such an assumption corresponds physically to vertically propagated waves : S-waves generate the horizontal motion, whereas P-waves generate the vertical motion.

However, during an earthquake, other types of waves are generated, such as surface waves (Love and Rayleigh) and other travelling waves. All these waves combine themselves and, due to multiple reflections and refractions in a soil or rock presenting heterogeneities, and at a ground surface showing mountains and valleys, result in a motion which is random not only in time but also in space, and which unless more data and calculation techniques are available, cannot be handled properly. In other words, we have certain knowledge about the frequency content, duration, attenuation but much less as far as the motion correlation between two adjacent points is concerned :

- With tele-seismic distant epicentres, all generated waves are damped out except long period surface waves. In this case, the motion travels at the ground surface with a given velocity.
- At intermediate epicentral distances, surface waves have been observed as for the San Fernando (9 Feb 71) earthquake owing to the Californian site conditions (active fault, shallow focus, alluvium) (1).
- With very short epicentral distance (a few kilometers), the presence of surface waves is not apparent and if so, they would be totally masked by the tremendous direct body waves emitted from the source, and which represent the main part of the tremor. In this case, the standard assumption of body waves can be considered as adequate. This is the case for most Nuclear Power Plants for which the SSE input data used for design generally corresponds to near-field events.

Due to the lack of data which could support a new design input available for small or intermediate distances and based on another wave propagation assumption, all calculations dealing with any other assumption, such as horizontally travelling waves, must be considered as pure mathematical exercices and the results carefully analysed. Many experts have already analyzed the influence of horizontally travelling waves in comparison with the body waves assumption and concluded that the standard assumption associated with usual material safety factors leads to satisfactory results (2).

The aim of this paper is to present such a comparison performed on a shock isolated (aseismic) structure and on a conventional (non isolated) structure which are designed on the basis of the body waves assumption and to determine the safety margin in the case of travelling waves assumption.

The shock isolated structure uses the aseismic bearing system (neoprene + friction plates) (3)(4).

The site corresponds to a rock site. In order to make a proper comparison, this study is performed with the same seismic input. This input has an horizontal velocity V = 1600 m/s (shear wave velocity of the rock) or V = infinite. These two velocities correspond respectively to the travelling and long wave assumption.

The comparison is evaluated for the conventional and shock isolated models by means of relative responses under the travelling wave and body waves.

In a second step, the analysis was developed evaluating the safety margins for both structures under both type of waves.

2. TRAVELLING WAVES - GENERAL

As far as the infinite half space is concerned, one can analyse the propagation of travelling waves (5). The equation of motions of the soil particle is:

$$u = \sin\left[2\pi\left(\frac{t}{T} - \frac{x}{L}\right)\right]\left(-\frac{2-k_1^2}{2\alpha_1}\exp\left(-2\pi\alpha_1\frac{z}{L}\right) + \frac{1}{\alpha_2}\exp\left(-2\pi\alpha_2\frac{z}{L}\right)\right)$$

$$v = \cos\left[2\pi\left(\frac{t}{T} - \frac{x}{L}\right)\right]\left(\frac{2}{2-k_1^2}\exp\left(-2\pi\alpha_1\frac{z}{L}\right) - \exp\left(-2\pi\alpha_2\frac{z}{L}\right)\right)$$

Where :

t = Time
T = Period
x = Horizontal distance
z = Depth (downward direction positive)
L = Wave length
$k1 = \frac{Vr}{Vs}$ $k2 = \frac{Vr}{Vp}$

Vr = Rayleigh wave velocity
Vs = Shear Wave velocity
Vp = Compression wave velocity
$\alpha 1 = \sqrt{1 - k_1 2}$
$\alpha 2 = \sqrt{1 - k_2 2}$

These equations show that the surface motion is a retrograde ellipse in the vertical plan of the propagating direction. These equations show that the amplitude of the motion decreases with depth, this reduction being particularly great the higher the frequency.

The velocity of propagation of the Rayleigh wave Vr is slightly lower than Vs. The ratio k1 is determined by the Poisson's ratio following the equation :

$$\frac{1}{8}k_1^6 - k_1^4 + \frac{2-\nu}{1-\nu}k_1^2 - \frac{1}{1-\nu} = 0$$

One can evaluate as a function of the hypocenter depth h the minimum epicentral distance which corresponds to the generation of surface waves. This range is further from the epicenter than the minimum distance S_p and S_s (5) :

$$Sp = \frac{k2\ h}{\sqrt{1 - k_2 2}}$$

$$Ss = \frac{k1\ h}{\sqrt{1 - k_1 2}}$$

ν	.25	.33	.40	.50
Sp Ss	.625 h 2.33 h	.577 h 2.59 h	.416 h 2.83 h	0. 3.25 h

The case of a horizontally layered soil profile has been also investigated. In this case there are higher Rayleigh modes. The velocity of the Rayleigh waves depends on the frequency. The higher frequency has a velocity corresponding to the first upper layers whereas the low frequency has a velocity corresponding to the deeper layers (7).

3. COMPARATIVE STUDY OF THE RESPONSES OF CONVENTIONAL DESIGN AND ASEISMIC DESIGN

The aim of this section is to evaluate the influence of travelling waves on the shock isolated design by comparison of the responses of two identical structures (group of buildings of the Nuclear Power Plant) one resting directly on the rock, the other put on aseismic bearings using the body wave assumption and the travelling wave assumption.

3.1 Seismic input

On account of the actual properties of the site (postulated fault at 8 km from the site, non shallow focus (in the order of 20 km), a very conservative assumption of travelling waves input consists of using the whole body waves record as a surface wave, travelling at a horizontal velocity v = 1.600 m/s.

In fact, the foundation mat of the structure is rigid in the horizontal direction and it will tend to average the ground motion. When the assumption of the uniformity of the input motion is abandoned, it may lead to a reduction of the translation motion (self cancelling effect). (8) (9)

The body waves inputs are the horizontal and vertical Artificial Time Histories SSE developed for the specific site. The calculations are performed in the time domain because of the non linear behaviour of the aseismic bearing system.

3.2 Mathematical model

The two mathematical models are lumped-mass models (Fig. 1). The first one (Conventional Model) represents two Reactor Containments, their Internal Structures and Nuclear Auxiliary Building on a common raft placed directly on the bedrock. The second one (Shock Isolated Model) represents the same buildings put on aseismic bearings.

- As far as the structural model is concerned, the individual structural stiffness and mass matrices of each building are obtained on the basis of static condensation of complete finite element model and then are set up so as to achieve the total stiffness and mass matrices.
- As far as the bedrock model is concerned, an equivalent infinite half space is derived using a method outlined by Christiano (10). Then stiffness matrices are obtained by inversion of the flexibility matrices calculated by means of Green's functions determined for an harmonically excited point load applied to the half space. Due to the non-linear behaviour of the structure, the analysis is performed in the time domain and consequently there is no possibility of accounting for frequency dependant parameters. The equivalent static stiffness and equivalent damping are adjusted following the frequency of the fundamental modes in the horizontal and vertical directions. As the horizontal and vertical matrices are separated, it is possible to choose adequate frequency in both directions.
- As far as the aseismic bearings model is concerned, the horizontal and vertical stiffnesses of the pads (neoprene + concrete pedestals) can be derived from the horizontal frequency and the vertical deflection d under the weight of the Nuclear Power Plant. The horizontal frequency N is chosen to be about 1 Hz. The friction coefficient of the friction plates is taken f = 0.25.

3.3 Methodology

A Nuclear Power Plant in a seismically active region has to be designed so as to withstand seismic load case + non seismic load case. As the seismic response of a shock isolated structure is smaller than that of a conventional structure, the seismic case of loading represents part of the total dimensioning loading for the former, whereas it represents most of these loading for the latter. Then the comparison between both models is made on the basis of total loadings by evaluating amplification factors (ratio between the total sollicitation under the travelling wave assumption and the total sollicitation under the body wave assumption). Evaluating the tension of prestressing cables and of compression and shear of concrete are achieved for different levels in the containment and internals.

An amplification factor higher than 1.0 means that the travelling waves assumption leads to higher results than the body waves assumption.

3.4. Main results

- The amplification factors in the shock isolated model are sometimes slightly higher than 1.00 (although they remain smaller or comparable with those of the conventional model).

Section	Stress	Amplification factor	
		aseismic	conventional
Containment top (level + 45.00)	Tension in prestressing	1.00	1.01
	Compression in concrete	1.00	1.01
Containment bottom	Tension in prestressing (- 4.50)	1.00	1.11
	Compression in concrete (0.50)	1.00	1.08
	Shear in concrete	1.06	1.31
Internals (- 3.50)	Compression in concrete	0.96	1.00
	Shear in concrete	1.02	0.97

- the 2 % damping ratio horizontal and vertical floor response spectra are computed at the top of the internal structures for the shock isolated and conventional models. They show some differences for both models (due to the excitation of the rocking modes by the travelling waves input).
- For a few points design values could be exceeded in both cases. The analysis of the safety margin domain was then developed.

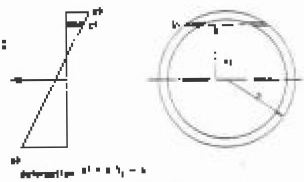
4. SAFETY ANALYSIS

Due to the arbitrary nature and doubt on the existence of such waves at small epicentral distances and to the unreliability of the arbitrary assumptions, such a study introduces conservative results. Consequently the values obtained should not be compared with the design limits, but should at the extreme be considered as values resulting from "damage studies" : damage study is an analysis performed to evaluate what happens beyond the design limits. These types of studies are made in the plastification or Ultimate Limit State domains.

Ultimate Limit State Analysis is used here. A global verification and a shear verification are performed for two critical areas (Base of the Containment and of Internal Structures). This consists of taking the Plant design (concrete, reinforcement, prestressing), evaluating the safety domain of the Plant and plotting on the same diagram representative points of SSE conditions (body waves and travelling waves conditions (v = 1 600 m/s).

4.1 Global verification - Interaction diagram

- Let a section be submitted to a normal force N and a moment M. The static equilibrium will be achieved after a deformation of this section. The basic assumption consists in a linear deformation of the section : the plane surface remains plane.



- N and M are in equilibrium by stresses in concrete and steels :

$$N = \sum_i \sigma c(\varepsilon i) Si + \sum_i \sigma_{a_1}(\varepsilon i) Si . \alpha_1 + \sum_i \sigma_{a_2}(\varepsilon i) Si . \alpha_2 + \ldots$$

$$M = \sum_i \sigma c(\varepsilon i) Si . \; i + \sum_i \sigma_{a_1}(\varepsilon) Si . Yi . \alpha_1 + \sum_i \sigma_{a_2}(\varepsilon i) Si . Yi . \alpha_2 + .$$

$\varepsilon c(\varepsilon i)$, $\sigma a_1(\varepsilon i)$, $\sigma_{a_2}(\varepsilon i)$ are the stress/strain relationships of concrete and steels (we can have different kinds of steels in the section) (Fig.2). $\alpha 1$, $\alpha 2$... are the steel/concrete surface ratio.

- The ultimate strains accounted for are :
  tension : steel 1 %
  compression : concrete 0.2 % (in the case of thin cylinders)

There are 3 kinds of stress/strain relationships for the steel in the Containment (Liner, Rebars, Prestressing...) Only the rebars are used for the Internals. The concrete parabola-rectangle curves are given for both Structures (Fig 2). These results are presented for each section and for the different load cases :

(1): Normal Operating (NO)	(3):NO+LOCA+SSE
(2): NO+LOCA (pressure effect)	(4):NO+LOCA+Travelling waves (1600 m/s)

The diagram showing sets (M,N) for which the strain εa or εb reaches the ultimate values of concrete or steel is called the interaction diagram.
The LOCA (pressure effect) has no effect on the Internals. From these results, it turns out that the SSE and travelling waves load cases are close to and located well inside the interaction diagram (Fig 4, Fig 5).

4.2. Shear verification

For each previous load case one can compute for each point of a section the horizontal compression σ'_y the vertical compression σ'_x and the shear stress τ, σ'_y depends on the horizontal prestressing and pressure effect of the LOCA.

The safety domain for the (σ'_x, τ) set values is defined on the basis of the French I.P. 2 recommendations (11) :

$$\tau^2 \leq (0.25)^2 . \left[1 + \frac{\sigma' x + \sigma' y}{\sigma' j}\right]^2 \left[\frac{\sigma' j}{\gamma b} - \sigma' x - \sigma' y\right] . \left[k \sigma j + \sigma' x + \sigma' y\right] + \sigma' x . \sigma' y$$

- The calculations are performed for different angular values of the circular section at the bottom of the containment in the NO case of loading and NO + LOCA case of loading.
  The results are always located well inside the safety domain. (Fig 6, Fig 7).

- As far as the Internal Structures are concerned, the horizontal compression is zero (no horizontal prestressing nor LOCA pressure effect).

  The vertical compression is calculated from the NO + seismic cases of loading (M, N). The shear stress is induced at the very bottom of the Internals by the shear force T and the LOCA.

  The points representative of SSE conditions and travelling waves conditions are located on the (τ ,σ) diagram. Three safety domains are evaluated using different rules (IP 2, CCBA 68, BAEL, ...) and included the previous points. (11, 12, 13).

5. CONCLUSION

The conventional assumption of seismic analysis is based on independant horizontal and vertical ground motions. As it is not possible to identify the contribution of each type of wave, an arbitrary assumption for evaluating the effect of travelling waves is taken. It consists of applying the total seismic motion as a surface motion, which travels at the shear wave velocity. For a Nuclear Power Plant sited on rock, the travelling wave assumption in comparison with the body waves assumption leads to equivalent results, for the shock isolated and for the conventional design. Although a few travelling wave results exceed the design values of the body waves assumption, they remain well inside the safety domain defined on the basis of the Ultimate Limit State. For a more refined evaluation of the effect of the travelling waves phenomenon, it will be necessary to correctly identify the ratio between the body waves and the travelling waves, in order to limit the conservatism of the travelling waves input.

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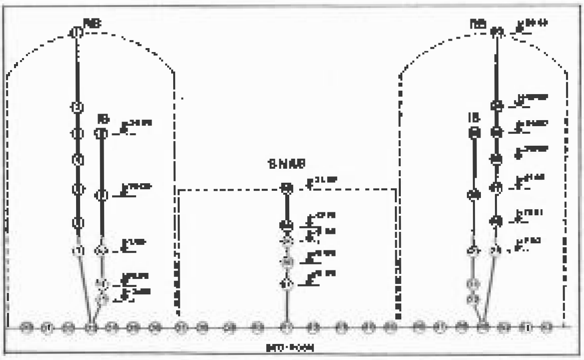


Fig.1 : Conventional Model

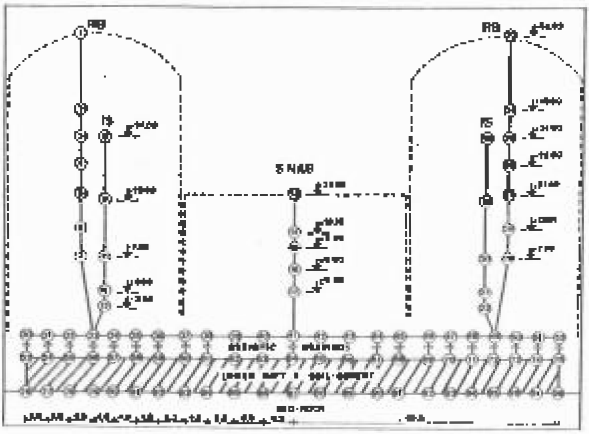
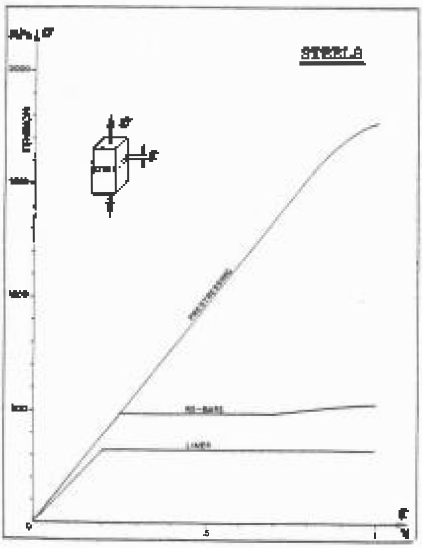


Fig.1b : Shock Isolated Model



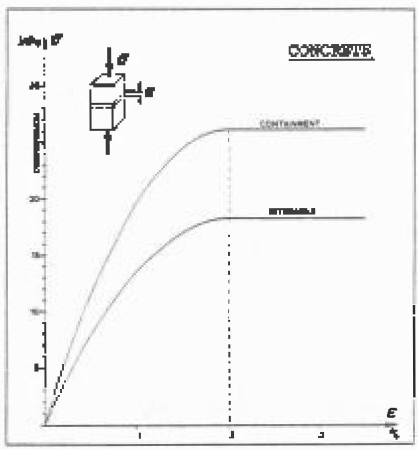
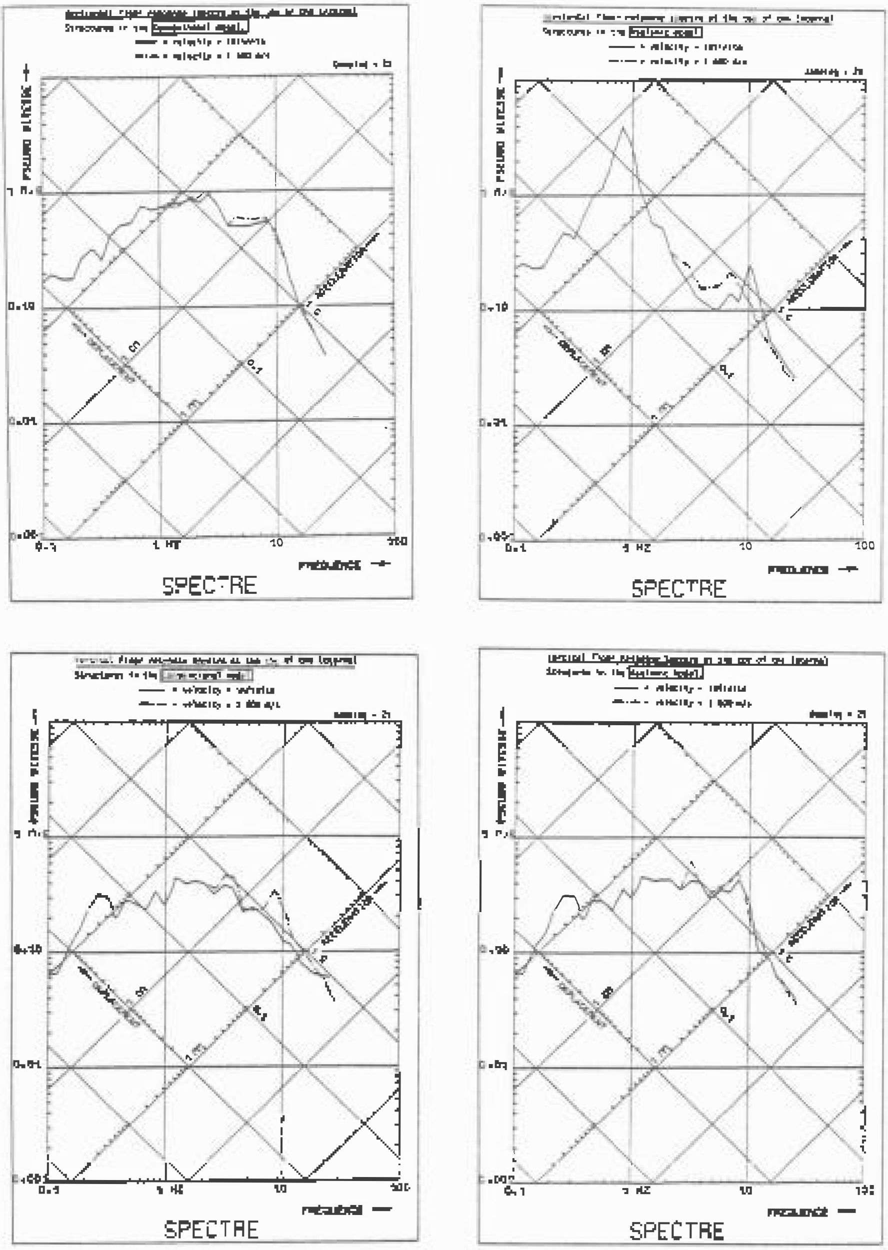


Fig.2 : Stress/Strain relationships of steels and concrete



CONVENTIONAL MODEL

SHOCK ISOLATED MODEL

Fig.3 : Horizontal and Vertical Floor Response Spectra at top of Internal Structures for Conventional and Shock Isolated Models.

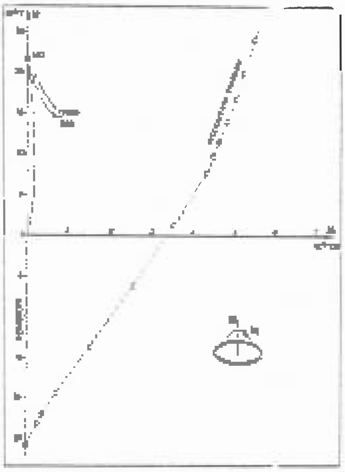


Fig. 4 : Internal Structures
Level -3.50
Global Verification

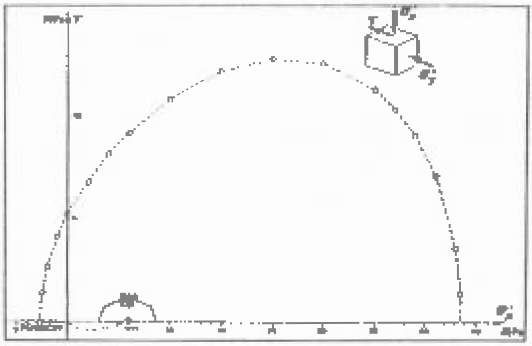


Fig. 6 : Containment - Level -0.50
Shear Verification (N.O.)

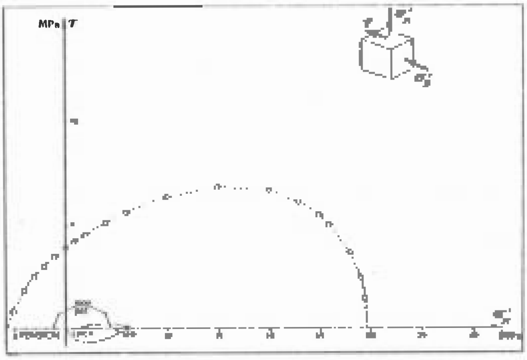


Fig. 7 : Containment Level -0.50
Shear Verification (N.O. + LOCA)

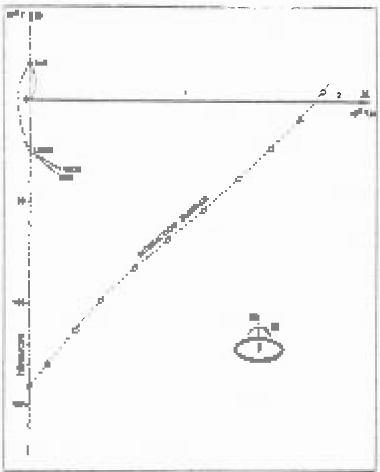


Fig. 5 : Containment Level -0.50
Global Verification

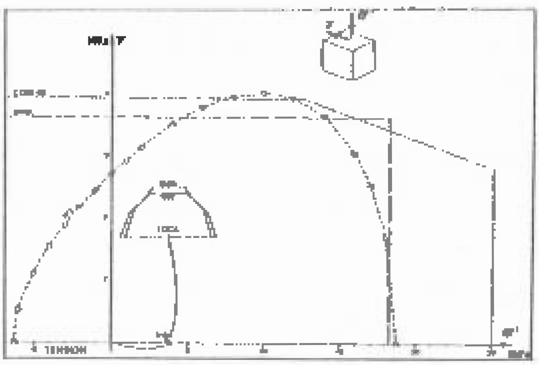


Fig. 8 : Internal Structures Level -3.50
Shear Verification (NO + LOCA)