Seismic Isolation for Nuclear Power Plants

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This paper describes the key aspects of the decision to use seismic isolation in nuclear power plant design. To date, only French nuclear plants have used seismic isolation. This paper examines why this is the case, and what are future prospects of isolation systems.

Seismic isolation is a relatively new design philosophy. Data are presented showing the increased quantity of research done in this field during the 1970's. This increase is correlated to the appearance of actual application of seismic isolation systems, in the late 1970's.

Seismic isolation reduces the seismic response of the nuclear plant. Acceleration and base shears decrease by factors of 3. Floor response spectra decrease by factors up to 10, with typical spectral peak reductions from 21g to 3.4g.

Seismic isolation controls the shape and frequency content of the design response spectrum at the nuclear plant foundation level. This can reduce risk from finding near field faults after the plant has been designed.

Seismic isolation allows the use of standardized nuclear plant designs in high seismic regions. This affects cost of the plant, and is perhaps the most compelling reason to use seismic isolation in nuclear plant design.

1.0 INTRODUCTION

Seismic isolation of nuclear power plants is an excellent concept. There has been recent increased interest in the concept, and seismic isolation has already been applied by the French for selected nuclear power plants in South Africa and France. Questions facing the engineering community are why have only the French applied this design concept, and what are the prospects for future use of this design concept.

This paper presents four items the engineer must consider in deciding to select a seismic isolation system for use in a nuclear facility. First, the engineer requires data on how to design, analyse and construct seismic isolation systems. Tables are presented which summarize the large quantity of seismic isolation research and the practical applications that have been carried out over the past few years. Second, a seismic isolation system provides improved seismic performance of the nuclear plant. Analytical and experimental results are presented which show the magnitude decreases in accelerations, base shears and floor response spectra for a seismically isolated structure. Third, a seismic isolation system can control seismic risk for the nuclear plant. This paper discusses how a seismic isolation system effectively filters the design response spectrum, thus alleviating the risk of the discovery of near field faults. Fourth, cost considerations play a major role in the selection process. This paper thus reviews how the Standardized Nuclear Power Plant SNUPPs concept becomes perhaps the most critical point in the final decision to employ a seismic isolation system.

2.0 HISTORICAL PERSPECTIVE ON SEISMIC ISOLATION AND MODERN APPLICATIONS FOR FOR NUCLEAR PLANTS

For the engineer to decide to use a seismic isolation system in a nuclear plant, he needs adequate data regarding how to design it, and what its performance record is. Table I presents an historical list of roughly the amount of technical research done on isolation systems. We notice from this list that there has been a large growth in research in this field since 1970. Table II presents a list of actual practical design applications of various types of seismic isolation systems. These two tables show that the quantity of technical research and practical applications appear to be correlated. Given that this recent surge in research provides enough background proving the merits of seismic isolation systems, the engineer is next faced with choosing an appropriate system for a nuclear plant.

In seismic isolation, the engineer wants to detune the nuclear plant structures from the input earthquake vibration. This concept is similar to the design for foundations of vibrating machinery. Potentially, then, a seismic isolation system could consist of a simple rubber bearing system, of adequately low stiffness, placed below the nuclear plant's foundation mat. The rubber bearings would lower the structure's predominant natural frequencies to a range well below where there is significant earthquake motion energy.

However, such a simple rubber bearing system has strong drawbacks. These drawbacks are the following: first, an unusual earthquake could occur, having dominant energy just at the very low frequency of the rubber-bearing-isolated nuclear plant; second, the nuclear plant may experience significant motions in the design wind load condition; third, if an earthquake occurs which actually exceeds the design basis earthquake, displacements could occur outside the capacity limits of the rubber bearings. This could fail the rubber bearings in an undesirable 'brittle' fashion.

In practical design, the engineer desires an isolation system which behaves in ductile fashion at the highest possible design loads. Thus, an extra component needs to be added to the rubber bearings, to provide a 'ductile' isolation system. At least four types of devices are commonly suggested in the literature (1, 2 and 3) that could be added to fulfill this function:

1) Steel energy absorbers.
2) Mechanical fuses.
3) Hydraulic dampers.
4) Friction plates.

In actual nuclear plant construction, a combination of friction plates and rubber bearings have been employed. A combination of steel energy absorbers and rubber bearings also provides an excellent isolation system. Mechanical fuses are not suitable, due to their brittle and somewhat unpredictable behaviour. Hydraulic dampers are not suitable for nuclear plant isolation systems due to their constant need for inspection, testing and replacement.

3.0 NUCLEAR PLANT PERFORMANCE ON A PRACTICAL ENERGY ABSORBING ISOLATION SYSTEM

The results which follow are based upon a seismic isolation system using rubber bearings and steel torsion energy absorbing devices. This system has been previously experimentally tested (4), and analytical results have been prepared for this report. The test structure and isolation system components are shown in Figures 1, 2 and 3. Table III describes the key frequencies of the structure, for both the conventional foundation (FIXED) and the isolated (ISOLATED) foundation. Figure 4 shows the response spectra of the input motion, El Centro NS 1940, scaled to have zero period acceleration of 0.63g.

The following paragraphs discuss the response of the isolated structure. Figure 5 shows the time history response of the top of the building, both for FIXED and ISOLATED conditions, when subjected to the El Centro motion. Figure 6 shows the corresponding time history of base overturning moments and shears.

Accelerations: The FIXED structure amplifies the ground ZPA to about 2g on top, even though the steel frame structure undergoes some yielding. In contrast, the ISOLATED structure reaches peak top story accelerations of only 0.61g, which shows no amplification of the ground motion ZPA.

Moments and Shears: The ISOLATED structure exhibits lower base overturning moments and base shears by a factor of 3 as compared to the FIXED structure.

Even more striking than the above structural response quantities is the comparison of the floor spectra between the FIXED and ISOLATED structures. These spectra are shown in Figure 7.

Floor Spectra: The ISOLATED structure exhibits average floor response spectra reductions of 2 to 3 times, from the FIXED structure. There is as much as 10 times reduction at certain resonance frequencies. For example, the spectral peak goes from 21g for the FIXED structure to 3.5g for the ISOLATED structure. The reader should note that such high (21g) spectra are not uncommon in actual nuclear plants designed in high seismic zones.

4.0 CONTROLLED SEISMIC RISK FOR ISOLATED NUCLEAR PLANTS

As discussed in the previous section, an isolated nuclear plant easily out performs a conventional fixed-base plant, when both are subjected to the same earthquake motion. In essence the isolation system modifies the earthquake motion that actually reaches the nuclear plant. The engineer can use this fact in assessing the seismic hazard risk for a nuclear plant.

One major source of seismic 'risk' lies in our incomplete knowledge of what the 'worst possible' earthquake will be at a particular site. This incomplete knowledge can in certain instances become a large risk, as, for example, the case of the Diablo Canyon nuclear plant in California. After the seismic design had been initiated for this nuclear plant, a new near-field fault, known as the Hosgri fault, was discovered. The presence of this fault has led to much concern for the seismic safety of Diablo Canyon.

For the base isolated nuclear plant, however, the discovery of a major near field earthquake fault may not adversely modify the seismic response of the plant. This is because the earthquake motion the plant actually experiences is only that motion which has been filtered through the base isolation system.

Figure 8 shows, schematically, the difference in the response spectra just below and above the base isolation system described in section 3 of this report. We observe here that the isolation system is effective in controlling the frequency content and acceleration amplitude of the motion actually 'felt' by the structures.

Thus, should a new 'design response spectra' be required for a nuclear plant, due to discovery of a new fault, or increased regulatory concern for safety, than a base isolated plant has an inherently built-in margin against this form of seismic risk.

5.0 SEISMIC ISOLATION AND THE SNUPPs DESIGN CONCEPT

One practical method engineers have used to control the high cost of nuclear plants is to use the Standardized Nuclear Power Plant (SNUPPs) design philosophy. Essentially, SNUPPs means that one set of engineering calculations, and construction drawings, can be used over and over again for several nuclear plants of the same design, even at different sites.

To date, the French have been the most successful group to use the SNUPPs design philosophy. Their performance record in building the 900 MWe series is admirable. Several units have come on line each year for the past few years, and a total of thirty four units of the 900 MWe series have been ordered. Thus, SNUPPs is a practical time and cost saving design philosophy.

However, the engineer loses the benefits of using the SNUPPs design for a nuclear plant in a high seismic region. For example, the French 900 MWe SNUPPs plant is built to withstand a design response spectra earthquake roughly equivalent to a NRC Regulatory Guide 1.60-0.20g motion. However, for higher seismicity sites, to about 0.30g, the French have found it practical to retain the SNUPPs design, when mounting the nuclear plant on a base isolation system. This is done at Koeberg and Cruas. The base isolation system effectively filters the higher 0.30g motion down to an equivalent 0.20g motion.

By so using the base isolation system, substantial cost savings can be achieved. Essentially, the extra cost required to build the special isolation foundation system is offset by the cost savings from the reduced engineering effort needed for a SNUPPs plant, and the less elaborate earthquake protection within containment than would otherwise have been needed.

As of 1983, almost no SNUPPs plants have been built in the United States. This is chiefly attributed to the historical diversity in American plant design. In the U.S., there are several reactor vendors, more than a dozen architect-engineers, and very many nuclear plant utility owners. This lack of SNUPPs plants has reduced the need for designing a seismic isolated nuclear plant in the U.S.

6.0 CONCLUSION

We have discussed in this paper four major aspects in the decision process to employ a seismic isolation system in nuclear plant design. These aspects are the availability of a proven track record for such systems, the lowered seismic response and floor response spectra, the improved seismic safety, and the cost savings when seismic isolation is used in SNUPPs plant design.

Seismic isolation systems could play a role in the U.S. nuclear industry at some future date. The U.S. reactor vendors are moving closer to developing a NRC licensed SNUPPs plant design. Thus, in high seismic zones, a base isolated SNUPPs design may become a practical solution for U.S. domestic or export nuclear plants in the near future.

7.0 REFERENCES

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2. Kelly, J.M., 'Testing of a Natural Rubber Base Isolation System by an Explosively Simulated Earthquake', Earthquake Engineering Research Center, EERC 80-25, 1980.

3. Kelly, J.M. and Skinner M.S., 'The Design of Steel Energy Absorbing Restrainers and Their Incorporation into Nuclear Power Plants for Enhanced Safety: Volume 4 - A Review of Energy Absorbing Devices, EERC 79 - 10, 1979.

4. Kelly, J.M., Eidinger, J.M., Derham, C.J., 'A Practical Soft Story Earthquake Isolation System', EERC 77 - 27, 1977.

TABLE I - Published References on Base Isolation and Energy Absorbing Systems

		MAIN TOPIC(S)			
YEAR	TOTAL REFERENCES	SEISMIC ISOLATION	ENERGY ABSORBERS	TEST RESULTS	ANALYTICAL RESULTS
1972	2	2		1	1
1973	4	1	3		1
1974	4	4			4
1975	7	5		1	6
1976	5	2	3	1	1
1977	13	8	5	7	6
1978	11	5	5	3	5
1979	8	7	5	4	4
1980	12	3	9	5	4
1981	8	7	1	3	4
TOTAL	74	44	31	25	36

TABLE II - Applications World Wide of Seismic Isolation Systems

Application	Components	Country	Date of Application
Rangitikei - Railway Bridge	Energy Absorbers Rubber Bearings	New Zealand	1976
School	Rubber Bearings Mechanical Fuses	Yugoslavia	1969
High Rise Office Building	Rubber Bearings Mechanical Fuses	Greece	1972
Nuclear Stations Koeberg	Rubber Bearings Friction Plates	South Africa	1975
Nuclear Stations Cruas	Rubber Bearings	France	1980
Office Building 4 Story	Rubber Bearings Lead Plugs	New Zealand	1980
Schools	Rubber Bearings	France	1977 - 80
Chimney	Energy Absorbers	New Zealand	1980
Railway Bridge Decks	Rubber Bearings	New Zealand	1970's

TABLE III- Structure Frequencies

	FIXED FOUNDATION		ISOLATED FOUNDATION	
	1ST Mode	2ND Mode	1ST Mode	2ND Mode
FREQUENCY	2.27 Hz	7.83 Hz	0.68 Hz	3.84 Hz
MODE SHAPE				
3RD FLOOR	1.00	1.00	1.00	0.97
2ND FLOOR	0.77	-0.62	0.98	0.40
1ST FLOOR	0.41	-1.23	0.94	-0.34
BASE FLOOR	--	--	0.89	-1.00

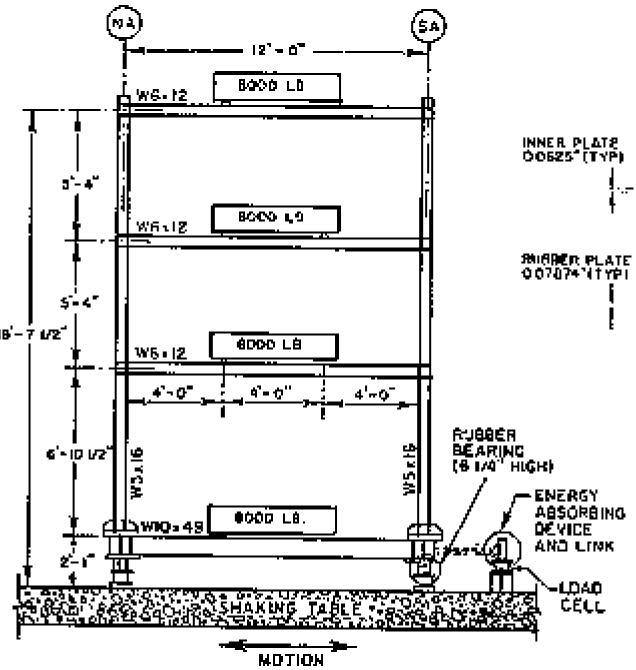

Fig. 1. Three Story Building on ISOLATED Foundation.

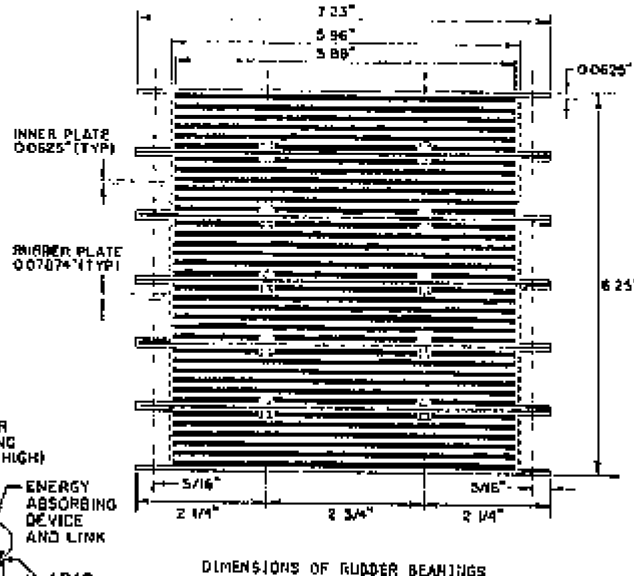

Fig. 2 Rubber Bearing Component

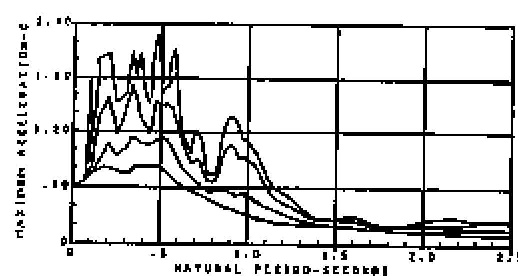

Fig. 4. El Centro Input Motion, Scaled to 0.63g.

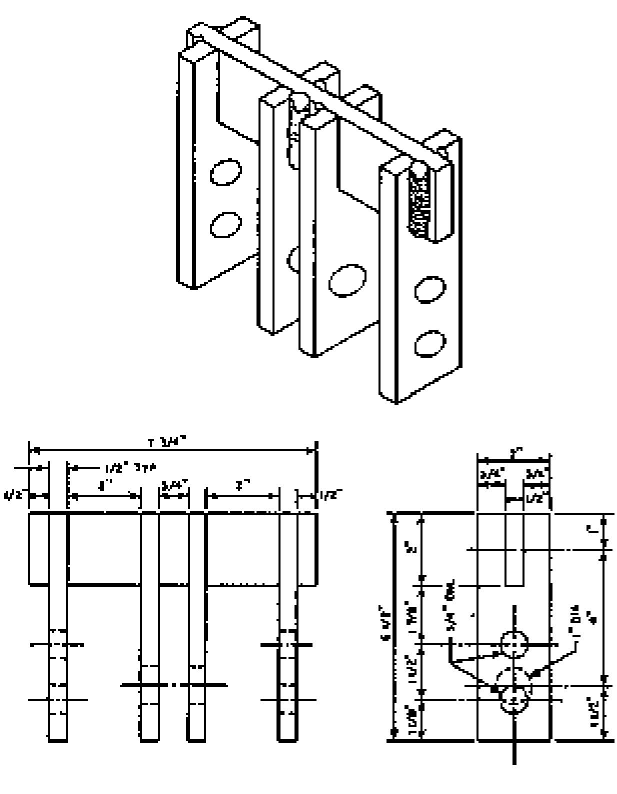

Fig. 3 Energy Absorbing Device Component.

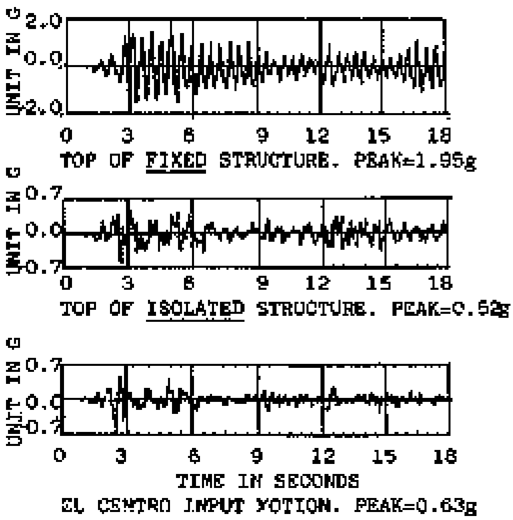

Fig. 5 Time History Response of ISOLATED and FIXED Base Structures - . Accelerations.

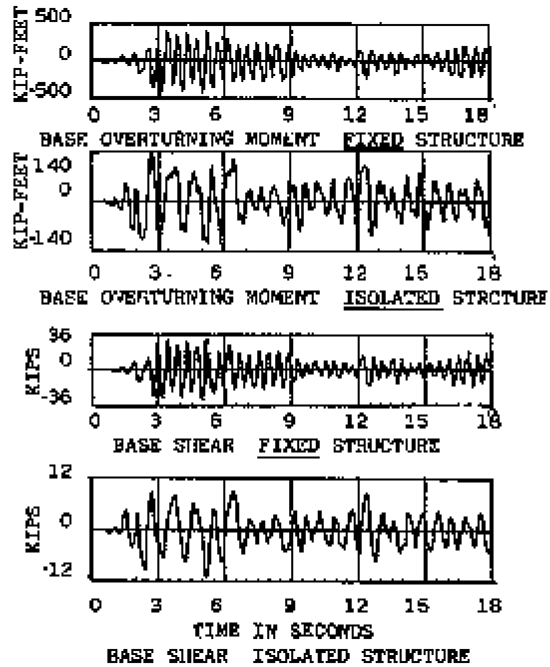

Fig. 6 Time History Response of ISOLATED and FIXED Base Structures - Base Shear and Base Overturning Moments.

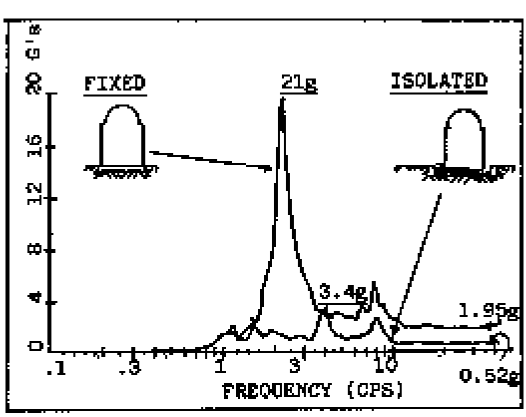

Fig. 7 Floor Response Spectra Comparison: Top of ISOLATED and FIXED Base Structures.

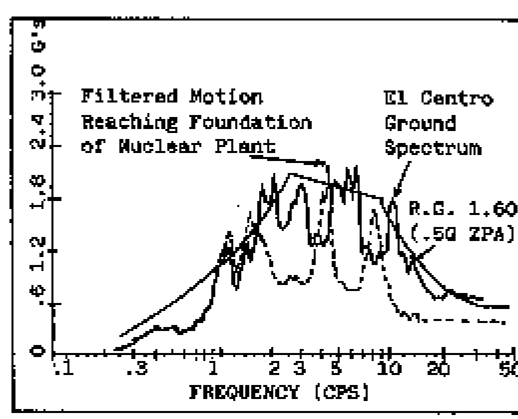

Fig. 8 Filtering effects of Isolation Systems - Design Response Spectrum.

Alexisismon Isolation Engineering for Nuclear Power Plants

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Summary

The constant evolution of the seismic maps has brought to a sharp focus the reliability of seismic input as well as the lack of large seismic margins in nuclear power plants. This lack is due to economical reasons since designing strength into a classical structure becomes very costly as the intensity of the earthquake increases.

Today, new structural designs using seismic isolation systems can provide strongly increased seismic safety economically and they are regarded as serious alternatives to classical designs for structures to be built in seismic areas.

The feasibility of using a seismic isolation design in a nuclear power plant from the viewpoints of licensibility and efficiency is studied in this paper.

In order to facilitate the licensing process, it must be possible to predict the behavior of the isolated plant by means of the established analysis procedures: no deviation from the classical soil/structure interaction, use of classical extreme loading distribution patterns, accommodation of potential loss of contact between structure and foundation, etc.., Severe requirements related to the isolation system itself are proposed: performance, verification of performance, etc.., In view of the need for high seismic safety margins, it is suggested that an isolation system should not allow the seismic base shear to exceed 10 % of the weight of the structure for all recorded accelerograms scaled to 1.50 g peak acceleration minimum. This makes possible the future standardization of the plants design.

1. Introduction

The recording of always more intensive accelerograms, the discovery of new faults, etc.., have led to the revision of the seismic criteria of several sites in the U.S. and in other seismic countries and have brought to a sharp focus the reliability of seismic input as well as the absence of large seismic margins in major structures such as nuclear power plants. In addition to the problem of uncertainty in determining seismic input there is another parameter which has become important and that is excessive costs. It is a well known fact that to provide adequate seismic resistance in the classical sense -- i.e., by designing strength into the structure -- becomes very costly as the intensity of the earthquake increases; in fact, a point is eventually reached where it becomes also technically impossible to provide the strength levels required.

Today, new structural designs using seismic isolation systems can provide strongly increased seismic safety economically and in fact with lower cost and they are regarded as serious alternatives to classical design for structures to be built in seismic areas.

Various isolation designs have been proposed all around the world (ref. 1 to 8) and some of them have already been implemented. Among the latter is the Alexisismon system (ref. 8 to 15) which was developed by the author from 1968 to 1972 and was implemented for the first time in 1972 in a multi-story building. Its concept and different schemes have been patented in the U.S. and in other seismic countries and its efficiency when applied to a nuclear power plant was demonstrated in a paper presented at SMIRT-6 in Paris (12).

The purpose of this paper is to address general problems of providing seismic isolation to nuclear power plants.

It must be first noted that, although nuclear power plants have specific architectural, structural and mechanical characteristics, as major structures usually have, none of them deviates so much from common design that it can prevent seismic isolation from being incorporated or operating, as it does in other structures, provided their secondary effects are reliably accommodated by the seismic isolation system.

The major concerns about seismic isolation applied to a nuclear power plant which are likely to be shared, to different extents, by owners, designers, constructors and authorities are (a) the licensibility of an isolated power plant, (b) the efficiency of its seismic isolation, and (c) the cost of the plant.

We shall address the first two areas in the following, attempting to define basic requirements to be met by horizontal seismic isolation when applied to a nuclear plant.

A third point will be addressed supplementarily, namely the feasibility of requiring from the seismic isolation design that it be adaptable to new seismic conditions.

2. Licensibility of an Isolated Nuclear Power Plant

A nuclear power plant design is checked according to very detailed licensibility procedures which most severely control design and analysis methods, design details, material specifications and other related items.

We consider as an imperative requirement for obtaining approval by the authorities that the whole new structural system of the isolated plant fits in the existing codes and that it be possible to predict its behavior by means of the established procedures. The very adoption of the seismic isolation concept itself representing a significant licensing issue, any other major deviation from current methodologies and currently approved practices should be avoided in using this new concept, as much as possible, so as not to further complicate and delay the licensing process.

In addition, it is again imperative, in order to be given approval by the authorities, that the seismic isolation design meet basic requirements related to the performance characteristics and the material properties of the isolation system itself.

In the following, we shall address successively the requirements related to the predictability of behavior of the isolated structure and to the performance of the isolation system itself.

2.1 Predictability of Behavior

To fit in the existing codes, the elements of the seismic isolation of the nuclear power plant must have the following properties:

2.1.1 First Property

The vertical (axial) stiffness of the elements of the seismic isolation and their spacing as well as the vertical (flexural) and the horizontal (axial) stiffness of the foundation mat must be such that, as far as vertical loads are concerned, the use of isolation does not alter the character of the soil/structure interaction.

In that manner, the established load input and the procedures to evaluate the soil reactions can be used. To achieve this, the following requirements have to be met:

(a) the vertical axial stiffness of the load transferring isolation elements has to be maximal;

(b) the softer the soil, the greater the horizontal axial stiffness of the foundation mat has to be;

(c) the softer the soil and the stiffer the structure, the smaller the spacing between isolation elements and the greater the vertical (flexural) stiffness of the foundation mat have to be.

In that way, as far as vertical stiffness is concerned, the isolation elements , the foundation mat and the soil form a system with the same characteristics as the soil of an unisolated nuclear power plant. The vertical deformations of the soil and of the base of the structure -- i.e., above the isolation level -- resulting from the dead loads will substantially be the same as those in an unisolated plant.

The above requirements (a) and in part (c) concern properties of the method of seismic isolation applied.

2.1.2 Second Property

Each isolation bearing element must be able to transfer highly variable vertical loads.

In the structural analysis of a nuclear power plant supported on the ground in the classical way -- i.e., without seismic isolation -- we assume that the soil reacts with forces whose extreme distributions on the surface of the foundation result in a large variety of patterns according to the design of the whole plant, and more specifically to its foundation structural system, and to its size and loading. This variation in the force distribution at the surface of the foundation, which of course changes with the time, is mainly the result of two factors. On the one hand, it results from the statistical character of the properties of the ground and on the other hand, of the inadequacy of the existing methods of analysis to predict accurately these forces. In the case of (in part) uniformly distributed vertical loads -- e.g., (in part) uniformly distributed dead loads of the nuclear buildings -- the relative variety of load distributions is limited to a minimum. In this case, the most economic design for the foundation of the plant is that which provides a distribution of the soil reactions similar to that of the vertical loads, as far as possible. If horizontal loads of great intensity are added to the vertical loads -- e.g., the horizontal loads resulting from intensive earthquakes -- , the behavior of the materials of the structural system as well as of the ground becomes quite nonlinear. The knowledge of the properties of the materials is more questionable and the methods of analysis and design less reliable which has as a result that more extreme patterns of distribution of the forces have to be considered. This means that still heavier loads to be transferred by each definite area of the foundation surface must be assumed and this is accounted for in the design of the structural system of the plant and the verification of the soil strength.

If the plant is seismically isolated, the elements which constitute the seismic isolation system are located between the soil and the structure on the surface of a foundation mat. To be efficient the isolation system has to reduce the horizontal seismic forces as well as the overturning moments by a factor of from 10 to 20 in the case of a severe real earthquake. As a consequence, (1) the variation of soil loading due to the earthquake is minimized also since the loading of the soil during the earthquake is not much different from the loading by the vertical loads acting alone; (2) pronounced nonlinearities in the behavior of the material disappear; and (3) the methods of analysis remain reliable. It is expected that this will be felt as an advantage by the authorities and that in the future it will not be necessary for an isolated structure that the same extreme distribution patterns be taken into account. But since the establishing of criteria for a new structural design requires a very long procedure, an isolation system in order to be licensed today must be able to transfer the highly variable forces of the most extreme cases of distribution described in the previous paragraph exactly as a classical structure.

2.1.3 Third Property

For the same reasons as above, the elements of the seismic isolation system which are in fact an extension or projection of the soil must not be influenced by a possible partial loss of contact with the structure.

Of course a main reason for such a loss of contact is the strong seismic overturning moments and, as they are minimized when seismic isolation is applied, one could think that the provision of such a property is superfluous and can be overlooked. But in order to facilitate the licensing process, it is not advisable that this favorable feature be taken into account. Consequently, the isolation system must be able to function without problems even if a partial loss of contact on some foundation areas occurs. Besides, there is always the possibility of differential settlements due to variations in actual loads or soil properties or to seismic surface waves, and the ability to handle these cases of loss of contact can prove quite useful.

2.1.4 Fourth Property

Currently, the existing seismic input is developed for substantially force-dependent structural systems and its suitability for the prediction of strongly displacement-dependent systems may be questionable. Most likely it will need to be thoroughly verified and even be replaced by new seismic input. Again, this need may result in excessive delays in obtaining licensing acceptance.

To avoid or minimize this delay, the displacement of the top of the isolation elements as a whole in relation to their base support must be substantially force-dependent -- i.e., base shear-dependent --. Then the well documented existing seismic input produced currently and used for the force-dependent oscillations of classical structures and the established associated analytical methods can be used also for the prediction of the seismic response (displacements-forces) of the isolated structure.

The damping (viscous coulomb) characteristics of the isolation system must be of the same order as those usually encountered in the common classical plants when loaded in the elastic range of stresses so that the reliability of the seismic response predicted be maximal.

2.1.5 Fifth Property

In the event of an earthquake, the isolated part of the structure translates in the horizontal direction in relation to the foundation. Synchronously the base of the translating structure deforms in the vertical direction as well as the soil surface, i.e., the foundation mat. The vertical inclinations (or rotations about horizontal axis) of each of the vertically coupled areas of the translating base and the foundation mat do not necessarily coincide and therefore the isolation system must provide for unrestrained accommodation of the inclination differences between the base of the superstructure and the foundation mat.

A related problem also arises under normal loading due to (a) the difference between the vertical stiffness of the superstructure and that of the foundation and (b) the different distributions of the superstructure loads and of the pressure exerted on the soil surface by the foundation mat. In view of this also, the above provision is indispensable.

2.2 Isolation Basic Requirements

2.2.1 Performance of the Isolation System

The structure must remain stable even when experiencing the large expected relative displacements associated to high efficiency, that is, the isolation system support must be able to safely transfer the vertical loads despite the horizontal relative displacements without any damage (buckling), and substantially return to its original position after a severe earthquake, that is, the system must include strong centering forces. The remaining displacements have to be non-cumulative to prevent a one-directional "walk" of the structure in the case of a sequence of earthquakes and strong aftershocks (see also ref. 16, p. 44).

The elements of the isolation system must remain substantially unstressed by contractions and expansions of the superstructure and the foundation due to thermal expansion/contraction or similar effects. This is necessary to protect them from a continuous stressing for the whole life of the structure, which would have a degrading influence on their long-term function.

2.2.2 Properties of Materials

It is necessary to know perfectly the properties of the materials and components of the isolation system in order to be able to provide a reliable prediction of the behavior of the isolated structure.

Therefore, the materials used for the construction of the components of the system have to be well known and must have been used repeatedly, and for a long period of time, in similar environments in elements performing similar functions. That is, one must accumulate much material and literature related to the analysis and the testing of long-term physical, chemical and mechanical properties of materials proposed for use.

The successful and reliable performance of the function required from each element of the system for the whole time period of its use must be guaranteed. These elements must have performed successfully the same function for long in major structures located in similar environments. Among the mechanical properties of each component, those which are essential to the isolation system must be easily verifiable, both before and during their use in the structure. This verification must not weaken their performance ability and should be relatively easy to do.

2.2.3 In-Situ Verification of the Mechanical Properties of the Whole System

In addition to the verification of the mechanical properties of its different elements, the system must be able to allow an easy and economic in-situ verification of the mechanical properties of the whole system of isolation. This should be possible at various stages of construction, especially when the main buildings have been constructed. Such an in-situ testing is considered indispensable, given the specificity of the nuclear power plant. Moreover, it must be possible to easily rectify any problems uncovered by testing the whole system.

2.2.4 Redundancy

The isolation system in an isolated plant plays a role of first importance regarding its safety since it supports the whole plant. In effect, we transfer attention, from a safety point of view, from the structure to the isolation system. In view of this, it must be so designed that there is a great plurality of isolation components, i.e., the transferring of all vertical and horizontal loads to the foundation must be distributed to a great number of components so that an unexpected failure of one or more of them (despite the checking of their properties before and during their use) does not influence the functioning of the whole isolation system.

3. Efficiency of Seismic Isolation in a Nuclear Power Plant

In view of

(a) the very special function performed by the nuclear power plants and the very special fuel they process,

(b) the public opinion concern about their safety and potential pollution of the environment resulting from failure,

(c) the frequent revision to higher levels of the seismicity of many sites (as a result of which, e.g., NRC suspended the operating license of a nuclear power plant built years earlier, causing the loss of billions of dollars and bringing again to a sharp focus the reliability of seismic input) and the uncertainty factor in the establishing of the relative seismic criteria,

(d) the high cost of a nuclear power plant, and

(e) the subsequent need for a standardized design, abolishing its dependence on the site seismicity and site different soil and geological conditions,

it is a basic requirement that high seismic input margins be provided to nuclear power plants.

We believe that an isolation system provides a reasonable efficiency if it can limit the seismic base shear to 10 % of the weight of the plant for all the recorded high intensity seismic accelerograms scaled to 1.50 g peak acceleration minimum.

4. Adaptability to New Seismic Conditions

Considering the constant evolution of improved seismic maps throughout the world (based on the accumulation of new seismic data), it can never be completely sure that a significant increase in the seismicity of an area will not occur.

In view of this, it is desirable that the isolation system be provided with the property of adaptability to new and greater seismic input.

5. Conclusion

This paper reviewed several properties that a realistic isolated nuclear power plant design should possess in order to obtain undelayed licences and to provide high seismic safety margins.

(1) Since the very adoption of the isolation design represents an important licensing issue, any other innovation with respect to the current licensing practices should be avoided. The new structural system should fit in the existing codes and its behavior should be predictable by means of the established analysis procedures.

(2) Since the incorporation of seismic isolation in a nuclear power plant involves a concentration of the safety concerns on the isolation system, the latter must comply with the most severe requirements with regard to the verification of its performance, the properties of its materials, etc.., The isolation system should ensure a perfect stability of the structure and its components.

(3) To satisfy the need for high seismic safety margins, an isolation system should not allow the seismic base shear to exceed 10 % of the weight of the structure for all recorded accelerograms scaled to 1.50 g peak acceleration minimum. This last property also makes possible the standardization of the nuclear power plants design.

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Application of Base Isolation to Nuclear Equipment

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The paper presents a base isolation concept and discusses its potential application to nuclear equipment subjected to high frequency (20-80Hz) loads which can result from Loss-Of-Coolant Accident (LOCA) and/or Safety/Relief Valve (S/RV) discharge events. The high frequency content of these loads has little effect on the structural integrity of most types of equipment. However, serious problems can be encountered in the operability of some electro-mechanical devices such as relays. Typical of this situation is control panels. Response spectra computed at component locations can exhibit high acceleration peaks in the high frequency range - often considerably above the levels for which the components can be qualified.

When equipment is subjected to high accelerations in the low frequency range, the equipment is usually designed in such a way that its fundamental frequency is well above the frequencies of the applied loads so that little or no amplification occurs in the equipment. This approach is typically used for equipment excited by seismic loads whose dominant frequencies occur in the 1 to 15 Hz range. When the fundamental frequency of the equipment is raised to 33 Hz or higher, it is considered rigid for seismic loads and no amplification is assumed to occur.

This approach may be used to reduce high accelerations occurring in equipment resulting from high frequency loads. However problems arise with local modes of vibration in the 20-100 Hz range. It may be neither practical nor economical to qualify such equipment by stiffening because of the difficulty in raising the frequencies of the local modes out of the frequency range of the applied loads.

In contrast, base isolation takes the opposite approach. It reduces the fundamental frequency of the equipment and eliminates the amplification effects due to high frequency loads. Qualification of the equipment is greatly simplified and is focused on the design of base isolation devices which optimally balance acceleration and displacement in the equipment under the required loading conditions.

Herein a typical control panel has been selected and is subjected to a LOCA load typical of an actual nuclear power plant. The panel was analyzed with and without base isolation devices for these loads. The results of the analysis demonstrate that a properly designed base isolation system effectively eliminates high accelerations due to the high frequency component of the load.

1 INTRODUCTION

This paper presents a base isolation concept and discusses its potential application to nuclear equipment subjected to loads in the high frequency range. High frequency loads can result from Loss-Of-Coolant Accident (LOCA) and/or Safety/Relief Valve (S/RV) discharge events.

It has been shown that the high frequency content of these loads has little effect on the structural integrity of most typical types of equipment. However, serious problems can be encountered in the operability of some attached devices such as relays and other electro-mechanical components. Typical of this situation is control panels. High frequency loads have little influence on the stress levels within the panels or the adequacy of the base anchorage. However, response spectra computed at component locations exhibit high acceleration peaks in the high frequency range - often considerably above the levels for which the components can be qualified.

When equipment is subjected to high accelerations in the low frequency range, the equipment is usually designed in such a way that its fundamental frequency is well above the frequencies of the loads and little or no amplification occurs in the equipment. This approach is typically used for equipment excited by seismic loads whose dominant frequencies exist between 1 and 15 Hz. When the fundamental frequency of the equipment is raised to 33 Hz or higher, it is considered rigid for seismic loads and no amplification is assumed to occur.

This approach has also been used to reduce high accelerations occurring in equipment resulting from high frequency related loads. However a problem may arise with some types of equipment with local modes of vibration in the 20-100 Hz range. It may be neither practical nor economical to qualify such equipment by stiffening because the fundamental frequency has to be raised to above 100 Hz and all significant local modes have to be eliminated.

This is particularly true in retrofit situations where equipment originally designed for seismic only has to be requalified for high frequency loads. Raising the local mode frequencies to above 100 Hz can require an increase in stiffness of a factor of 10 or more. For equipment already built (and installed) this can be a formidable task - both practically and economically.

In contrast, base isolation takes the opposite approach and reduces the fundamental frequency of the equipment. The amplification effects of local modes on the equipment response due to high frequency loads is eliminated. Qualification of the equipment is greatly simplified and is focused on the design of base isolation devices which optimally balance acceleration and displacement in the equipment under the required loading conditions.

2 BASE ISOLATION

A base isolation system isolates equipment or a structure from harmful vibration. The most common use of base isolation is seen in heavy machinery or equipment whose vibration and noise are isolated from the floor at the mounting base. Another use exists in buildings which are isolated from earthquake ground motion. This paper studies the latter application and extends it to nuclear equipment. A significant amount of research has been performed on base isolation systems for buildings in both the U.S. and New Zealand [1][2]. Base isolation devices have been incorporated in the design of several structures in earthquake regions throughout the world and is now a viable concept to be considered by designers.

An effective earthquake isolation system for buildings must satisfy the following requirements:

(1) Reduction of the fundamental natural frequency of the building well below the frequencies where peak seismic accelerations occur.

(2) Maintenance of an acceptable level of displacement in the building when it is softened by the use of base isolation.

The first requirement determines how much amplification (or reduction) will be achieved in the building response against earthquake ground motion. When this requirement is met, the frequency of the base isolation system governs the building response and the original frequencies of the building itself produce negligible response. The second requirement, determines how soft the base isolation system can be before the displacement becomes unacceptable. It is apparent that these two requirements conflict with each other. Therefore the earthquake isolation system for a building must be designed such that the requirements for acceleration and displacement are satisfied in an optimum manner.

Nuclear equipment generally has the following characteristics, compared with those of a building:

(1) Its fundamental frequency is much higher.

(2) It weighs much less.

(3) It may experience high frequency related loads in addition to seismic loads.

It is not difficult to design a base isolation system which significantly reduces the equipment response to high frequency related loads such as LOCA and S/RV discharge loads. However more thought is required to design a system for effective isolation of seismic loads as well.

It is worth discussing at this stage the mechanism by which base isolation is effective. Contrary to a common belief, the high frequency modes of vibration of a system are not eliminated by the addition of base isolation devices - in fact, the natural frequencies remain essentially unchanged (except for the addition of "rigid body" modes). However, the corresponding mode shapes have superimposed on them a rigid body component (corresponding to base movement) which serves to substantially reduce the corresponding modal participation factor. As a result, these modes contribute little to the total response at points within the system - the high frequency motion is effectively "filtered" out.

Figure 1 shows a typical base isolation device applicable to nuclear control panels. Multilayer rubber bearings and steel sheets are designed such that the device provides the required stiffness in the horizontal direction and yet maintains a high stiffness in the vertical direction. The latter is required to prevent the introduction of any undesirable rocking or vertical modes of vibration.

3 APPLICATION

3.1 Control Panel

Control panels in nuclear plants appear to have the best potential for the use of base isolation. Therefore these panels are studied in more detail to demonstrate the effectiveness of base isolation for high frequency loads.

A typical control panel is a cabinet type of structure which is constructed using structural members such as angles and channels for its framing and plate steel for its paneling. A number of safety related devices will be mounted on the panel. Because of the structural characteristics of the panel there will be a number of local plate modes below 60 Hz. In addition, there may be some global modes of vibration in this range depending on the stiffness of the base/anchorage system. LOCA, S/RV discharge and seismic loads are likely to be the major loads. High frequency loads which could result from LOCA and S/RV discharge events may have peak accelerations in the 20-100 Hz range. These loads will excite both local and global modes of the panel and the devices mounted on the panel may experience high and possibly unacceptable accelerations.

3.2 Stiffening Approach

When a control panel experiences high acceleration in the 20-100 Hz frequency range, the conventional solution is to stiffen the panel and raise its fundamental frequency well above the exciting frequencies. If this approach is used then all the significant local modes of the panel have to be suppressed by stiffeners and the fundamental frequency has to be raised above 100 Hz. The following difficulties arise with this stiffening approach:

(1) All significant local modes of the panel which are sensitive to local structural characteristics will have to be identified.

(2) Extensive stiffening of the panel will be required to suppress the local modes.

Additionally, each panel may require a unique stiffening scheme because local modes of the panels are sensitive to local structural characteristics. When there are a number of panels which require stiffening, this approach will become very costly.

3.3 Base Isolation Approach

General

The difficulties associated with stiffening a control panel appear to be resolved by using base isolation. When a base isolation system is used, the panel response to high frequency related loads will be governed by the stiffness of the base isolation system rather than local structural characteristics of the panel. As a result, it is not difficult to identify panels which have similar dynamic response to the above loads. The parameters which determine the panel response will be the global stiffness and mass distribution in a panel and the base isolation stiffness.

This ease of the identification will result in the development of a few standardized base isolation devices and installation details which can be used for the qualification of all control panels of interest.

Example

Figure 2 gives a typical control panel chosen to study the effectiveness of base isolation for the above loads.

The panel is 96" wide, 48" deep and 84" high. It weighs approximately 3,000 pounds. Two finite element models were developed for this study:

(1) As-built model without base isolation.

(2) Model with the base isolation system which reduces the fundamental frequency of the panel to approximately 5 Hz.

The loading used is an acceleration time history, at the panel base, resulting from a LOCA event. The input time history was obtained at the floor where the panel is located. This example considered only the horizontal acceleration because in most cases the vertical acceleration induces much smaller response in the control panel.

Figure 3 shows the floor response spectra generated from the LOCA acceleration time history. This spectrum is presented to show the distinctive characteristics of the LOCA load. The LOCA floor response spectrum shows two dominant peak accelerations at about 25 and 75 Hz which could excite local as well as global modes of the panel.

The analysis was performed to obtain the response spectra (at two locations in the panel) which a device mounted on the panel could experience. When the panel response was computed, three sets of damping (2%, 5% and 10%) were used for the base isolation related modes and one set (2%) was maintained for the remaining modes.

The analysis consisted of:

(1) Mode/frequency analysis of the panel.

(2) Time history analysis of the panel using the modal superposition method.

(3) Response spectrum generation for devices on the panel.

Table 1 provides the first twenty natural frequencies and associated modal participation factors calculated for each model. Only the factors normal to the front of the panel, are presented because the greatest amplification occurred in that direction when subjected to the LOCA load. A number of local plate modes exist which could be significantly excited by the LOCA load. In the model where the base isolation system is incorporated, the results in Table 1 indicate that the local plate modes become insignificant and consequently the global base isolation related modes will govern the panel response. Note that in the base isolated panel the modal participation factors of all the higher modes are substantially less than the model with no base isolation

The acceleration response spectra resulting from the LOCA load is given in Figures 4 and 5. These response spectra are envelopes of those obtained at two locations on the panel.

As anticipated from the results in Table 1, the base isolation system drastically reduced the effects of the local plate modes on the panel response. All the peak accelerations above 30 Hz observed in Figure 4 were effectively eliminated by the use of the base isolation system as seen in Figure 5.

The maximum acceleration of the panel response spectrum with and without the base isolation is 0.9g and 84g respectively. The displacement obtained for both models is negligible under the LOCA loading considered.

A survey of the literature indicates that the use of 5% damping and even higher is easily justified for the base isolation devices being considered. The panel displacement decreases as the damping value for the devices increases and will not be more than one-half inch for all the loads considered - LOCA, S/RV, and seismic.

4 CONCLUSIONS

This paper has presented the application of base isolation to nuclear equipment subjected to vibratory loads with acceleration peaks in the 20-80 Hz frequency range.

The results of the study demonstrate that the base isolation system effectively eliminates high accelerations in the equipment, resulting from the high frequency component of these loads.

In contrast to the conventional stiffening approach used for equipment qualification, base isolation reduces the fundamental frequency of the equipment and substantially reduces the effect of the local modes on the equipment response to high frequency loads. The equipment response is governed by the global characteristics of the equipment and the base isolation stiffness rather than the local characteristics. Therefore the use of base isolation will reduce the effort required for the qualification of the equipment.

The use of base isolation appears to be particularly suitable for existing equipment that requires requalification because of newly identified, high frequency related loads. The use of base isolation will require substantially less structural modifications to the equipment than the conventional stiffening approach. Base isolation is even more cost effective when it is considered at the design stage of equipment.

Computech has designed several base isolation devices. The devices are relatively inexpensive and easy to install in both new and existing equipment. It is our opinion that the use of a base isolation system offers a cost-effective solution to the problems of nuclear equipment subjected to high frequency loads (LOCA and S/RV) as well as to seismic loads. Base isolation should therefore be seriously considered as a potential solution to the requalification of any existing nuclear equipment as well as its possible use for new equipment.

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TABLE 1

NATURAL FREQUENCIES AND MODAL PARTICIPATION FACTORS OF CONTROL PANEL

MODE NO.	CONTROL PANEL MODEL: NO BASE ISOLATION		CONTROL PANEL MODEL: BASE ISOLATED PANEL	
	FREQUENCY (Hz)	MPF	FREQUENCY (Hz)	MPF
1	37.4	0.708	4.9	0.088
2	47.7	-1.063	5.0	2.007
3	48.6	0.314	5.2	-0.057
4	55.5	0.157	18.6	-0.003
5	56.7	0.805	25.1	0.011
6	61.1	-0.330	39.1	0.006
7	62.7	-0.039	41.7	0.009
8	65.0	0.067	49.4	-0.002
9	68.6	-0.415	53.1	-0.008
10	69.5	0.004	57.0	0.010
11	71.7	-0.059	57.1	0.002
12	75.2	-0.348	62.1	-0.004
13	78.6	0.106	64.2	0.010
14	80.2	0.046	64.8	0.004
15	88.0	0.095	68.8	0.005
16	92.2	0.165	69.3	0.003
17	94.2	0.015	71.4	0.002
18	96.3	0.117	75.6	0.001
19	100.6	0.117	78.1	-0.005
20	101.3	-0.090	80.0	-0.002

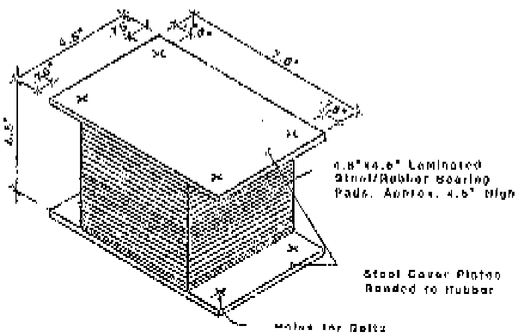

FIGURE 1
BASE ISOLATION DEVICE

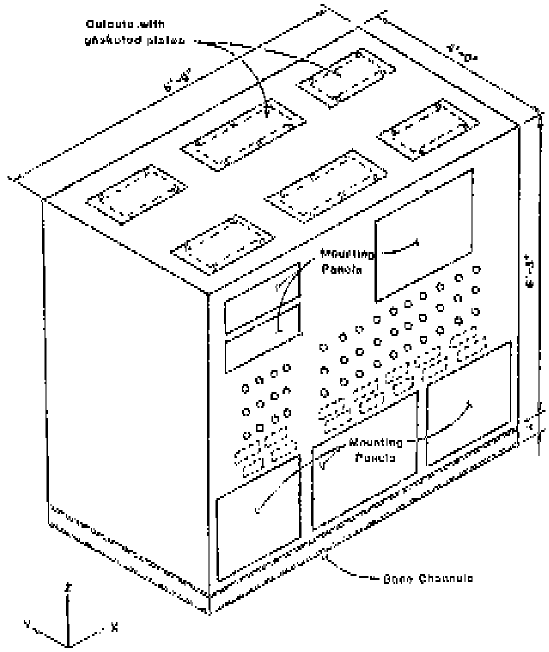

FIGURE 2
TYPICAL CONTROL PANEL

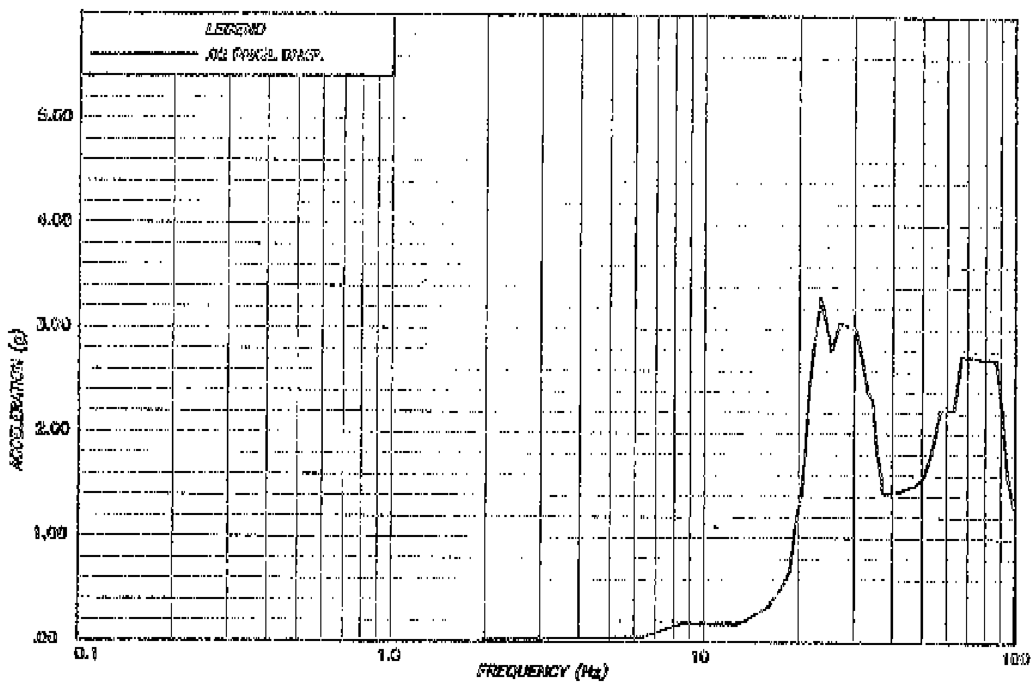

FIGURE 3
FLOOR RESPONSE SPECTRUM DUE TO LOCA HORIZONTAL

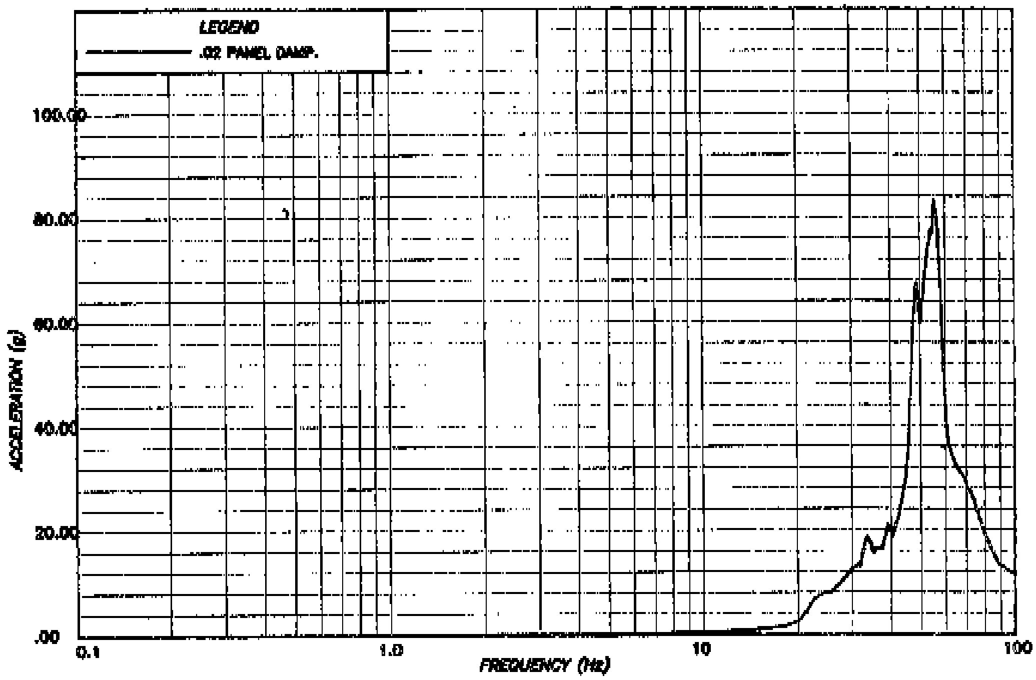

FIGURE 4
IN-PANEL RESPONSE SPECTRUM DUE TO LOCA
(WITHOUT BASE ISOLATION)

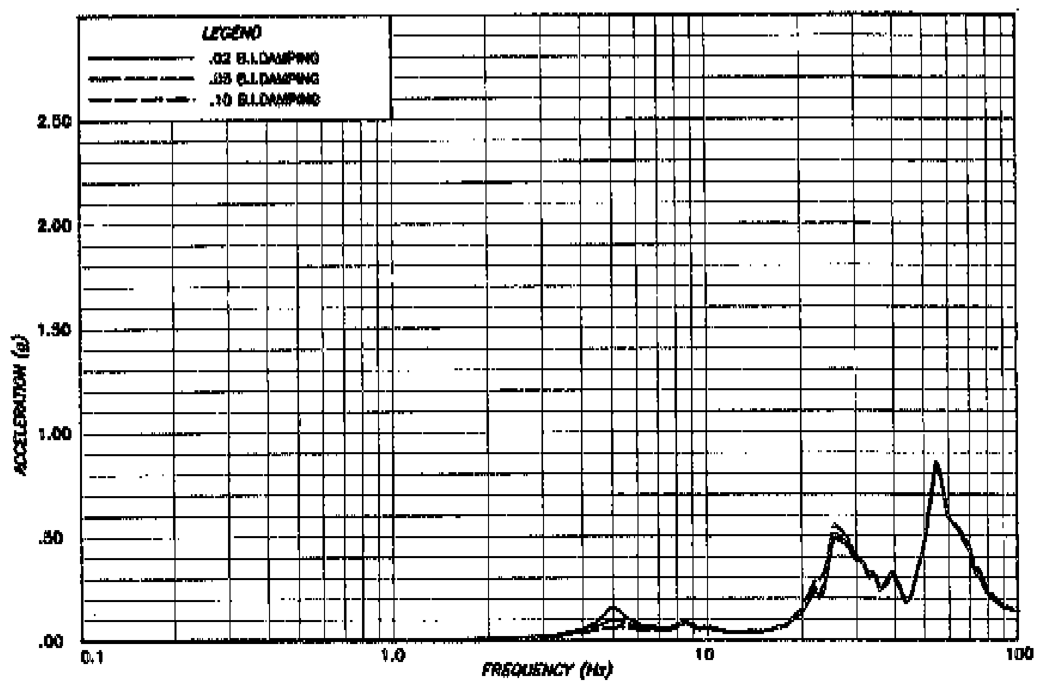

FIGURE 5
IN-PANEL RESPONSE SPECTRA DUE TO LOCA
(WITH 5H_2 BASE ISOLATION)

Response of a Nuclear Power Plant on Aseismic Bearings from Horizontally Propagating Waves

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SUMMARY

The nuclear island of Koeberg with a large basemat, a nonlinear base isolation effective in the horizontal direction only, founded on rock, is analysed for horizontally propagating waves. The horizontal and vertical components of the control motion are either associated with inclined P- and SV-waves or with a combination of a P-wave and the first two Rayleigh modes. The response is compared to that resulting from vertically incident waves. Although the analysis of soil-structure interaction uses the principle of superposition by performing first the free-field analysis and then the actual interaction calculation, the structure in the latter can exhibit nonlinear behaviour. The governing equation of motion in the time domain contains convolution integrals, which can be removed by assuming a frequency-independent dynamic-stiffness matrix of the unbounded soil.

It is demonstrated that the aseismic bearings result in a significant isolation in the horizontal direction for vertically incident waves. The vertical excitation affects considerably the response in the horizontal direction in the frequency range of the vertical modes for the nonlinear base isolation.

When compared to vertical incidence, the horizontal component of the propagating wave is filtered significantly because of the size of the raft. However, this results in only a slightly smaller structural response, since the aseismic bearings already represent a filter. The vertical response is hardly affected by horizontally propagating waves. An additional rocking component arises, generated by the horizontally propagating vertical component. As the aseismic bearings do not isolate against this rocking component, the corresponding horizontal response bears comparison with that of a conventional structure. For the base-isolated structure, the rocking input leads to a considerable increase of the horizontal response. The actual design incorporating other loading cases is affected much less. Horizontally travelling waves hardly modify the maximum horizontal displacements and the amount and duration of sliding. A larger variation of the vertical Neoprene forces occurs, but no uplift arises.

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1. INTRODUCTION

Structures are routinely designed for vertically incident body waves. Assuming the wave pattern to consist of inclined body waves and of surface waves, the waves propagate horizontally across the site. The seismic input of a surface structure will in this case consist of (reduced) translational components and additional rotational components not present for vertical incidence. The effects of horizontally propagating waves on the seismic response of conventional structures of common dimensions have been evaluated systematically (Ref.1). Only for extreme assumptions of the wave pattern are these effects of importance, increasing generally for larger dimensions of the basemat. Structures which are isolated at the base in the horizontal direction only are examined in Reference 2. As the rocking component of the seismic input is not isolated in this concept, the effects of horizontally propagating waves are in general larger than in conventional structures. Along with the structural configuration, the properties of the site are of importance to evaluate wave-passage effects. For soft sites with large material damping, the surface waves attenuate significantly (Refs. 3,4). Thus in general, only rock sites have to be examined for horizontally propagating surface waves.

As the nuclear island of the Koeberg Nuclear Power Plant in South Africa has a large basemat, a horizontal isolation mechanism and is founded on rock, horizontally propagating waves are investigated. A schematic section passing through the nuclear island is shown in Figure 1. The isolation mechanism using aseismic bearings is well documented in the literature (e.g. Ref. 5,6,7). All safety-related structures of the nuclear island are founded on a common upper raft. The aseismic bearings are located between the lower and upper rafts. On top of concrete pedestals resting on the lower raft, Neoprene pads, which are flexible in the horizontal direction only, are placed. In addition, friction plates are used to limit the horizontal force transmitted to the structure. This results in a nonlinear isolation mechanism acting in the horizontal direction only. The lower raft rests on a layer of soil-cement which is on top of a layered rock site.

The nuclear island of Koeberg is analysed for horizontally propagating waves in Reference 8 , whereby a constant apparent velocity, selected conservatively, is assumed. A more realistic seismic input compatible with the equations of elastodynamics applied to the layered site is used in this article. For the sake of conciseness, the results of only one set of parameters (which do not necessarily govern the design) are presented. Horizontally propagating SH- and Love-waves are not analysed, as the horizontal isolation mechanism also acts for the additional torsional seismic input generated by these waves. A two-dimensional model is adopted for the analysis of the soil-structure interaction caused by SV-, P- and Rayleigh waves.

2. MODELLING OF STRUCTURE AND SOIL

The dynamic model of the nuclear island of Koeberg is shown schematically in Figure 2. A simple model is deliberately selected which, however, can represent the characteristic effects arising from horizontally propagating waves. For this purpose only three buildings are included in the 2-dimensional model. The structures are modelled with beams and lumped masses. The upper raft consists of three rigid sections connected by two flexible parts. The lower raft is assumed to be flexible over the whole length. Both rafts are rigid in the horizontal direction. Each row of the aseismic bearings in the third dimension is modelled as a discrete element representing the horizontal and vertical flexibilities of the Neoprene pads and the friction plates. A total of 25 elements arise, which are not equally spaced. The pads are designed to exhibit a horizontal stiffness which results in a horizontal frequency = 0.9 Hz of the total system, assuming the upper raft and the structures to be rigid and the lower raft fixed. The coefficient of friction equals 0.15. The lower raft is founded on the layer of soil cement of 6 m thickness resting on bedrock. The latter is modelled with 10 homogeneous layers which rest on a homogeneous halfspace (-120 m). The material properties of the site are specified in Table I. The shear-wave velocity increases with depth, while the damping ratio of the bedrock is constant.

When analysing nonlinear soil-structure interaction by the substructure method, the nonlinearity has to be restricted to the structure. The unbounded soil must remain linear. The governing equations of motion are formulated using the nuclear island on nonlinear aseismic bearings shown in Figure 1 for illustration.

The interface of the lower raft and the soil is discretised with 25 substrips analogously to the elements modelling the aseismic bearings. Taking the constraints imposed by the rigid connections into account, the total dynamic model has 94 dynamic degrees of freedom.

The dynamic-stiffness matrix of the soil is of order 50 x 50. For a constant surface traction acting on each substrip, the dynamic-flexibility influence functions can be calculated by integrating for a specific frequency in the wave-number domain, which then leads to the dynamic-stiffness matrix (Refs. 9, 10). To examine the frequency dependency, the dynamic-stiffness coefficients in the vertical direction and for rocking of the total lower raft of length 134.4 m, assumed to be rigid and massless, are evaluated. They are plotted in Figure 3 (nondimensionalised by the corresponding value at 9 Hz, treating the real and imaginary parts separately). Although the site is layered, the dependency on frequency in the range of interest is not overwhelming. The frequency selected for calculating the dynamic-stiffness matrix equals 9 Hz (As a check, other frequencies were also selected. The differences in the dynamic response were found to be defensible).

3. FREE-FIELD RESPONSE

As the structure-soil interface is selected at the top of the soil cement (+ 6 m), the free-field soil profile of the site consists of the layer of soil cement of 6 m thickness resting on bedrock.

To determine the free-field response of a site, the control motion, the control point in which this motion is assumed to act, and the wave pattern have to be specified. This is illustrated for the Koeberg site in Figure 4. The horizontal and vertical components of the control motion consist of artificial 30-s acceleration time-histories, the response spectra of which follow the US-NRC Regulatory Guide 1.60, normalized to 0.30 g and 0.20 g, respectively. The horizontal-response spectrum is shown for a damping ratio of 5% as a solid line in Figure 5. The control point is located at the top of the bedrock (0 m), without the presence of the soil-cement. With respect to the total site (including the layer of soil-cement), the control motion thus acts at a fictitious rock outcrop. The free-field motion at the top of the soil-cement (+ 6 m) is determined for two different body-wave assumptions. The control motion is first deconvoluted to the halfspace (- 120 m), resulting in the amplitudes of the incident waves and then convoluted to the top of the soil cement. For vertically incident body waves the corresponding horizontal-response spectrum is plotted in Figure 5. For inclined body waves the angles of incidence of the P- and SV-waves ψ_P and ψ_{SV} in the halfspace are selected as 5° and 30°, resulting in apparent velocities of 7627 m/s and 5068 m/s, respectively. The corresponding amplitudes A_P and A_{SV} of the incident waves in the halfspace depend on both the horizontal and vertical components of the control motion. The horizontal-response spectrum for inclined body waves hardly differs from that for vertical incidence (Figure 5). This also applies to the vertical component (not shown). Besides assuming these two wave patterns, denoted as "vertical incidence" and "inclined waves" further on, a wave train consisting mainly of Rayleigh waves that results in the same motion at the top of the soil cement as the vertically incident body waves is also investigated, as described below. For a layered system the apparent velocity depends on the frequency. For the first two R-modes, the dispersion curves are shown in Figure 6. For frequencies above 9.1 Hz, where the second R-mode starts, the horizontal and vertical components of the motion at the top of the soil-cement can be associated with the two R-modes. For frequencies below 9.1 Hz, a P-wave with ψ_P= 5° (apparent velocity = 7627 m/s) is used in addition to the first R-mode. Details are discussed in Reference 4. This wave pattern is denoted as "R- and P-waves".

The analytical procedure is described in References 3 and 11.

4. SINGLE REACTOR BUILDING

For the sake of comparison, the single reactor building of the Koeberg nuclear-power plant, assumed to be founded on its own basemat on top of the soil-cement with Neoprene pads (without friction plates), is also investigated. To evaluate the influence of the aseismic bearings on the response, the same structure, but without the base isolation, is also examined.

Introducing the base isolation reduces the frequency of the first mode of the soil-structure system from 3.3 Hz to 0.9 Hz and that of the second from 6.6 Hz to 4.3 Hz. The horizontal component of the seismic input excites mainly the first mode for the isolated system while both modes contribute to the response for the rocking component (arising from horizontally propagating waves) (Ref.2).

The horizontal and vertical total accelerations in three points are specified for the reactor building with and without the base isolation in Table II. The favourable effect of the aseismic bearings on the horizontal response caused by vertically incident waves shown for comparison is clearly visible. The response caused only by the rocking seismic input motion of the R- and P-waves does not differ significantly for the two structures. This results in larger horizontally propagating wave effects on a percentage basis for the structure with the base isolation (taking the translational and rocking seismic input motions into account). The self-cancelling effect on the vertical response is small, as is visible in

Table II. The horizontal-response spectra are presented in Figure 7. For both structures the peaks at the frequencies of the first and second modes are clearly visible. The peak at 0.9 Hz of the base-isolated structure is not affected by horizontally propagating waves, while the second peak at 4.3 Hz is increased from 0.98 g to 2.13 g or by a factor 2.17 (top shield building). For the structure without base isolation the peak at 3.3 Hz is magnified from 24.8 g to 27.8 g or by a factor 1.12. As the seismic excitation forms only one of several load cases arising in the load combination governing design, the significance of these factors is limited (Ref. 2). Assuming e.g. that the contribution of the other load cases equals that of the seismic response of the structure without base isolation for vertical incidence (24.8 g) results in an increase of (24.8 + 27.8) / (24.8 + 24.8) = 1.06 for the structure without base isolation and of (24.8 + 2.13) / (24.8 + 0.98) = 1.04 for the base-isolated structure.

5. NUCLEAR ISLAND OF KOEBERG

The vibrational mode shapes of the linear system (omitting the friction plates) are useful when interpreting the dynamic behaviour. The first six modeshapes and the associated natural frequencies are shown in Figure 8. As expected, the structure with the upper raft moves (from a practical point of view) horizontally as a rigid body in the first mode.

To gain further insight into the different effects influencing the dynamic response, it is convenient to perform the analysis in two steps, the kinematic and inertial interactions.This decomposition remains computationally attractive, as the kinematic interaction part of the analysis is linear for this structural configuration.

The response spectra of the seismic input motion (resulting from the kinematic-interaction part of the analysis) are shown in Figure 9. The large dimension of the raft of the nuclear island leads to a stronger reduction of the horizontal component in the higher-frequency range than for the single reactor building shown for comparison (Figure 9a). Up to 10 Hz, the rocking input (multiplied by the same characteristic height = 30.8 m) is approximately the same for the three cases described in Figure 9b. Assuming the entire upper raft also to be rigid in the vertical direction results in a more pronounced diminution in the higher-frequency range.

The maximum total accelerations are shown in Table III for the nuclear island of Koeberg with Neoprene pads without and with friction plates for the various wave patterns. For vertical incidence the favourable influence of the Neoprene pads on the horizontal response leads to a small amplification with height. Adding friction plates reduces the horizontal accelerations even further. The differences in the horizontal response of the two reactor buildings for vertical incidence are small. They are caused by the second mode (among others) (Figure 8) which is excited by the vertical component of the seismic input. For the horizontally propagating R- and P-waves, the rotational seismic input results in an increase of the horizontal response throughout the structure. The differences for the reactor buildings discussed above are more pronounced. The inclined waves defined in Section 3 also increase the horizontal response compared to that for vertical incidence but, in general, less than for R- and P-waves.

In-structure response spectra for the nuclear island with Neoprene pads but without friction plates are presented in Figures 10 and 11 for the horizontal and vertical directions, respectively. The results for R- and P-waves are compared to those for vertical incidence. In the horizontal direction the peak at the frequency 0.89 Hz of the first mode is, as expected, dominant throughout the structures (Figure 10). The other peaks can also be associated with the frequencies of certain modes. The second mode, with a frequency 3.82 Hz (Figure 8), is easily detected at the top of the shield building and at the top of the internal structure. As expected from the mode shape, the peak is more pronounced for the horizontally propagating wave than for vertical incidence. The significant increase of the second peak at the top of the shield building also arises from the contribution of the third mode with a frequency 4.34 Hz. The fourth mode causes the peak at 7.15 Hz, clearly visible at the top of the internal structure. The peak for vertical incidence arises from the vertical seismic input. Modes 3, 5 and 6 are responsible for the peaks in the range between 4 and 10 Hz present for R- and P-waves at the top of the auxiliary building. In the vertical direction, horizontally propagating wave effects are small (Figure 11).

To study the influence of the vertical component of the seismic input motion on the horizontal response for vertically incident waves, the horizontal in-structure response spectrum on the upper raft is shown in Figure 12. As the upper raft of the symmetric structure is rigid horizontally, no such influence can exist in a linear system. Introducing friction plates and analysing for the horizontal component of the seismic input only, the favourable effect observed in the total horizontal acceleration in Table III is present in the response spectra throughout the frequency range (dashed versus dotted line in Figure 12). Adding the vertical component to the seismic motion, the favourable reduction of the peak at 0.89 Hz is unaffected (solid line). However, an additional peak arises at the frequency around 10 Hz, where the modes with large vertical-participation factors are located. This nonlinear coupling can be illustrated by examining the resultant of all horizontal Neoprene forces. Its time history is shown in Fig. 13 (horizontal and vertical components of seismic input for vertical incidence). The maximum permissible value of this resultant (calculated as the product of the vertical resultant of all Neoprene forces and of the friction coefficient) is also indicated. It depends on the vertical earthquake and would be constant for no vertical excitation. The high-frequency content of this maximum is imposed on the actual resultant horizontal Neoprene force during the time interval when all friction plates slide.

To evaluate the horizontally travelling wave effects on the in-structure response spectra for the nuclear island of Koeberg (with friction plates), the top of the shield building is selected. As can be seen from Figure 14, the spectra are similar to those shown for the linear system in Figure 10a. The increase for inclined waves (not shown) is somewhat smaller than for R- and P-waves. As discussed in connection with Figure 7b for the single reactor building, the impact of the large increase on a percentage basis is considerably reduced when load combinations are examined.

Finally, turning to the relative displacements, the time-history of the lower raft relative to the upper raft is plotted for vertical incidence in Figure 15. In addition the sliding displacements of the three Neoprene pads indicated by circles in Figure 2 are also shown. Because of the rocking motion of the reactor-building unit 2, the sliding starts at different times. The different sliding displacements result in residual horizontal Neoprene forces present after the seismic excitation has died out. These are plotted for the 25 Neoprene pads for R- and P-waves and for vertical incidence in Figure 16. No tension arises vertically in the Neoprene pads during the complete seismic excitation for all wave patterns. A partial loss of contact thus does not occur.

6. CONCLUDING REMARKS

1. The nuclear island of Koeberg with a large basemat, a nonlinear base isolation effective in the horizontal direction only, founded on rock, is analysed for inclined body waves and for a combination of surface and body waves. The response is compared to that resulting from vertically incident waves.

2. Although the analysis of soil-structure interaction uses the principle of superposition by performing first the free-field analysis and then the actual interaction calculation, the structure in the latter can exhibit nonlinear behaviour. The governing equation of motion in the time domain contains convolution integrals, which can be removed by assuming a frequency-independent dynamic-stiffness matrix of the unbounded soil.

3. The horizontal and vertical components of the control motion are either associated with inclined P- and SV-waves or with a combination of a P-wave and the first two Rayleigh modes.

4. It is demonstrated that the aseismic bearings result in a significant isolation in the horizontal direction for vertically incident waves. The vertical excitation affects considerably the response in the horizontal direction in the frequency range of the vertical modes for the nonlinear base isolation.

5. When compared to vertical incidence,
   a) the horizontal component of the propagating wave is filtered significantly because of the size of the raft. However, this results in only a slightly smaller structural response, since the aseismic bearings already represent a filter;
   b) the vertical response is hardly affected by horizontally propagating waves;
   c) an additional rocking component arises, generated by the horizontally propagating vertical component. As the aseismic bearings do not isolate against this rocking component, the corresponding horizontal response bears comparison with that of a conventional structure. For the base-isolated structure, the rocking input leads to a considerable increase of the horizontal response;
   d) horizontally travelling waves hardly modify the maximum horizontal displacements and the amount and duration of sliding. A larger variation of the vertical Neoprene forces occurs, but no uplift arises.

6. Although the ratio of the response for horizontally propagating waves and that for vertically incident waves is considerably larger for the base-isolated structure than for a conventional one, the actual design incorporating other loading cases is affected much less.

Acknowledgment

The authors are indebted to the Electricity Supply Commission of South Africa and the Atomic Energy Board for permission to publish results of the study on horizontally propagating waves for the NPP Koeberg.

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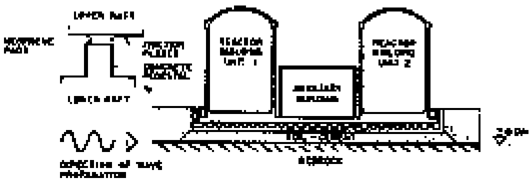


Fig. 1 Nuclear island on aseismic bearings of Koeberg Power Plant

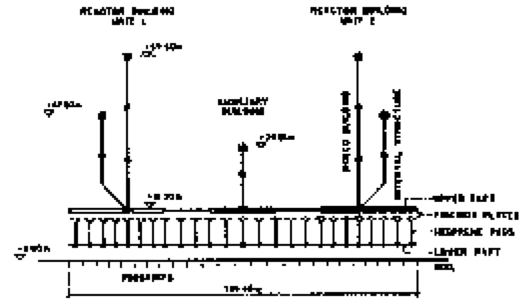


Fig. 2 Dynamic model of nuclear island of Koeberg

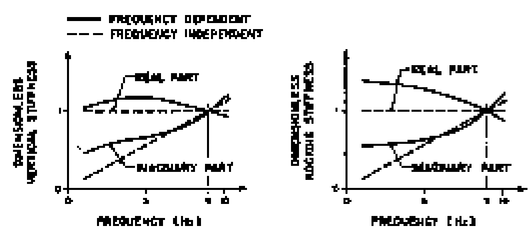


Fig. 3 Frequency dependency of dynamic stiffness of soil

DEPTH [m]	SHEAR MODULUS [GN/m²]	DENSITY [Mg/m³]	SHEAR WAVE VELOCITY [m/s]	POISSON'S RATIO	DAMPING RATIO
+ 4. / 0.	1.17	2.15	733.	0.30	0.05
0. / - 5.	3.00	2.60	1074.	0.35	0.05
- 5. / - 10.	3.40	2.60	1144.	0.35	0.05
- 10. / - 20.	5.00	2.60	1387.	0.40	0.05
- 20. / - 30.	8.00	2.60	1754.	0.40	0.05
- 30. / - 40.	12.00	2.60	2148.	0.40	0.05
- 40. / - 50.	18.00	2.60	2631.	0.35	0.03
- 50. / - 60.	27.00	2.60	3125.	0.30	0.03
- 60. / - 80.	31.00	2.60	3455.	0.29	0.03
- 80. / -100.	38.50	2.60	3844.	0.27	0.03
-100. / -120.	46.00	2.60	4206.	0.26	0.03
-120. / ∞	50.00	2.60	4385.	0.25	0.03

Tab. I Free-field properties

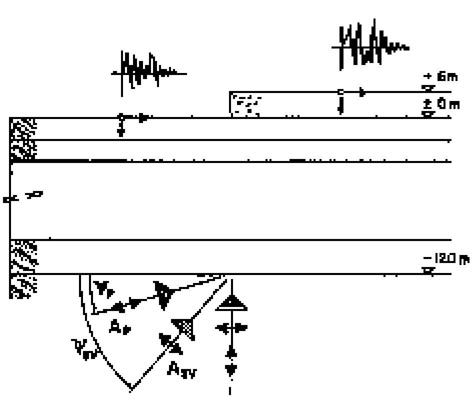


Fig. 4 Definition of seismic input

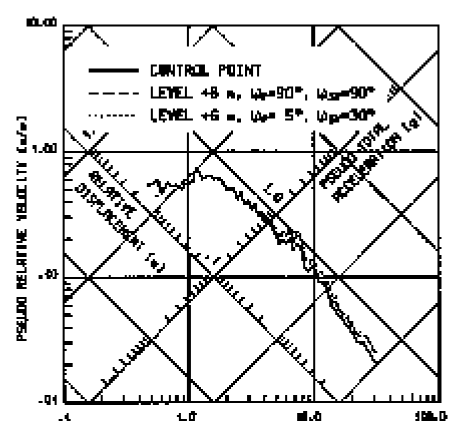


Fig. 5 Response spectra (5 % damping) of horizontal free-field motion

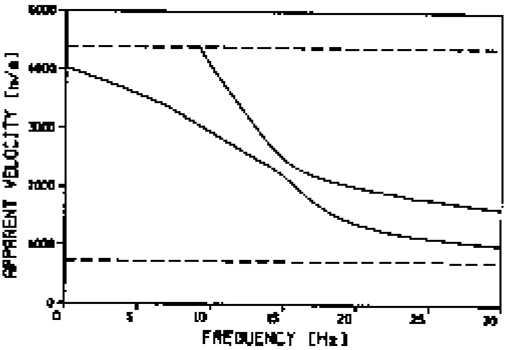
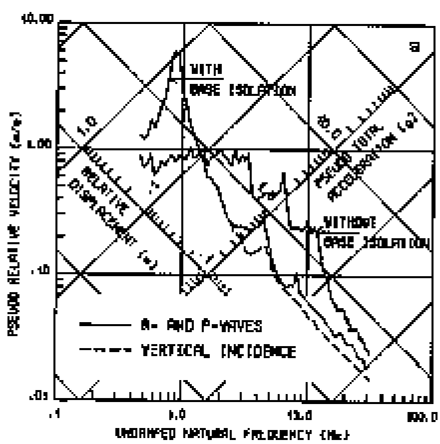


Fig. 6 Dispersion

	WITHOUT BASE ISOLATION			WITH BASE ISOLATION		
	VERTICAL INCIDENCE	R- AND P-WAVES		VERTICAL INCIDENCE	R- AND P-WAVES	
		ROCKING INPUT ONLY	COMPLETE INPUT		ROCKING INPUT ONLY	COMPLETE INPUT
HORIZONTAL:						
CENTRE BASEMAT	0.454	0.040	0.448	0.259	0.364	0.346
TOP SHIELD BUILDING	1.009	0.270	2.016	0.554	0.063	0.429
TOP INTERNAL STRUCTURE	0.182	0.152	1.294	0.318	0.182	0.398
VERTICAL:						
CENTRE BASEMAT	0.326		0.317	0.326		0.317
TOP SHIELD BUILDING	0.695		0.674	0.695		0.674
TOP INTERNAL STRUCTURE	0.376		0.364	0.376		0.364

Tab. II Maximum total acceleration [g], single reactor building



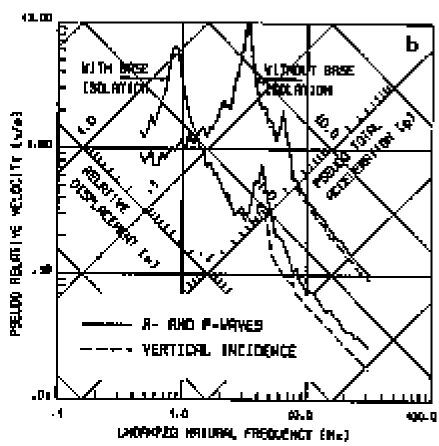


Fig. 7 Horizontal in-structure response spectra (2 % damping), single reactor building
a) centre basemat b) top shield building

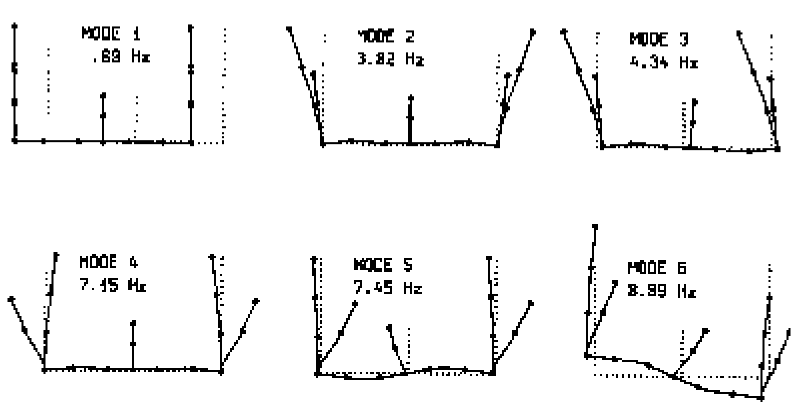


Fig. 8 Mode shapes and natural frequencies, nuclear island of Koeberg

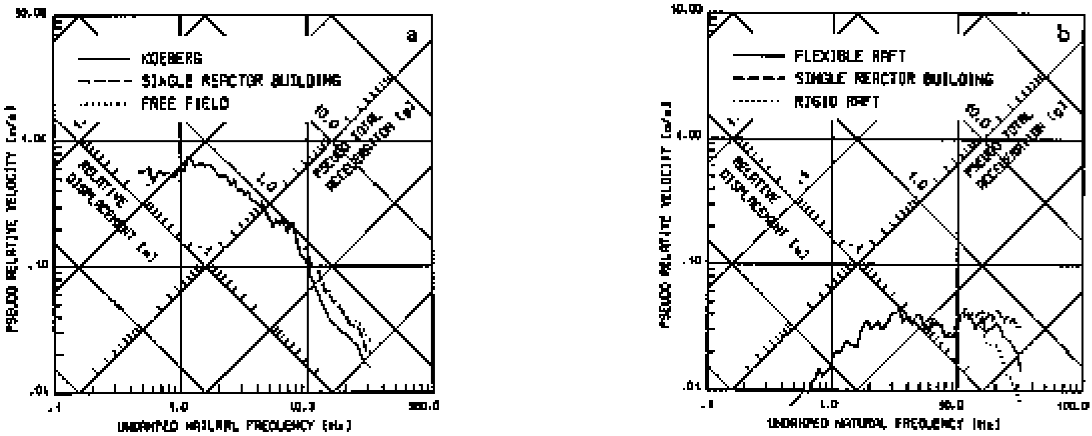
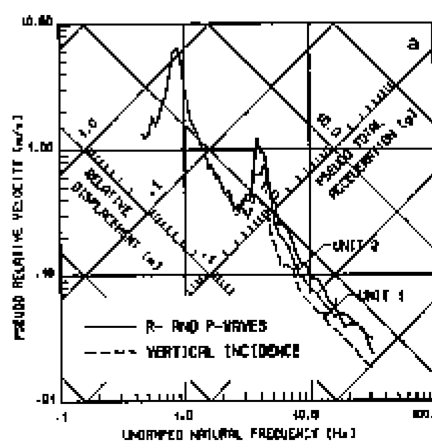
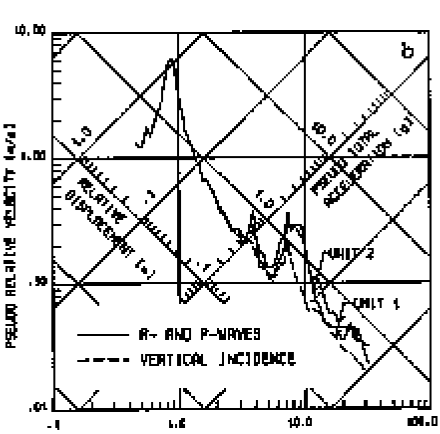


Fig. 9 Response spectra (5 % damping) of seismic-input motion,
nuclear island of Koeberg, R- and P-waves
a) horizontal b) rocking * 30.8 m, reactor building unit 1

	WITHOUT FRICTION PLATES		WITH FRICTION PLATES		
	VERTICAL INCIDENCE	R- AND P-WAVES	VERTICAL INCIDENCE	R- AND P-WAVES	INCLINED WAVES
HORIZONTAL:					
UPPER RAFT	0.304	0.338	0.184	0.181	0.186
TOP SHIELD BUILDING UNIT 1	0.388	0.528	0.292	0.425	0.337
TOP INTERNAL STRUCTURE UNIT 1	0.329	0.508	0.261	0.312	0.322
TOP SHIELD BUILDING UNIT 2	0.355	0.451	0.255	0.326	0.307
TOP INTERNAL STRUCTURE UNIT 2	[illegible]	0.397	0.275	0.304	0.257
TOP AUXILIARY BUILDING	0.310	0.386	0.252	0.289	0.275
VERTICAL:					
CENTRE UPPER RAFT UNIT 1	0.301	0.303	0.301	0.309	0.334
CENTRE UPPER RAFT UNIT 2	0.361	0.295	0.304	0.296	0.388
CENTRE UPPER RAFT AUXILIARY BUILDING	0.342	0.284	0.302	0.285	0.310

Tab. III Maximum total acceleration [g],
nuclear island of Koeberg





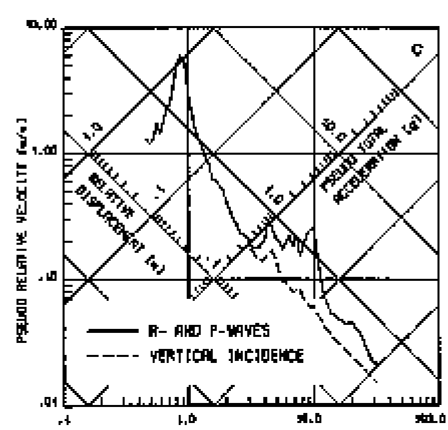


Fig. 10 Horizontal in-structure response spectra (2 % damping),
nuclear island of Koeberg without friction plates
a) top shield building b) top internal structure c) top auxiliary building

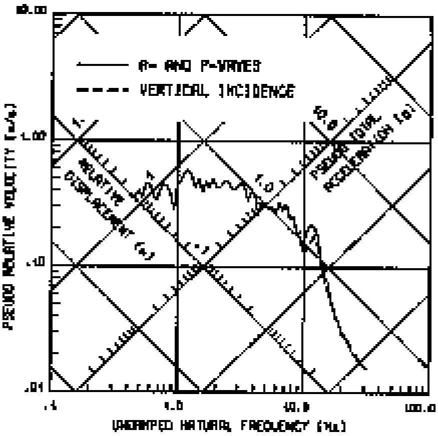


Fig. 11 Vertical in-structure response spectra (2 % damping), nuclear island of Koeberg without friction plates, base auxiliary building

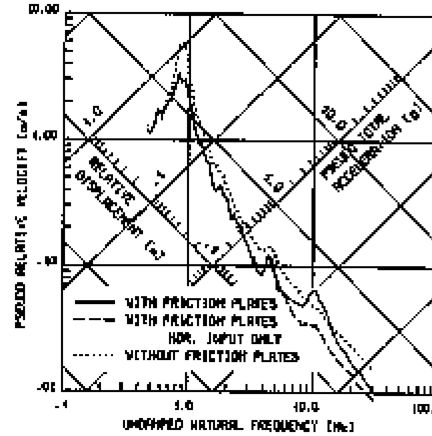


Fig. 12 Horizontal in-structure response spectra (2 % damping), nuclear island of Koeberg, upper raft

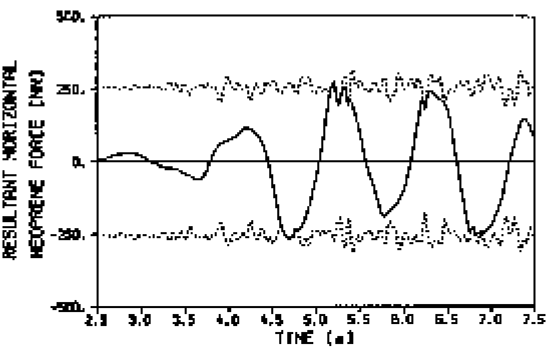


Fig. 13 Time-history of resultant horizontal Neoprene forces, nuclear island of Koeberg, vertical incidence

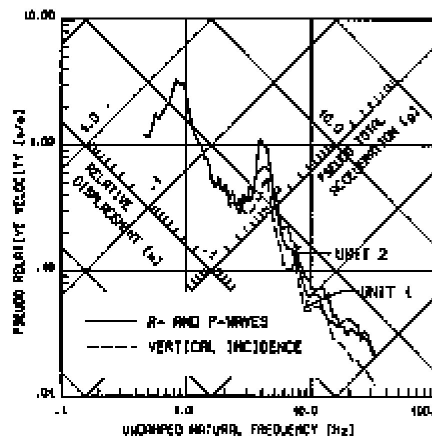


Fig. 14 Horizontal in-structure response spectra (2 % damping), nuclear island of Koeberg, top shield building

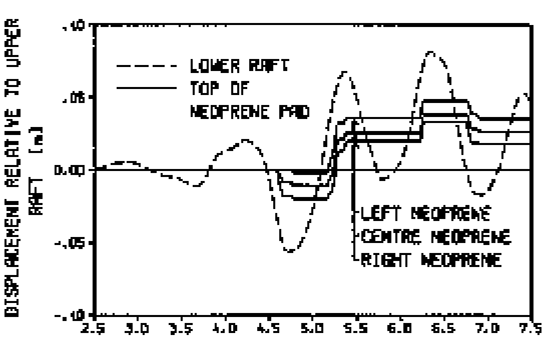


Fig. 15 Time-history of horizontal relative displacements, nuclear island of Koeberg, reactor building unit 2, vertical incidence

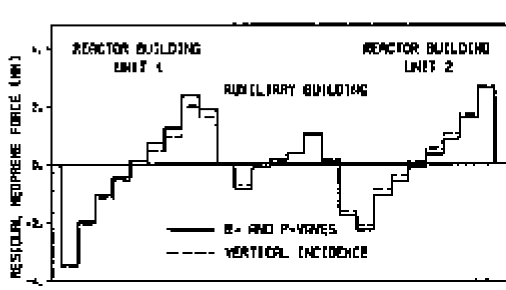


Fig. 16 Residual horizontal Neoprene forces, nuclear island of Koeberg

Comparative Analyses of a Nuclear Island on Linear and Non-Linear Aseismic Bearing Pads

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SUMMARY

As part of a feasibility study, a Nuclear Island resting on a foundation incorporating a double mat system with interposed aseismic bearing pads, which isolate the plant structures from severe horizontal motions, was analyzed for Safe Shutdown Earthquake excitation. The Nuclear Island comprises six Category I buildings constructed on independent upper mats resting on aseismic bearing pads. The six buildings are joined at the upper mat level by reinforced concrete structural connections which impose an in-phase horizontal vibration of the buildings during a seismic event. Two seismic isolation schemes were considered in the analyses. The first foundation included elastomer bearing pads attached directly to the upper mat and, through pedestals, to the common lower mat. The pad distribution and thickness were chosen so that the natural frequency of the Nuclear Island is equal to 0.6 Hertz for horizontal vibration. In the second scheme, sliding elements with a friction ratio of 0.2 were placed just below the upper mat and the pads were designed so as to achieve a natural frequency of the Nuclear Island equal to one Hertz. To investigate the response of the Nuclear Island when subjected to the design earthquake, a three-dimensional model of the Island and of its foundation was developed. The three-dimensional model included the building super-structures, the structural connections between the individual building mats, the aseismic interface and lumped parameter soil-structure interaction impedances.

The dynamic linear analysis was performed for the non-sliding pads utilizing the computer code DAPSYS and consisted of mode frequency analyses and modal superposition. The code ANSYS was used to perform the non-linear analyses by numerical integration of the equations of motion for the sliding pad scheme. In both analyses, the model was subjected to a postulated seismic event, represented by two horizontal and one vertical synthetic accelerograms matching the USNRC Regulatory Guide 1.60 Response Spectra, for a peak ground acceleration of 0.26g. The comparative behavior of the two seismic isolation schemes is presented in terms response spectra at various locations in the models, forces and moments in the structural mat connections, displacements of the upper mats, and deformation of the pads. For the Nuclear Island configuration and the seismic excitation considered, the analyses indicated that introduction of sliding pads reduces the acceleration level and overall response of the Nuclear Island structures. In particular, the horizontal displacement of the structures and the shear forces in the upper mat structural connections were significantly reduced by the use of sliding pads.

1. Introduction

Seismic foundation isolation schemes are an attractive solution for the design of nuclear power plants in seismic areas. By reducing the horizontal accelerations, especially in the high frequency range, they allow the use of a standardized design on sites with significant seismicity. Two solutions have been adopted to date to isolate the most important buildings of nuclear power plants during a postulated Safe Shutdown Earthquake (SSE). The first isolation scheme which was utilized in France (Cruas Nuclear Power Plant) includes elastomer bearing pads while the second scheme (used at the Koeberg Nuclear Power Plant in South Africa) also comprises sliding elements.

It s generally expected [1] that the first scheme will perform satisfactorily for peak ground accelerations up to about 0.23 g and broad design spectra e.g. the USNRC Regulatory Guide 1.60 [2].

This paper presents a comparison between these two isolations schemes for which dynamic analyses were performed as part of the feasibility study of a nuclear power plant to be designed for a peak ground acceleration of 0.26 g.

2. Foundation Conditions and Seismic Input

The soil profile considered for the feasibility study consisted of three main horizons:

- An upper deposit of basalt extending to a depth of 26 meters below foundation level with a shear wave velocity increasing from 1,100 to 1,800 meters per second,

- A sedimentary deposit consisting of about 80 percent clay, extending to a depth of 98 meters, with a shear wave velocity of 940 meters per second, and

- A deep deposit of andesite with a shear wave velocity of 1,500 meters per second.

The Safe Shutdown Earthquake (SSE) considered in the transient dynamic analyses for this nuclear power plant was defined by the horizontal and vertical response spectra of the USNRC Regulatory Guide 1.60 scaled to a peak ground acceleration of 0.26 g. Synthetic acceleration time histories developed [3] to match the design response spectra, at five percent damping, were used for this study.

3. Structural Arrangement of the Nuclear Island

The study considered the arrangement consisting of six Category I buildings resting on an aseismic interface. For both isolation schemes, this aseismic interface [4] includes reinforced elastomer bearing pads. The upper mat complex, which supports the Nuclear Island buildings, comprises the individual building mats linked together by means of stiff reinforced concrete connections. The lower mat, which is founded at a depth of about 11 meters below the final plant grade, is surrounded by a retaining wall. The Turbine Halls and other non-Category I buildings are placed adjacent to the Nuclear Island and are directly founded on the rock with conventional foundation schemes.

The distribution of the aseismic bearings under the six buildings of the Nuclear Island is shown in Figure 1. The aseismic bearings are composed of two, four, six or eight reinforced elastomer pads, each of which is an integral element of horizontal layers of elastomer alternating with metal plates. The purpose of the metal plates is to provide confinement of the elastomer and thereby minimize the shearing stresses in the elastomer caused by the vertical loads. In the second isolation scheme, a sliding bearing is located just above the elastomer pad. The lower plate is attached to the elastomer pad, whereas the upper one is attached to the upper mat. The two plates are made of special alloys to achieve a constant coefficient of friction approximatively equal to 0.2.

This arrangement provides an element with high vertical stiffness and low stiffness to horizontal shear forces. The effects of the elastometer pads are:

- o To control the horizontal stiffness of the foundation such that the natural frequency of the horizontal structure-foundation mode is well below the frequency range for high amplification of seismic motion.

- o To impose an almost pure horizontal translation mode on the structure, with little rocking participation.

- o To minimize or eliminate participation of higher structural modes by filtering the higher frequency earthquake input motions.

- o To transform the controlling seismic design parameter from acceleration into displacement by forcing the plant frequency to be in the low frequency portion of the design response spectrum.

The additional effect of the sliding bearings is to limit the maximum horizontal shear force transmitted to the buildings. In this manner, the horizontal accelerations are significantly reduced at the price of small residual displacements which can be corrected if slippage should occur.

4. Development of Analytical Models of the Nuclear Island

To investigate the response of the Nuclear Island when subjected to the design earthquake (SSE), a three-dimensional model of the island and of its foundation was developed. The structures of the Nuclear Power Plant which were included in the model are shown in Figure 1. The analytical model developed for the purpose of computing the seismic response is composed of the following components :

- o Building superstructures
- o Structural mat connections
- o Aseismic interface
- o Soil-structure interaction parameters

Each of these elements are discussed in the following subsections.

4.1 Building Superstructures

For each building, lumped masses, and lumped moments of inertia at the floor locations, were used to represent the mass of the floors, walls and equipment, while the walls between floors were modeled as beam elements (Figure 2).

4.2 Structural Mat Connections

In order to prevent relative displacements between the upper mats during the postulated seismic event, structural connections are provided between adjacent buildings. These connections comprise a solid concrete slab spanning from one building to the other and extending along the entire interface between the buildings being connected. Owing to the reinforcing effect of the building superstructures on their own slabs, the building mats were assumed to be rigid and the connections between them were given the ability to resist all possible relative motions between the buildings they link. The structural connections were therefore represented in the mathematical model as 12 by 12 stiffness matrices connecting the lowest nodes of the stick models of the connected structures (Figure 2).

4.3 Aseismic Interface

For both isolation schemes, the upper and lower mats of the building foundations are interposed by elastomer pads, mounted on concrete pedestals, cast integrally with the bottom mat (see Figure 1). The elastomer pads are distributed so as to give a uniform vertical working stress of 5.5 MPa under static conditions. The total thickness of elastomer is thirteen and five centimeters for the non-sliding and the sliding isolation schemes, leading to natural frequencies of 0.6 and one Hertz, respectively.

The stiffness and damping of the complete group of pads beneath each building were represented by six springs and six dampers (three translational and three rotational). These lumped parameters were obtained from the springs and dampings at each pedestal and their distribution with respect to the center of the building. For the sliding scheme, special sliding elements were placed in series with the springs (Figure 2).

4.4 Soil-Structure Interaction Parameters

In order to take proper account of the interaction between the structures and the foundation soils during earthquake loading, the foundation impedance of the Nuclear Island was determined according to the following steps:

- o Establish a design soil profile with pertinent zero-strain dynamic properties for the different layers.

- o Estimate the reduction in shear modulus and the material damping ratio of the substrata caused by "free-field" earthquake motion using the computer code SHAKE [5].

- Calculate the zero-frequency (static) impedance matrix of the global Nuclear Island foundation and its variation using frequency using the methodology developed by Kausel [6].

- Based on the above, calculate each term of the global dynamic stiffness and damping matrices applicable to the soil-structure interaction frequency of each mode of vibration.

- From static elastic half space theory, redistribute the stiffness under each building so that the global behavior of the Nuclear Island foundation is properly represented. This was achieved by means of a higher order winkler (HOW) model which introduces coupling between adjacent buildings (Figure 2) so as to simultaneously account for translational and rotational interaction [7]. Similarly, redistribute, for each mode, the global damping matrix under each building foundation.

5. Dynamic Analyses of the Nuclear Island

The mathematical model of the buildings, the structural connections, the aseismic interface and the foundation represented in Figure 2, was subjected to a linear analysis for the non-sliding isolation scheme and a non-linear analysis for the scheme with sliding bearings. These analyses are described in the following subsections, while the results are presented and compared in Section 6.

5.1 Non-Sliding Isolation Scheme

The dynamic linear analyses for the non-sliding pads which were performed utilizing the computer code DAPSYS [8], consisted of mode frequency analyses and modal superposition with composite modal damping. The composite modal damping is computed in DAPSYS as a weighted average of structural and soil damping [9], using the strain energy stored in the respective components as the weighting factor. This approach distinguishes the hysteretic nature of the structural and soil material damping from the viscous nature of the radiation damping. The modal analyses covered frequencies up to 30 Hertz and the response of each mode was integrated with a time step of 0.01 seconds.

5.2 Sliding Isolation Scheme

The ANSYS computer Code [10] was used for the transient non-linear analyses in this study. In order to obtain better computational efficiency, the substructuring technique of the finite element method was utilized for the superstructure and the upper mat of the Nuclear Island. The stiffness, mass, damping and gravity load vectors of this superelement were then stored on a computer file. The complete model of the plant was then developed by connecting the substructure described above to fixed supports under each building through the mathematical representation of the aseismic pads and of the soil impedance.

The damping of the superelement was introduced by means of Rayleigh damping while the damping of the aseismic pads and of the foundation soil were input as damping matrices.

Transient dynamic solutions were obtained by numerical integration of the equations of motion using the Houbolt numerical integration scheme which is unconditionally stable for all integration time steps. An integration time step of 0.005 second proved adequate for the non-linear analyses.

6. Presentation and Discussion of the Results

In order to investigate the performance of the aseismic foundation-structure systems and assess their feasibility, the following results were obtained from the dynamic analyses :

- Floor response spectra at the base and top of the buildings.
- Relative displacements of the upper mats, magnitude of slippage (if any), vertical and shear stress in the aseismic bearings.
- Forces and moments in the structural connections between buildings.

The response spectra at the base of the Reactor Building are shown in Figure 3 and compared to the design ground response spectra. The horizontal spectra illustrate the filtering effect of the aseismic isolation schemes in the high frequency range. The responses are very sharp and of similar magnitude near the fundamental frequencies of 0.6 and one Hertz for the non-sliding and sliding schemes, respectively, and are nearly flat above frequencies greater than about three time the fundamental frequencies. The maximum horizontal accelerations are reduced from 0.25 g to 0.23 g when sliding is permitted. Owing to the high vertical stiffness of the elastomer pads, the vertical spectra are virtually identical.

The horizontal relative displacement and rocking time histories at the base of the Reactor Building are given in Figure 4. The horizontal displacements and the torsion of the nuclear island for non-sliding pads are about twice those of the sliding pads. On the other hand, the rocking is only slightly larger for the non-sliding scheme. From the linear analysis (non-sliding scheme), the displacements and load time histories acting in each individual pad were obtained. Special attention was given to the distortion, the vertical stress and the ratio (H/V) of the horizontal to the vertical load acting in the pads. These results show that the maximum relative distortion is equal to 1.4 in all pads, that some pads experience an uplift and that large values of H/V (on the order of 0.35) exist, especially in the pads located along the periphery of the island. The results of the non-linear analyses (sliding scheme) where H/V is limited to the friction coefficient of the sliding plates (0.2) show that the maximum pad distortion is reduced to one.

Finally, the time histories of vertical shear force and bending moment acting in a 7.2 meter linear segment which was used to represent the circular structural connection between the Electrical and Reactor Buildings at the location indicated in Figure 1 are given in Figure 5. The sliding scheme performs again more favorably especially for the shear force which is reduced by a factor of about two.

7. Summary and Conclusions

The performances of two schemes for the seismic isolation of a nuclear island have been evaluated and compared for a seismic motion matching the USNRC response spectra with a maximum ground acceleration of 0.26 g. The study has shown that for the soil conditions, the structural arrangement and the postulated seismic event considered, the aseismic foundation scheme including sliding plates was preferable. Also, the non-sliding isolation scheme which required a high thickness of elastometer yielded large relative distortion (1.4) in the pads and local uplift which would have required special techniques for design and construction of the pads.

This study has demonstrated that even a slight increase in acceleration level, over that considered to be acceptable for a non-sliding isolation scheme, is sufficient to show definite advantages for the sliding plate isolation pads in terms of structural response.

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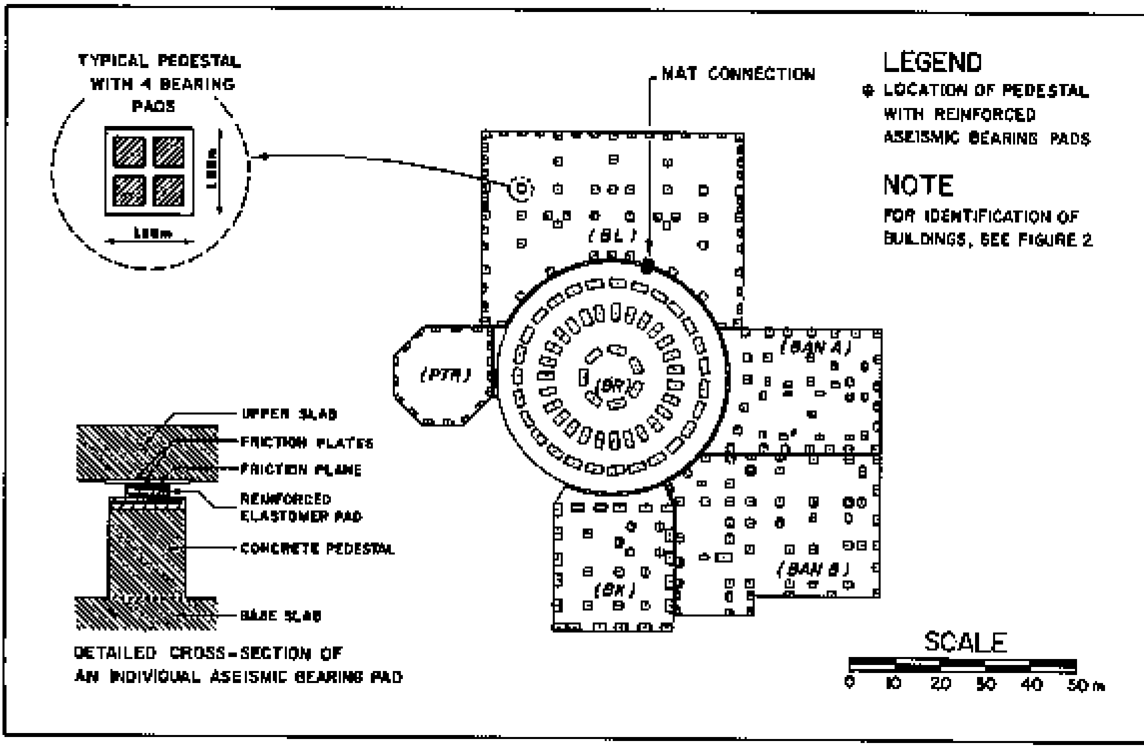

FIGURE 1 LAYOUT OF NUCLEAR ISLAND AND ASEISMIC BEARING PADS

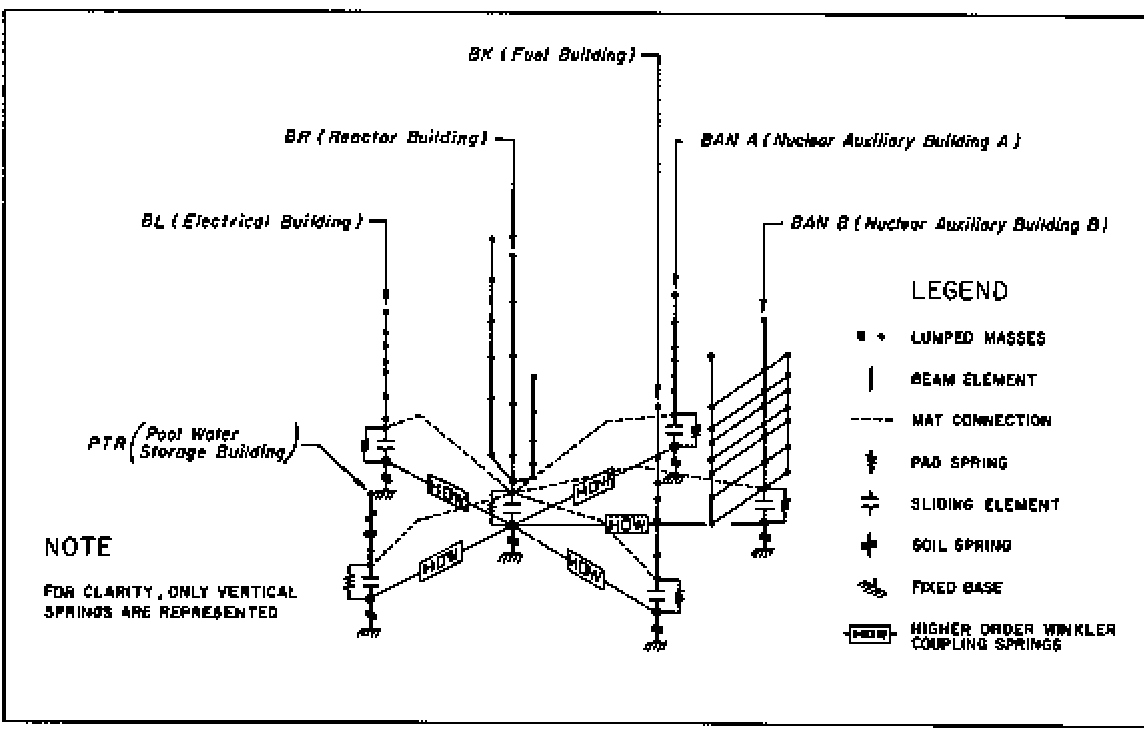

FIGURE 2 MODEL OF NUCLEAR ISLAND

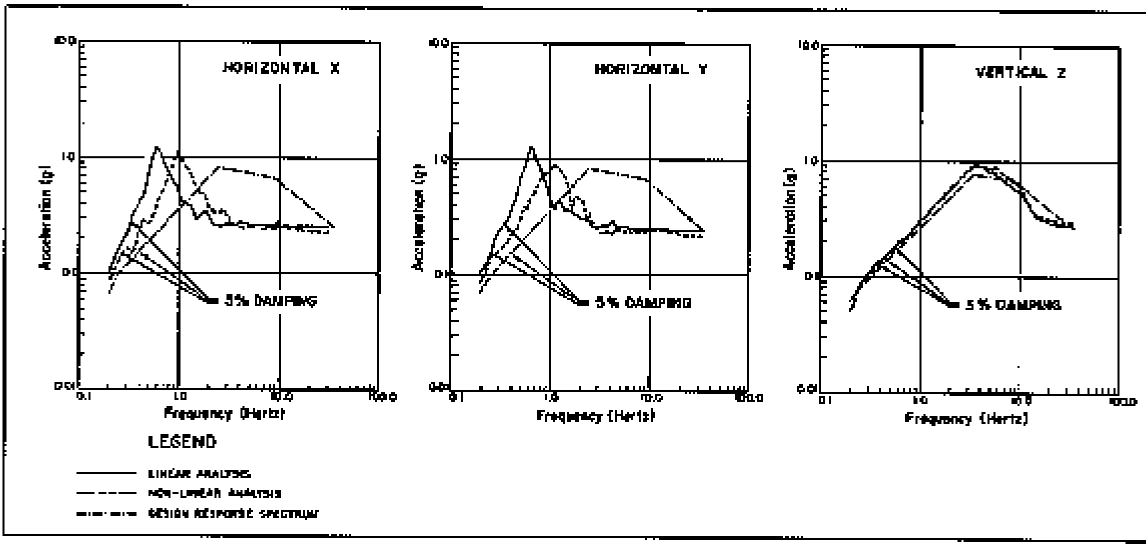

FIGURE 3 RESPONSE SPECTRA AT BASE OF REACTOR BUILDING

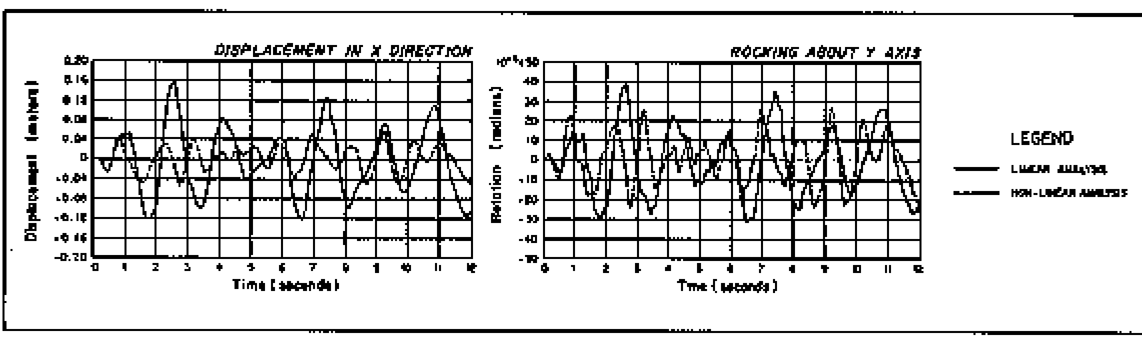

FIGURE 4 MOTIONS AT BASE OF REACTOR BUILDING

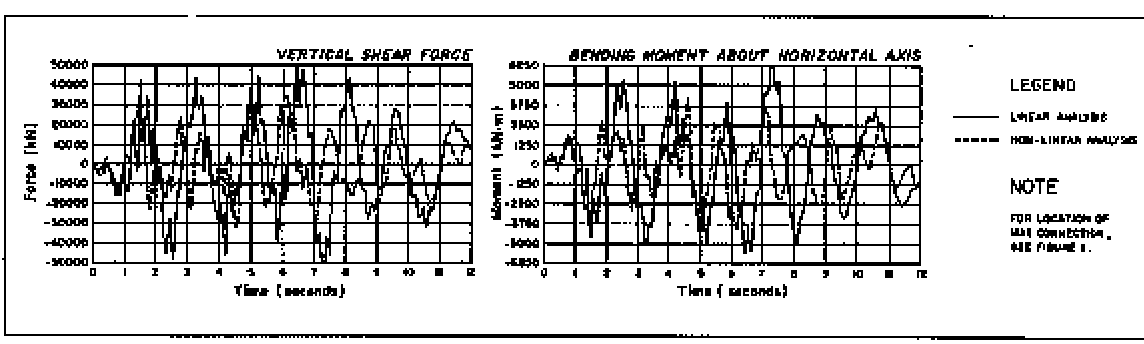

FIGURE 5 FORCES AND MOMENTS IN A TYPICAL MAT CONNECTION