PB88-235338

Project No. 86-246
March 1988

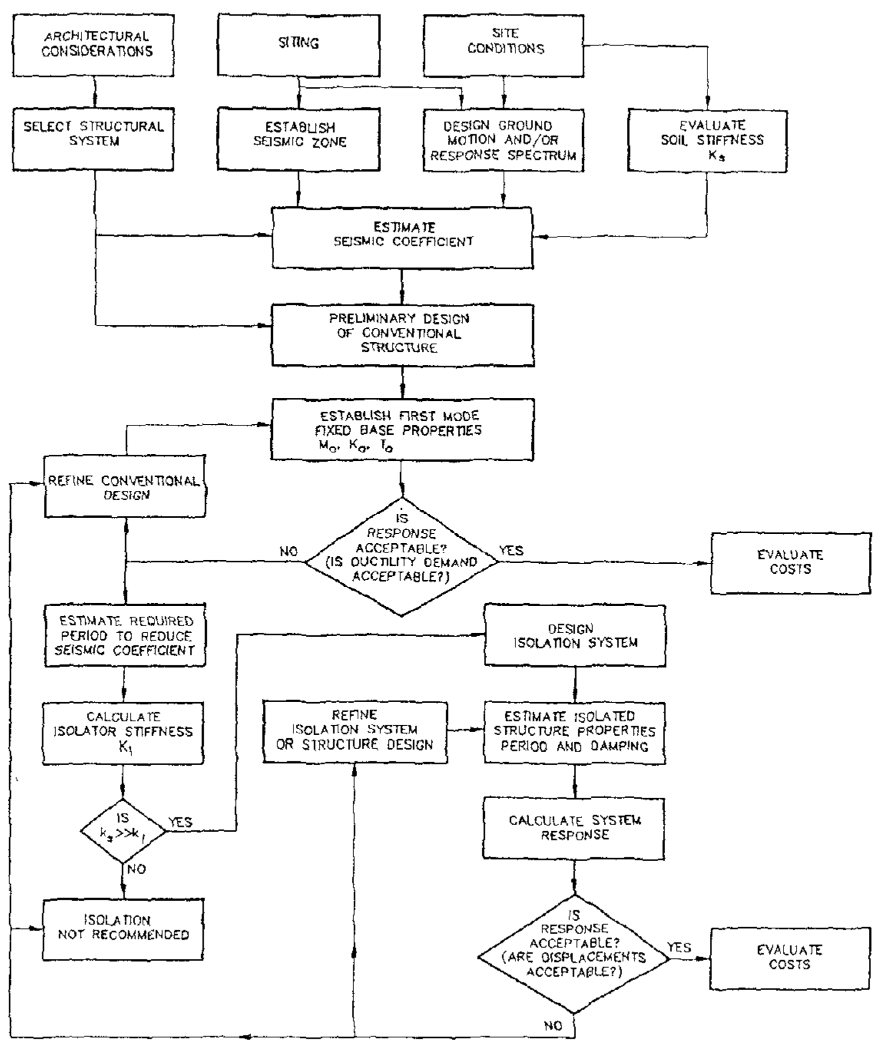
Paul C. Rizzo Associates, Inc.
CONSULTANTS

Report

Seismic Design of Structures Using Base Isolation

National Science Foundation
Washington, D.C.

Qualitative Guidelines



50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NSF/ENG-88003	2.	3. Recipient's Accession No. PB88 235338/AS

4. Title and Subtitle	5. Report Date
Seismic Design of Structures Using Base Isolation, Qualitative Guidelines	March 1988
	6.
7. Author(s) N.R. Vaidya; E. Bazan-Zurita	8. Performing Organization Rept. No. 86-246
9. Performing Organization Name and Address Paul C. Rizzo Associates, Inc. 10 Duff Road, Suite 300 Pittsburgh, PA 15235	10. Project/Task/Work Unit No. 11. Contract(C) or Grant(G) No. (C) (G) CES8615240
12. Sponsoring Organization Name and Address Directorate for Engineering (ENG) National Science Foundation 1800 G Street, N.W. Washington, DC 20550	13. Type of Report & Period Covered 14.

15. Supplementary Notes

16. Abstract (Limit: 200 words)

The purpose of the report is to review the overall concept of base isolation and to develop conceptual criteria for determining the applicability of base-isolated systems. The report develops qualitative guidelines for selecting base isolation as a design strategy. Based on the base isolation design philosophy and case histories, a qualitative criteria for selecting base isolation is proposed, and the important issues which must be considered in this selection process are discussed. Various base isolation schemes are briefly summarized and their anticipated performance described. The purpose here is to characterize the various base isolation concepts, identify their relevant parameters, and discuss briefly their anticipated performance in reducing the effects of earthquakes.

17. Document Analysis a. Descriptors

Structural design	Isolation
Structural engineering	Earthquake resistant structures
Earthquakes	Damage

b. Identifiers/Open-Ended Terms

Earthquake engineering
Base isolation

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
NTIS		146
	20. Security Class (This Page)	22. Price PC A07 9.95 / MF A01 6.95

(See ANSI–Z39.18) See Instructions on Reverse OPTIONAL FORM 272 (4–77)

SEISMIC DESIGN OF STRUCTURES USING BASE ISOLATION QUALITATIVE GUIDELINES

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A REPORT SUBMITTED TO
THE NATIONAL SCIENCE FOUNDATION

PROJECT NO. 86-246
MARCH 1988

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ACKNOWLEDGMENTS

The work described in this report was supported in part by the National Science Foundation under Grant No. CES-8615240 and in part by the Army Corps of Engineers and Naval Facilities Engineering Command. The support of the above organizations is gratefully acknowledged.

DISCLAIMER

This report was prepared on the basis of work funded by the U.S. National Science Foundation in cooperation with the Army Corps of Engineers and the Naval Facilities Engineering Command. Any opinions, findings, and conclusions or recommendations expressed in this report are solely those of the authors and do not necessarily reflect the views of the National Science Foundation, the Army Corps of Engineers, or the Naval Facilities Engineering Command

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SEISMIC DESIGN OF STRUCTURES USING BASE ISOLATION QUALITATIVE GUIDELINES

1.0 INTRODUCTION

Structural upgrading for the purpose of mitigating effects of earthquakes can consider several alternative solutions. Very broadly, they can be fitted into two categories--one is to strengthen the structure and the other is to isolate the structure from the seismic ground motion. Both alternatives are applicable in the design stage for new construction, but if properly detailed, can also be applied to retrofit existing structures.

Among the alternatives for seismic structural upgrading, the traditional approach has been to add strength, stiffness, and ductility to the structural systems. Typically, additional bracing and/or shear walls are considered to take safely the horizontal forces generated from the design seismic event and transmit them to the building foundation without significant relative displacements. The seismic forces chosen for design must provide reasonable assurance that the building will not collapse in the event of the assumed severe earthquake, although the structure may be subjected to permanent damage.

In structures of historical significance, the traditional methods may not be the most attractive if they mar the architectural appearance of the facade, which must be preserved to elicit, at least partially, the worth of the facility. For certain essential buildings, their intrinsic value dictates that in addition to protecting life, the structure and contained systems suffer as little damage as is feasible to reasonably assure the continued operation of the facility during and following the earthquake. Thus, the importance of the structure, site conditions, seismic potential, and other related issues may prompt consideration of alternate upgrading.

Deviating from the traditional approach, two alternative approaches have been broadly considered by the profession to mitigate earthquake effects:

- Reduction of input motion through insulation of the structure, and
- Attenuation of input motion through isolation of the structure.

Insulation takes the form of backpacking or incorporating a trench around the facility to segregate it from the free-field. Conceptually, the trench reflects and attenuates incoming seismic waves and, thus, effects a reduction of the input ground motion in the vicinity of the building foundation.

It is noted, however, that an empty trench seems unlikely to be of much help unless a significant part of the seismic ground motions is due to surface waves. And even here, any reduction in incoming seismic waves could be offset by the loss of lateral support to the building and, in some cases, the loss of damping provided by the soil against the building foundation. Unless the empty trench can be located far enough away from the building, this technique may not be as useful for eliminating seismic motion as for ground shock from explosives.

In general, isolation takes the form of supporting the building on a flexible foundation such that the seismic ground motions are attenuated as they are imparted to the structure through the isolating mechanism. The general concept is illustrated on Figure 1. As the isolation mechanism is located at the base of the structure, it is referred to as base isolation.

In a broad sense, most base isolation schemes consist of separating the ground and the supported structure by a flexible connection with one or more energy absorbing devices acting either in parallel or in series. Depending upon the system of isolation employed, either the flexible link or the energy absorbing device takes on the greater significance. Thus, at the one extreme, attenuation of seismic motion depends primarily on the capacity of the isolation system to absorb the seismic energy imparted at the structure/foundation interface; in the other extreme, the flexible link alone may be designed to reduce the damage potential by means of a frequency shift of the structure relative to the earthquake ground motions. This is also referred to as "detuning."

The purpose of this report is to review the overall concept of base isolation and to develop conceptual criteria for determining the applicability of base-isolated systems. Although additional research is required before the cost-benefit of conventional versus base-isolated seismic upgrading can be quantified, it is possible to define situations where base isolation is recommended. Conversely, cases where traditional solutions are more favorable can also be identified.

This report develops qualitative guidelines for selecting base isolation as a design strategy. Based on the base isolation design philosophy (Section 2.0) and case histories (Section 3.0), a qualitative criteria for selecting base isolation is proposed, and the important issues which must be considered in this selection process are discussed (Section 4.0). Various base isolation schemes are briefly summarized and their anticipated performance described (Section 5.0). The information about these schemes has been collected from published literature, technical papers, notes, etc. The purpose here is to characterize the various base isolation concepts, identify their relevant parameters, and discuss briefly their anticipated performance in reducing the effects of earthquakes. Depending on the type and use of the building structure and structural considerations imposed by base isolation (Section 6.0), some concepts may be more suitable than others. An overall summary and recommendation for additional research are provided in Section 7.0 as concluding remarks. It is hoped that the information presented in this document will aid in selecting an appropriate design strategy and evaluating base isolation systems during the conceptual design stage of a building structure.

2.0 BASE ISOLATION DESIGN PHILOSOPHY

A general overview of the basic principles of base isolation can be found in texts on earthquake engineering (e.g. Newmark and Rosenblueth, 1971). As this would signify, the concept is not new; its history in seismic applications, which dates back to the early 1900's, has been very aptly described by Kelly (1979). The more recent interest has been motivated by several factors. In particular, the ground motion characteristics of recent, destructive earthquakes have demonstrated the need for a higher level of earthquake performance of buildings. Equally important, the development of reinforced elastomer bearings has extended the applicability of base isolation systems to the earthquake protection of structures.

2.1 CONCEPT

In the strategy of base-isolated aseismic design, the entire structure is founded on several bearings which are relatively flexible in the horizontal direction and simultaneously are capable of supporting the vertical gravity load of the structure. An example is the recently completed Law and Justice Center Building in San Bernardino, California. The design of base isolation for this building is illustrated on Figure 2 (Huang et al., 1986). In another example, Figure 3 shows a schematic of the Koeberg Nuclear Island foundation. The nuclear island structure is supported on about 2,000 bearings mounted on 600 concrete pedestals. All of the bearings together with any secondary and additional damping devices, mechanical fuses, restraints, etc., form the base isolation system.

The seismic design of the base-isolated structure establishes the required performance characteristics for the entire base isolation system. These may comprise the required total vertical and horizontal stiffness and damping parameters of the base isolation system. The distribution of these gross parameters to the individual bearings, as shown for example on Figure 3, is a structural design function, and roughly corresponds to the distribution and arrangement of vertical gravity loads and the vertical load-carrying structural elements.

2.2 SEISMIC RESPONSE

The difference in the seismic response between base-isolated and conventionally-founded structures is qualitatively illustrated on Figure 4. For a conventional foundation, the ground motion produces significant relative displacements in the structure. Depending on the structural system, either shear deformation or flexural deformation predominates. In general, buildings exhibit both types of deformation in responding to seismic ground motions. The base-isolated structure, on the other hand, responds predominantly by displacing more or less as a rigid body on the flexible bearings. Relatively small amplitudes of ground motion are transmitted to the structure and, consequently, the deformation of the structural system in its fundamental modes is small. In this sense, the structure and its contents are isolated from ground motions.

Isolation from ground motion is effected by providing a low horizontal stiffness to the bearings. This substantially reduces the frequency of the predominant horizontal mode of the structure vibration, typically lower than 1.0 Hertz. Most earthquake ground motions have predominant frequencies in the 3 to 10 Hertz range and are therefore effectively filtered by the isolation system, allowing only relatively small accelerations to be imparted to the isolated structure. The general principles are illustrated on Figure 5 which shows an idealized response spectrum of earthquake ground motion. For simplicity, it is assumed that the isolated structure displaces horizontally on the collection of bearings in a more or less rigid fashion, such that the base-isolated structure can be represented by a single-mass oscillator. The conventional structure may have predominant periods in the range where the ground motion is amplified. By providing a flexible base, the fundamental period is shifted from T_1 to T_2, which is outside the range where the ground motion is amplified. Very simply, the principle of base isolation is to effect this shift of the predominant mode of vibration. Indeed, in the extreme case one could support the entire structure on frictionless bearings which theoretically would totally isolate the structure in the horizontal direction. Obviously, the resulting base displacements would be very large. Even a nominal shift in the frequency, such as attempted in most base isolation solutions, increases the displacement response to the point that some measure to control these displacements needs to be incorporated as an integral part of the isolation system.

As an example of differences in seismic response, consider the building shown on Figure 6 (NAVFAC P-355). This is a three-story administration building with reinforced concrete bearing walls and a series of interior vertical load-carrying columns and girder bents. The structure is about 48 feet by 192 feet in plan view. Figures 7 and 8 (Vaidya and Eggenberger, 1984) present a comparison of response of the conventional structure to that of the base-isolated structure for the El Centro N-S record scaled to a peak acceleration of 0.6 g (g = gravity acceleration). In addition to the elastic response of the conventional structure, story shears and overturning moments corresponding to SEAOC Zone 4 requirements are also presented. The base-isolated frequency was taken to be about 0.67 Hertz and the response was calculated for viscous dampings of 5 and 15 percent, and for the case where energy dissipation in the form of a frictional interface is included in the base isolation system. It is seen from Figures 7 and 8 that the accelerations, forces, and moments are significantly reduced and displacements are increased relative to those corresponding to the fixed-base structure. However, a major part of the displacements occur in the base isolation system and interstory displacements are small. This confirms that the base-isolated structure responds approximately as a rigid body.

2.3 SEISMIC PERFORMANCE

In simple terms, the seismic behavior of base-isolated structures can be looked at in terms of the response of damped low-frequency, single-mass oscillators, similar to how conventionally-founded structures are analyzed. From the practical standpoint, the similarities end there and the seismic performance of base-isolated structures is radically different from that of the conventional.

To illustrate the above, consider Figure 9. Curves A and B represent the design recommendations by SEAOC (1985) for conventionally-founded buildings. Curve A represents forces for use in design of a shear wall building with a response reduction factor, R_w, equal to 6.0 together with special provisions to obtain ductility implied by the R_w factor. Curve B represents a response spectrum for use in a modal response spectrum analysis. The combined modal response obtained in a response spectrum analysis using Curve B are scaled to values associated with Curve A. Curve C is the linear elastic response spectrum of the El Centro N-S record for a five percent damping. In general, the difference between the elastic spectrum of an expected earthquake and the design forces represents the magnitude of the ductility demand in the structural components in the event of the earthquake. It can be seen that the largest ductility demand is in the period range from about 0.1 seconds to 0.5 seconds. This is generally true for most earthquake records, unless the building is located at an unusually soft site (e.g., Mexico D.F.).

It is evident from Figure 9 that the conventionally designed structure relies on the ductility of its structural components to absorb a significant part of the seismic energy imparted to it. On the other hand, by design, the base-isolated structure does not participate significantly in the seismic response and its structural components are designed to remain within the elastic range for the reduced levels of seismic forces. Thus, the potential for its structural damage during a severe earthquake (damage which is generally accepted in the conventional seismic design philosophy) is minimized. But because of this, the isolation bearing becomes the all-important structural element, sacrificing some of the redundancies inherently present in conventional design. For a base-isolated building, the structural design needs to assure a proper vertical and horizontal load distribution to the dispersed and discrete bearing elements. Very broadly then, the design philosophy attempts to relieve the risk to the building structural elements by an approximately equal sharing of the risk by the bearings.

2.4 DISCUSSION

Much has been written and published about base isolation and its strategy of seismic design. A recent workshop (ATC, 1986) on base isolation and passive energy dissipation brought together the current thoughts on the use of numerous techniques for aseismic design of building structures. Some recommendations for the design requirements of base-isolated structures have been proposed (SEAONC, 1986, SEAOSC, 1986). Other guidelines (Vaidya and Eggenberger, 1984) present specific design issues that need to be considered if this strategy is chosen.

There is still not a complete acceptance of base isolation by the profession although it is agreed by most that the principle is sound and is applicable at least to certain types of structures. The apprehension results, perhaps justifiably so, from the lack of extensive field data on performance of structures constructed on aseismic bearings (Huang et al., 1986). For the most part, current design has to rely rather heavily on the results of analyses using mathematical models and scale model shaker table tests which generate results in a controlled laboratory environment. Indeed, these do not encompass the myriad of variable conditions that will be encountered in the field. The collection of field data over several years of observed actual performance remains the primary means to refine the technique of base isolation.

The reliability of the base isolated design is governed, perhaps more significantly, by uncertainties in seismic ground motion and structural response. Uncertainties regarding seismic hazard apply to conventional as well as isolated structures. Over the last two decades, a wealth of information has been gathered on the nature of ground motion and the sources of uncertainties in its prediction. Regarding structural response, base-isolated structures lend themselves to a more representative mathematical model for estimating seismic effects than conventional structures. Isolation demands little participation from the superstructure and the non-structural components in responding to seismic ground motion, and therefore reduces the attendant uncertainties in the mathematical model and renders analytical effort more tractable.

The principal structural element used in the base isolation system, the bearing, is an engineered product admitting to QA/QC with quite a respectable history of use in other applications, some in far more arduous environments. At the time of installation, selected bearing tests can reasonably assure that installed bearings meet the required design characteristics. The long-term performance of the bearings, however, is less assured as all elastomers harden with age. The degree of stiffening is affected by the environment as well as the bearing size. Currently, no good method exists to check the bearing properties in situ except for testing witness samples stored in the same environment. Uncertainties in response due to elastomer aging need to be studied systematically. So, until such time as enough long-term data is accumulated, the design should provide means for bearing replacement.

On the basis of the wealth of analytical and experimental data and some field performance data, it appears that base isolation is a viable alternative to traditional methods of seismic design of building structures.

3.0 SURVEY OF USE

The most common civil engineering applications of the concepts of isolation have been motivated by the need to protect against:

- Ambient vibrations, e.g., machinery,
- Effects of seasonal temperature changes, e.g., thermal growth of bridges, decks, and
- Effects of seismic ground motion.

The first two applications have been pursued by the engineering community for over 30 years, and reinforced elastomer bearings are commonly specified to mitigate their effects. Isolation against seismic ground motion has been considered since the early 1900's (Kelly, 1979). However, it was not until about 1970 that base isolation to protect buildings against earthquakes saw practical application. In a large part, it was because of the successful application of elastomer bearings in the first two areas that their use in the third one was adopted. Several recent papers describe the historical development of the base isolation concepts and their practical applications (e.g., Lee and Medland, 1978; Kelly, 1986; and Buckle, 1986).

This section includes a a brief survey of use based on published literature. Its intent is to convey the different types of structures for which the concept of base isolation has already been used. To date, several structures are base-isolated for seismic protection in the United States and other countries, and several countries have active ongoing research and development programs in this area.

Different types of structures benefit from base isolation. These include buildings, bridges, viaducts, towers, residential houses, storage facilities, etc. Buckle (1986) has summarized a world overview of this activity, and it underscores the proliferating interest in this application. Table 1 presents a summary of pertinent data of building structures that have been constructed using base isolation. Several of these structures are discussed in more detail in the following paragraphs. It is interesting to note that the application base is rather broad, extending from structures as important and complex as nuclear power plants to those as simple as residential units.

3.1 HEINRICH PESTALOZZI SCHOOL, SKOPJE, YUGOSLAVIA

This school building is a three-story high, reinforced-concrete structure built in 1969, designed by a Swiss group as part of the post-1963 earthquake reconstruction effort. Plan dimensions are 60 by 13 meters (197 by 43 feet). The building rests on 54 blocks of natural rubber combined with foam glass stabilizers (Staudacher, 1985). A total building weight of 5,300 kips is supported by the rubber pads, and no measurable signs of aging had been detected by 1981. The rubber bearings differ from those used today in that they do not contain reinforcing steel plates, and are fabricated by gluing together several layers of rubber. These bearings are an early version of the Swiss seismafloat system. This building is perhaps the first structure to use elastomer bearings for seismic protection (Kelly, 1986).

3.2 OFFICE BUILDINGS, ATHENS, GREECE

Two office buildings (one a six story; the other, three story) in Athens, Greece are supported on a base isolation system called the "Alexisismon" (Ikonomu, 1979). The base isolation system includes a combination of: 1) sliding pot bearings which support all vertical loads, permit rotations around the horizontal axis, and allow displacements of the structure relative to the ground on the order of 20 to 40 inches; 2) rubber blocks which are free of buckling problems, as they do not support any vertical load, and provide restoring forces; and 3) connecting steel bars acting as restrainers/fuses which resist wind loads but break during a large earthquake. The components of the Alexisismon system are designed to behave elastically (except for the breaking fuse) under the seismic excitation.

The base isolation system for the six-story building was designed for a maximum relative displacement between the base of superstructure and ground of 0.50 meters (20 inches), and has been analyzed for simultaneous translational and torsional seismic inputs (Ikonomu, 1984). Using the El Centro 1940 N-S record, it was found that the building, the equipment, and the components of the isolation system will perform satisfactorily up to a peak ground acceleration of 1.35 g. Including torsional ground motions reduces that limit to 1.19 g. The base seismic coefficient does not exceed 0.11.

3.3 THE WILLIAM CLAYTON BUILDING, WELLINGTON, NEW ZEALAND

This government office building incorporates a base isolation system developed in New Zealand. The building is a three-story reinforced-concrete frame structure with a full basement. Interstory heights are five meters (16.4 feet), and plan dimensions are 97 by 40 meters (315 by 130 feet). The structure comprises 15 by 4 bay frames on 7.2 meters (23.5 feet) centers, for a total of 80 columns. The columns rest on natural rubber bearings 600 millimeters (24 inches) square in plan by 207 millimeters (8.15 inches) thick. The bearings include lead plug inserts of 105 millimeters (four inches) diameter to improve their damping characteristics (Megget, 1978). A schematic view of the building and the isolation system is presented on Figure 10.

Time history dynamic analysis was used to estimate seismic response of the building. The first ten seconds of 1.5 times El Centro earthquake was used, and the analysis (Megget, 1984) showed that the maximum base shear of the isolated structure is 20 percent of that corresponding to the non-isolated case. The analysis also showed that only a roof beam yields for the selected ground motion, without hinge reversal. Maximum interstory distortions of 0.002 times the story height were obtained, about one-fifth of the maximum drifts of the non-isolated structure.

3.4 KOEBERG NUCLEAR PLANT, SOUTH AFRICA

The safety structures of the Koeberg Nuclear Power Station are supported on the Electricite de France (EDF) base isolation system. This system of base isolation (referred to later on as EDF system) was developed for nuclear power plant structures specifically to allow the use of standardized design for a range of site conditions, including areas of high seismicity (Plichon, 1975; Jolivet and Richli, 1977). The structures rest on a double mat foundation with the isolation bearings installed between the two mats. A schematic view of the foundation arrangement is shown on Figure 3.

About 2,000 bearings support the building structures. The bearings are 30 inches square in plan and five inches thick and consist of steel-reinforced neoprene pads. The bearings incorporate a frictional interface with a friction coefficient of 0.2. The isolation system is described in more detail in Chapter 5.0.

The fixed-base fundamental frequencies of the structures ranged from about 3 to 8.0 Hertz. When base-isolated, the predominant frequency of the complex of buildings is reduced to about 0.9 Hertz. A comparison of the seismic horizontal response of the conventionally-founded containment building structure and the base-isolated containment building is shown on Figure 11. In both cases, the response is obtained for the El Centro 1940 earthquake record scaled to 0.6 g. It is noted that acceleration levels are almost constant throughout the height of the isolated structure and are significantly less than those corresponding to the non-isolated structure.

Figure 12 shows typical time histories of relative horizontal displacements between the upper and lower mats of the nuclear island. These responses have been obtained for the El Centro 1940 N-S record scaled to various values of peak ground acceleration. The total relative displacement at any instant of time consists of the linear distortion of the neoprene plus the slippage on the friction surface. The displacement time histories shown on Figure 12 also show superimposed time histories of slip on the friction surface.

The floor response spectra in the base-isolated structures also shows a marked difference from the conventional. The peak in the floor response spectra of a base-isolated structure lies in the low frequency region which is generally out of range of predominant equipment natural frequencies. The comparison of floor response spectra for conventional and base-isolated structures is shown for example on Figure 13 and 14 for locations on the upper mat and the top of the reactor building, respectively. The floor response spectra have been obtained for a peak ground acceleration of 0.3g, and a time history of ground motion consistent with the USNRC Regulatory Guide 1.60 spectrum (USNRC, 1975). It should be noted from the comparison of the floor response spectra that the spectral values at predominant equipment frequencies (in the range from 5 to 15 Hertz) are significantly reduced when base isolation is used.

3.5 ELEMENTARY SCHOOL, LAMBESC, MARSEILLES, FRANCE

This three-story school building is a precast concrete panel structure that was constructed on a base isolation system called the GAPEC, developed at the Centre National de la Recherche Scientifique in France. The school has plan dimensions of about 77 by 26 meters (250 by 85 feet). It comprises three buildings separated from each other by 10 centimeters (four inches) seismic gaps. The isolation system uses 152 bearings of a multi-layer construction in which natural rubber sheets and steel plates are bonded by vulcanization. The vertical stiffness of the bearings is approximately 500 times the horizontal stiffness. The bearings are typically one foot in diameter with a total rubber thickness of 40 millimeters (1.6 inches) and they behave linearly for shear strains up to 100 percent (Delfosse, 1977).

The predominant fundamental frequency of the fixed-base buildings was about five Hertz and was reduced to 0.6 Hertz by the isolation system. The school was designed for an earthquake of intensity VIII. Base isolation decreased significantly the precast panel thickness required for seismic resistance.

3.6 RADIOACTIVE WASTE BUILDING, TORILLON, FRANCE

This building is a three-story reinforced-concrete structure with 40 centimeters (16 inches) thick external shear walls (Delfosse and Delfosse, 1984), which is schematically shown on Figure 15. Plan dimensions are 24 meters (79 feet) by 13 meters (43 feet), and the structure is about 13.75 meters (45 feet) high in addition to the 1.50 meters (5 feet) high basement where the isolation bearings are located. The building is used to store radioactive waste and protection against environmental contamination was a major design consideration requiring the use of thick concrete slabs and basement shear walls. The base isolation system comprises of 52 isolators located at the intersections of the moment-resisting frames and shear walls.

Each isolator consists of laminated layers of natural rubber and steel plates. The rubber was specially compounded to resist oxidation because the building is on the seashore. Of the total number of bearings used, 32 bearings are 40 centimeters (16 inches) in diameter with an effective rubber thickness of 4.8 centimeters (1.9 inches). The remaining 20 bearings are 50 centimeters (20 inches) in diameter with an effective rubber thickness of 5 centimeters (2 inches). Without the isolation, the period of the structure was calculated to be 0.30 seconds, and under the design earthquake, the maximum elastic response acceleration was calculated as 0.61 g's. With isolation, the period is 0.76 seconds and the acceleration is reduced to 0.33 g's.

3.7 CRUAS-MEYSSE NUCLEAR POWER PLANT, FRANCE

The Cruas Power Plant structures are similar to those of the Koeberg NPP in South Africa. The complex consists of four reactor units and the associated safety-related buildings. Cruas NPP is located in the alluvial plain of the Rhone Valley river near Cruas-Meysse, France. The safety-related buildings are supported on a common mat to form a nuclear island. The nuclear island is supported by approximately 2,000 reinforced neoprene bearings (D'Appolonia, 1979) which are about 30 inches square in plan and about four inches thick. Each elastomer pad is an element of three layers of neoprene and 12-millimeter (0.5 inches) steel plates.

The seismic design criteria were governed by small magnitude near field earthquakes resulting in a peak ground acceleration equal to 0.2 g. The fixed-base frequencies of the buildings ranged from about 4.5 Hertz to about 10 Hertz. Accordingly, significant amplification of ground motion was expected, and in order to reduce this, the base isolation concept was implemented in the design. Base isolation also permitted the use of standardized design at this site. The base-isolated frequency of the nuclear island is about 1.0 Hertz. The maximum displacement capacity of the pads is 3.0 inches, significantly higher than the estimated maximum displacement for the design earthquake, one inch.

3.8 FOOTHILL COMMUNITIES LAW AND JUSTICE CENTER (FCLJC BUILDING)

The FCLJC building, located in San Bernardino, California, is the first major base-isolated structure constructed in the United States. The building is four stories tall and includes a mechanical penthouse and a full basement, as shown on Figure 2. The plan dimensions are 414 by 110 feet. The lateral load resisting system was originally a ductile moment-resistant frame and was changed to a steel-braced frame down to the first floor and a concrete shear wall from there down to the foundation.

Steel reinforced rubber bearings are placed under each of the 98 columns of the building. The bearings vary in size to balance differential settlement of the columns. Typically, these are 32 inches in diameter and 16 inches thick. The rubber from which the isolators are made is a chemically enhanced, highly-filled natural rubber with high damping and non-linear behavior. The shear stiffness of this rubber is high for small strains but decreases by a factor of four or five as the strain increases, reaching a minimum value at a shear strain of 50 percent. For large strains greater than 100 percent, the stiffness increases again. The damping follows a similar pattern, decreasing from an initial value of 20 percent to a maximum of 10 percent and then increasing again.

The vertical frequency of the overall system is about 10 Hertz; the horizontal frequency of the base-isolated structure approaches 0.5 Hertz (Way and Lew, 1986; Tarics et al., 1986). The isolation system was designed for a maximum credible earthquake of magnitude 8.3 that could be originated along the San Andreas Fault at 20 kilometers from the building site. The design spectrum is similar to the one proposed by the NEHRP recommendations for Seismic Area 7 (BSSC, 1986).

The study reported by Way and Lew (1986) shows that the fixed-base frequency of the original space frame structure was 0.9 Hertz. The frequency of the isolated brace frame building is 0.5 Hertz. The base shear was originally 0.8 g and was reduced to 0.35 g by the base isolation system. Acceleration at the top of the building was reduced from 1.6 g to 0.4 g. The maximum anticipated displacement was 12 inches for the non-isolated building and increased to 15 inches in the isolated structure.

The FCLJC building is instrumented with 19 accelerometers by the California Strong Motion Instrumentation Program. Huang et al., (1986) describes the location of the accelerometers and the recording system. Accelerograms obtained during the October 2, 1985 Redlands earthquake are reproduced on Figure 16. Record of Channel 12 was obtained at the foundation below the isolators, while Record 9 was obtained at the basement level above the isolators. It is seen from the initial portion of the records that high-frequencies around 12 to 16 Hertz were filtered out by the bearings and were not transmitted to the upper levels of the structure. Design calculations reported by Reid an Tarics (1983) show that the building will deform essentially as a rigid body with very similar acceleration throughout its height. This is illustrated on Figure 17, from which it can be seen that the dominant period is approximately two seconds.

3.9 SEBASTOPOL, USSR

A demonstration building in Sebastopol in the Crimea has been built on steel bearings (Nazin, 1978). The building is a seven-story reinforced-concrete structure constructed on steel rocker bearings. The structure behaves like an oscillator with a three-second period. If conventionally-founded, the natural period of this structure would have been about 0.5 seconds. The building structure is reported to have experienced an earthquake in 1977 and performed satisfactorily. The bearings are steel ovoids that force elevation of the building when it is laterally displaced, producing a gravity restoring force (Kelly, 1986).

3.10 SCHOOL BUILDING, MEXICO D.F., MEXICO

This is a five-story school building owned by the government of Mexico, D.F. The structure consists of reinforced concrete frames, some of them infilled with masonry walls and was completed in 1974. This type of construction is commonly used in Mexico and other Latin American countries for school facilities. They have suffered severe damages in different earthquakes, especially when partially infilled frames result in short columns susceptible to shear failures. The isolation device is shown on Figure 18 and consists of two horizontal steel plates with about 100 steel balls that can roll between them. The balls are about one centimeter (0.4 inches) in diameter and are contained in a group by steel rings. The isolators are tied to their supported footings by a system of cables that limits their displacement to eight centimeters (3.1 inches).

No damage occurred in this building during the 1985 Mexican earthquake; however, the building is in the region of stiff soil of the city where the damage was also minimal in conventionally-founded buildings.

3.11 OILES TECHNICAL CENTER, JAPAN

This five-story reinforced-concrete frame building, used for research offices, is the largest base-isolated structure in Japan. Plan dimensions are 36 by 30 meters (118 by 98 feet) with columns on 6.0 and 9.0 meters (20 and 30 feet) centers. A section of the building is shown on Figure 19.

The base isolation system uses cylindrical reinforced rubber bearings with a lead core. Typically, the pads are 70 centimeters (28 inches) in diameter with a core of 14 centimeters (5.5 inches) in diameter. The first fixed base frequency of the building was 4.1 Hertz and was reduced to 1.1 Hertz with the inclusion of the bearings.

The building was analyzed using five records of actual earthquakes and three artificial records with different scaling factors to have peak ground velocities ranging from 5 to 75 centimeters/second (Miyasaki et al., 1986). For a peak ground velocity of 75 centimeters/second, the maximum floor acceleration was 1,170 centimeters/second^2 for the non-isolated buildings and it was reduced to 223 centimeters/second^2 by the base isolation system. The maximum bearing displacement was calculated to be between 11.8 and 24.4 centimeters (4.6 and 9.6 inches) with a maximum bearing shear strain between 50 and 100 percent.

3.12 THE CITY AND COUNTY BUILDING, SALT LAKE CITY, UTAH

The City and County Building in Salt Lake City, Utah is a 93-year old structure. After studying several reinforcement options, base isolation was proposed to seismically upgrade this building because it constitutes a less destructive option to conventional retrofitting procedures. This is the first major rehabilitation work planned in the United States and construction was scheduled to commence in mid-1986. The building has five main floors and a 12-story clock tower; plan dimensions are 130 by 170 feet. The structural system consists of unreinforced brick and sandstone bearing walls. The tower is approximately 40 feet square in plan at its base, built of unreinforced masonry (Walters et al., 1986).

A total of 504 bearings are proposed to isolate the building. The bearings are spaced 4.0 to 8.0 feet. The bearings will be 15 inches thick and 16 to 19 inches in diameter. The final design will depend on the selection of the bearing types. The base-isolated frequency of the building is specified to be higher than 2.0 seconds. The design maximum free-field ground acceleration for the site has been determined to be 0.20 g. The corresponding spectral acceleration for the non-isolated structure is estimated as 0.55 g and it is reduced to approximately 0.09 g using base isolation (Forell/Elseseer, 1986).

3.13 UNION HOUSE BUILDING, NEW ZEALAND

This is a 12-story building with plan dimensions of 24.5 by 25.5 meters (80 by 84 feet) constructed in Auckland, New Zealand in 1982 (Boardman et al., 1983). The superstructure is a three bay by three bay reinforced concrete frame laterally stiffened by steel diagonals encased in concrete. The underlying soil is a soft layer of marine deposits and hydraulic fill. Sandstone formations are found about 10 to 13 meters below grade level.

The columns are supported in the sandstone by means of 12 meter (39 feet) long piles which are separated from the ground by surrounding cylindrical steel cases. The bottom ends of the piles have hinges that provide them with additional lateral flexibility. Energy dissipation is provided by steel plate cantilever dampers. The structure and the base isolation system are schematically shown on Figure 20. The isolated building has a period on the order of two seconds.

This building illustrates a successful application of the base isolation concepts to sites having layers of soft soil with underlying layers of rock.

4.0 ISSUES IN THE SELECTION OF BASE ISOLATION

As in any design, diversity forces a choice. Even within the traditional mold, the options for the building system are many and varied. The choice of the optimum solution is generally made in conference with the architect, structural engineer, geotechnical engineer, and the seismologist. As new materials present themselves and are eventually absorbed into construction use, the traditional mold loses significance, or at least has to expand its definition to include concepts that these new materials have obviated.

The advent of new materials for the fabrication of bearings and energy dissipators has brought to bear their potential in the design of earthquake-resistant structures. The successful use of these products in other areas, but nevertheless to mitigate vibration problems in general, has encouraged their use in reducing earthquake damage. In this respect, they have expanded the possible options available to the engineer. Depending on one's perspective, their use may be considered as a deviation or merely an expansion of the traditional role. The authors prefer the latter perspective.

An engineer facing the design of a building in a seismically hazardous area has many choices to make before a cost-effective solution to the earthquake-resistant construction is reached. The first choice is whether the building under consideration should be conventionally reinforced or should be base-isolated. If base isolation is selected, should the engineer design his own system or use one of the patented systems? If the use of a patented system is chosen, which system should be selected? Finally, what structural issues specifically related to the base isolation of choice need special attention to reasonable guarantee successful performance of the entire structural system?

Obviously, the design process is dynamic. The elements of design interact with one another and extend also to the tasks performed prior to the selection of a scheme to mitigate earthquake effects, such as establishing the structural design strategy, architectural features, and siting. Both initial and life-cycle costs are an important consideration in the design process also. Previous limited studies (Tarics, 1982; Vaidya and Eggenberger, 1984) have indicated that base isolation can be provided at an equal or lower initial cost compared to conventional approaches. Decreased risk is seen to favorably affect life-cycle costs. For these reasons, it is anticipated that base isolation will rather quickly emerge out of the research realm into practice, if it has not already. The decision process, referred to as the "anatomy of the decision" by Rigney (1986), will become a routine exercise for the engineer.

Accordingly, before moving on to the description of the various types of base isolation systems, the following paragraphs are devoted to discussing procedures that may be used to establish if the concept should be considered in the first place. The need for this first important decision admits the possibility of an inappropriate use of base isolation. Indeed, it should be obvious that base isolation is not a panacea that could provide earthquake protection irrespective of the type and use of the building structure. In some cases, it may be ideally suited; in others it may be only marginally suited; and in still others it may actually degrade the performance of the building. Therefore, the judicious selection of buildings and isolation systems is a major step toward assuring that base isolation will actually enhance the seismic performance and, hence, may be considered as a viable alternative approach for upgrading structural design.

In general, the seismic design of structures consists of two quite separate and independent tasks:

1. Determine or estimate the response of the structure to the earthquake ground motions. This includes accelerations, forces, and displacements.

2. Design the structure and size the structural elements to resist the forces and meet displacement criteria.

The results of both these tasks depend upon the seismic ground motion characteristics on the one hand and the structural system on the other. Very broadly, the significant issues to be considered in the design decision are:

- Seismic Hazard;
- Site Conditions;
- Physical Dimensions of Structure;
- Construction Materials;
- Facility Importance;
- Soil-Structure Interaction;
- Lateral Load Resisting Systems; and
- Inelastic Behavior of Structure.

The first two of the above issues pertain to the severity of the potential seismic ground motion and the last six pertain to the structural system.

4.1 SEISMIC HAZARD

It is apparent that construction in zones of low seismic hazard will not justify the additional cost of base isolation. Depending upon the building's physical dimensions, other loads such as wind may govern the design of lateral force-resisting elements. In such zones, conventional techniques for upgrading structural design to account for small earthquake forces are certainly the preferred approach. However, in areas with a slightly greater seismic hazard, such as defined in the UBC for Zone 2 and perhaps Zone 3, reinforced elastomeric pads without additional energy absorption, wind loads permitting, may be a cost effective alternative. For regions of relatively high seismic hazard, base isolation with an effective energy dissipation may be the only way to reasonably assure continued operation of the facility during and following a major earthquake.

Seismic ground motion characteristics at a site are affected by the magnitude of the seismic event, the proximity of the site to the source, and the site conditions. Ground motion is strongest close to a fault and its severity diminishes with increasing distance in accordance with attenuation characteristics dependent on the crustal structure through which the seismic waves propagate. High frequency ground motions attenuate faster than low frequency motions. Therefore, ground motions in the near field of an earthquake contain predominantly high frequency motion while ground motions at significant distances from the source may contain predominantly low frequency motion. Empirical relations between magnitude, distance, and predominant frequency content have been developed (e.g., Seed, et al., 1968; Joyner and Boore, 1982) on the basis of strong motion recordings. Figures 21 and 22 illustrate examples of such relationships. These concepts can also be illustrated directly with real time histories.

Acceleration, velocity, and displacement time histories recorded for the El Centro, 1940 earthquake (magnitude = 6.3) and the Taft, 1952 earthquake (magnitude = 7.7) are shown on Figures 23 and 24, respectively. The El Centro record was obtained at a distance of less than 10 kilometers from the source while the Taft, 1952 earthquake was recorded at 43 kilometers. The peak ground motion parameters for these are presented in Table 2, and the response spectra of these two records are shown on Figures 25 and 26. By inspection of Table 2 and the response spectra, it can be seen that the Taft, 1952 record contains significant long period motion compared to the El Centro, 1940 record. The implications of this for a base-isolated structure are obvious. Considering that the structure to be isolated is relatively rigid (predominant frequency of three to ten Hertz), base isolation is more suitable if the relatively high frequency ground motion, such as recorded at El Centro in 1940, was expected at the site. In the case of Taft, 1952 record, the lower frequency motion would be potentially amplified due to the predominant frequency of the base-isolated structure, making it difficult to control the displacement response. In other words, base-isolated structures may be affected more by large, distant earthquakes than conventional structures at the same locations. If the likelihood of encountering predominantly low frequency motion is high, the use of only base isolation may not be appropriate, as detuning may demand an unrealistic control of displacement. The use of an energy dissipating system in this case may be more appropriate.

4.2 SITE CONDITIONS

Site subsurface conditions also affect the type of seismic ground motion experienced by the structure. The soil column overlying bedrock can amplify the bedrock motion in accordance with its characteristic frequency. Thus, a ground motion record obtained on soft soils exhibits a significantly lower frequency content than a record from a stiff soil or a hard rock site. Figures 27 and 28 show the mean and the mean-plus-a-standard-deviation response spectra, respectively, for four different site conditions varying from hard to soft (Seed et al., 1974). These response spectra reflect the influence of site conditions on the predominant frequency content of ground motion. The stiff to hard sites exhibit frequency content in the range of about three to ten Hertz, while the soft sites exhibit a predominant frequency content in the one to two Hertz region. The implication of this is that base-isolated structures are also affected more by soft site conditions than conventional structures.

In usual design practice (for conventional structures), the above issues regarding ground motion are considered only if conditions, such as the importance and characteristics of the structure, dictate the use of site specific design criteria. In most cases, standard design spectra, e.g., Newmark-Hall (1982) or ATC 3-06 (BSCC, 1986), are used in a dynamic analysis to estimate response quantities. For example, Newmark-Hall (1982) constructs an elastic spectrum on the basis of normalized ground motion parameters of acceleration, velocity, and displacement equal to one g, 48 inches per second, and 36 inches, respectively. Amplification factors shown in Table 3 are applied on these normalized ground motion parameters to obtain the elastic ground response spectrum (Figure 29). The amplification factors are derived on the basis of statistical averages of response spectra from several recorded ground motions.

4.3 SOIL STRUCTURE INTERACTION

Soil-structure interaction may be important for massive and rigid structures such as those of a nuclear power station. For relatively light structures, soil-structure interaction is small, even when these structures are conventionally-founded. Light structures are characterized by relatively high fundamental frequencies in the soil-structure interaction modes. The effect of soil-structure interaction on base-isolated structures has been studied by Constantinou and Kneifati (1986), who conclude that soil-structure interaction is insignificant when the ratio of fundamental fixed base frequency to the base-isolated frequency is greater than 15. For other, more flexible structures, soil-structure interaction is insignificant for those modes for which the coupled system frequency is greater than 0.95 times the base-isolated frequency. In other words, soil-structure interaction is important when the base-isolated frequency is about the same as the soil-structure interaction frequency. A broad implication here is that if soft soils prevail, base isolation may not be appropriate.

4.4 STRUCTURAL SYSTEMS

The physical dimensions, construction materials, and the primary lateral load resisting systems also determine the applicability of base isolation and the type of system that could optimize performance. The facility importance and the extent and type of damage that can be tolerated are also significant in assessing the applicability of base isolation concepts.

Structural systems used to resist seismic lateral forces can be classified as:

- Moment-resisting space frames,
- Braced frames, and
- Shear walls.

In some cases, combinations of these systems have been used. Space frames resist the earthquake forces by the bending action of the columns and beams. They are characterized by relatively large deflections and low natural frequencies of vibration. Shear wall and braced frame buildings are normally rigid in comparison and are characterized by small lateral deflections.

Because base isolation reduces the predominant frequency of a structure, it follows that it may not be suited for relatively flexible structures which may have a low frequency to begin with. Structural systems which use moment-resisting space frames are generally more flexible than those using shear walls. Similarly, taller buildings are more flexible than shorter buildings. Table 4 presents, for example, the first three fundamental periods of a seven-story and a 30-story building, both using space frames for lateral resistance. The fundamental period of the 30-story building is 3.0 seconds, which is considerable longer than the fundamental period of 0.9 seconds for the seven-story building. Base isolating the 30-story building may decrease its acceleration response in the second and the third modes, but will potentially increase the displacement response in the first mode. This building may not be a suitable candidate for base isolation. The seven-story structure, on the other hand, could potentially benefit if the period of its fundamental mode is increased from 0.9 seconds to say 2.0 seconds. If the corresponding increase of displacements falls within tolerable limits, base isolation may be chosen as a viable design strategy for the seven-story structure.

Because shear wall buildings are more rigid, they are generally more suitable candidates for base isolation. When conventionally-founded, they are characterized by small deflections and high frequencies (in the range of about 3.0 to 10.0 Hertz). However, if the height to width ratio becomes large (typically, more than about an eight or nine-story building) overturing may become a problem. When base-isolated, the acceleration response of these structures is significantly improved. Although the total displacements also increase, these are generally not a problem because most of the displacement occurs in the base isolation system.

4.5 INELASTIC BEHAVIOR

In general, seismic design of structures permits some non-structural damage for smaller seismic events (with a return period of 50 years or so) and some structural damage for major events. Reflecting this philosophy, conventionally-founded structures are generally not designed for forces represented by the elastic response spectrum associated with a major earthquake. Some allowance for inherent ductility in the structural components is made and the elastic spectrum is reduced to an appropriate inelastic spectrum. Lateral force requirements in most building codes, e.g., the UBC and SEAOC, incorporate this reduction by adopting a response reduction factor, R_w (SEAOC, 1985).

Excursions into the inelastic range of material properties (ductility demand) during a seismic event and the resulting degradation of stiffness dissipate the earthquake energy imparted to the structure. The extent of the excursions into the inelastic range reflects the expected damage and also the potential for secondary effects resulting from large displacements. An upper limit of ductility demand is therefore imposed to minimize these effects. Indeed, the ductility demand depends upon the type of structural system and the materials of construction. Typical component ductility demands are presented in Table 5. Similarly, response reduction factors to elastic design limits also depend upon the structural system used. Typical values suggested by SEAOC are presented in Table 6. High values of R_w indicate a higher energy absorption capacity and low values of R_w reflect non-ductile structural systems.

In zones of relatively low seismic hazard, any of the structural systems may be used, perhaps without recourse to significant ductility demand. In zones of high seismic hazard, where potential inelastic behavior is relied upon, base isolation may be considered to reduce or eliminate the extent of the inelastic behavior and the consequent damage. The question then is, if base isolation is considered, what is the appropriate structural system? This question can be fully answered only with a complete knowledge of the project specifications, including building use, allowable displacement, allowable damage to contents, feasibility of construction, etc. In a way, base isolation may be looked upon as a means to reduce the seismic effects on a superstructure from those corresponding to a high seismic zone to those for a lower seismic zone. From this point of view, the structural system may be chosen in the usual manner appropriate for the lower seismic zone. An important difference, however, must not be overlooked--the base isolation system forms an integral part of the overall structural system and will interact with the dynamic characteristics of the superstructure.

Given a structural system, it is not quite straightforward to compare conventional and base-isolated designs. Lower forces than the expected linear response are justified in a conventional design on the basis of available ductility, but this position may not be justifiable in the base-isolated case, whereby the very design, significant inelastic behavior of the structure is precluded. By the same token, the risk of damage to the structural components in the conventional design is higher than in the base-isolated design. The relevant questions to be considered in the decision are then, for a similar risk of damage or loss of function due to seismic hazard:

- Does the base isolation system reduce the seismic forces on the structure to the same extent as does the potential non-linear structural behavior?

- Is the additional cost to provide base isolation justified?

Potential applications of base isolation require that the design risks be compared at least qualitatively with those associated with conventional design. For example, even in a zone of high seismic hazard, if the conventionally-designed structure can tolerate significant structural damage both in precluding collapse and in discontinued facility operation, base isolation may not be economically feasible. However, if the facility has to remain operational during and after a severe earthquake, then even some potential additional cost of base isolation may be justified for the resulting reduced risk.

4.6 QUALITATIVE SELECTION PROCEDURE

The following discussion is directed toward screening potential candidate structures for base isolation. It is qualitative in nature and should be supplemented with case-by-case quantitative evaluations in the event that, on the basis of this qualitative procedure alone, base isolation cannot be clearly eliminated as a strategy.

Without encumbering the discussion with specific details at this point, but keeping in mind that what is being attempted is isolation by detuning, energy dissipation by some damping mechanism, or usually some combination of the two, the flow chart on Figure 30 presents the major issues to be considered in the design process. Several details that must be considered in each step are included in the flow chart presented on Figure 31 and are discussed in the following paragraphs.

It appears at the outset that base isolation is suitable for the following general situations:

- The site is located in a zone of high seismic hazard.

- The structure is not founded on soft soil.

- The building is low to medium height.

- The building has a relatively low shape factor ($H/L \leq 1$).

- The contents of the building are sensitive to high frequency vibration.

- The lateral load resisting system results in a rigid structure.

Architectural considerations lead to the selection of the structural system while siting and site conditions determine the seismic zonation and the applicable seismic coefficients. The procedure envisioned here begins with a preliminary conventional design, for example, on the basis of UBC (ICBO, 1985) or SEAOC (SEAOC, 1984). For this design, evaluate the dynamic properties, specifically the first mode frequency, structure mass, and stiffness. Estimate the level of the expected ductility demand in the structural elements in the event of the design earthquake and the maximum credible earthquake. If the ductility demand is acceptably small, then no further consideration of design options is necessary and the conventional design may be refined and the costs evaluated.

Generally, the ductility demand is influenced by the structural system, site seismic potential, and the predominant mode frequency. If the ductility demand is judged to be unacceptable, then either revise the conventional design or consider base isolation.

If the base isolation option is chosen, first estimate the required predominant mode frequency to reduce the seismic coefficient significantly. Establish the stiffness and damping parameters of the base isolation system. If the site soil stiffness is relatively low, about the same magnitude as the required base isolation stiffness, then base isolation may not be appropriate. If the site soils are firm, select and design the base isolation system. Evaluate the isolated structure dynamic parameters and compute system response. If the response is acceptable, refine the structure design and evaluate costs.

Generally, unacceptable response in the base-isolated system takes the form of excessively large displacements, and the attendant problems associated with the interface of base-isolated and non-base-isolated components of a building system. If the displacements are unacceptably high, revise the base isolation system design.

On the basis of the above qualitative discussion, some immediate observations can be made regarding the preliminary screening of structures for base isolation. Figure 32 presents a graphic relationship between the fundamental period of the conventional structure (T_C) versus the period of the same structure when appropriately base-isolated (T_B). In the T_C-T_B space, regions in which base isolation is clearly inappropriate can be identified. For example, base-isolated periods less than one second are inappropriate, as this would put the structure in the amplified region of the seismic spectra. Similarly, base-isolated periods larger than about 2.0 seconds may be undesirable because of the relatively large displacement response. These two criteria define Lines A-A and B-B. Further, it is anticipated that to derive benefit from isolation, the isolated period should be at least two times the period of the conventionally designed structure. This defines Line C-C. The region enclosed by Lines A-A, B-B and C-C thus represents more or less qualitatively the feasible combinations of the conventional structure period and the base-isolated structure period. Indeed, the feasible region can be further refined by considering cost issues.

It should be emphasized that Figure 32 should be used only in a qualitative manner and its use is further restricted to those situations where mitigation of earthquake forces is effected predominantly by isolation. In many cases, however, the functions of isolation and energy dissipation may not be clearly distinguishable. In the extreme case where only energy dissipation (e.g., friction) is used at the base, a base isolation frequency is undefinable. In the case of a highly non-linear base isolation system, an initial system stiffness is valid at best for small displacements, such as those produced by wind and ambient vibration. Defining a system stiffness for moderate displacements requires the knowledge of the displacements themselves, and even as such, a stiffness, and therefore a base-isolated frequency, can only be estimated.

As can be gathered from the above discussion, several issues need to be looked at simultaneously. Quantitative values of estimated response of various base-isolated structural systems would expedite the process. However, in most cases a simplified analysis treating the base-isolated structure as a single mass oscillator provides a good preliminary estimate of the response. A simplified approach for one particular type of base isolation system has been described by Kelly et al. (1986).

5.0 BEARING SYSTEMS

From the brief discussion about the design philosophy and what is being attempted, it follows that the "ideal" base isolation system will perform the following functions.

1. Minimize lateral loads on the structure and minimize the attendant relative displacements.
2. Safely support the vertical load of the structure.
3. Provide restraint against other environmental loads such as wind.

The primary elements of any base isolation system are isolation, energy dissipation, and restraint. When mitigation of the earthquake forces are effected predominantly by friction-based elements, either restricted to the base or dispersed through the structure, the design is sometimes called a passive energy dissipation system. In essence, this may be looked upon as increasing the inherent damping in a structure by enhancing the elements that cause damping in the first place. Thus, very broadly, passive energy dissipation acts in quite a different manner than does isolation. Although these methods are discussed here, their broad scope, applicability, and design variations make it necessary to restrict the discussion to only those types that are located in the base of the structure. In this context, they are treated much the same as base isolation in which the requirements for energy dissipation have taken on the greater significance.

Of the three elements that constitute a base isolation system, restraint assumes a relatively subsidiary role; its use is seen as an afterthought to supplement the performance of the former two elements, namely, isolation and energy absorption. Isolation and energy absorption are then the significant elements engineered into the overall structural strategy, thus deviating from the traditional approach, their main tenet being to reduce the destructive horizontal forces and motions imparted to the structure. Pure isolation attempts to achieve this by changing the frequency response of the structure (detuning), while energy absorption relies on dissipating the earthquake energy at the structure/foundation interface.

In most currently promoted bearing systems, neither isolation nor energy absorption is solely used at the exclusion of the other; rather, these two elements are combined in a manner to produce synergism and obtain the greatest reduction of destructive horizontal motions. Various combinations have been proposed; some actually have been implemented for aseismic design; several have been tested in a laboratory environment to substantiate their claim at reducing seismic lateral loads; and most have been subjected to mathematical analysis to illustrate their potential benefits. Oversimplifying here, only for the sake of clarity, it can be stated that the different ways in which the elements of isolation and energy absorption are combined result in different systems. Consequently, one is not that radically different from another, as there are only two basic elements to be combined, and if we keep this in mind and look at all systems in this light, it is not too difficult to understand the expected performance of each.

The following paragraphs briefly describe the bearing devices and their anticipated performance in isolating the structures they support. The objective here is to briefly present proposed, analyzed, tested, and implemented systems and highlight those which appear at the present time to be the most advanced with respect to their use and study.

In accordance with the concept used, base isolation schemes can be categorized as structural, geotechnical, and specially-engineered systems installed between the structure and its foundation.

The Imperial Hotel built in Tokyo in 1921 provides an example of one of the geotechnical methods. The foundation of this hotel building consists of short piles that extend only as far as a soft layer of soil underlying eight feet of stiff soil. Thus, the structure floats on the soft layer and is isolated by it against seismic ground motion.

Examples of the structural method are the flexible first-story concept (Arnold, 1984) or the sleeved pile concept (Figure 20) (Boardman, et al., 1983; Biggs, 1982). In the sleeved pile concept, each foundation pile is enclosed in a casing which allows lateral movement of the pile inside the casing. Additional structural members are provided at the first floor elevation both as energy dissipators and to prevent column/pile buckling between the first floor and bedrock. Indeed, this length could vary according to site conditions, and the safe load to prevent buckling may dictate relatively large pile dimensions.

Although the geotechnical and structural methods such as described above come under the general category of base isolation, they are excluded from the scope of the present report. These may be looked upon and treated as special structural and geotechnical designs.

Among the specially engineered systems, there are various types that are designed to support the vertical load of the isolated structure and provide horizontal flexibility. The prevalent schemes can be categorized according to:

- Function;
- Materials; and
- Damping mechanism.

For example, the functions of supporting the vertical load and providing horizontal flexibility may be performed by a single element or separately by two elements. Most prevalent base isolation schemes use reinforced elastomer bearings, but other concepts employ a low frictional interface which provides the isolation. Additional devices for energy dissipation may be included in parallel with the primary isolating element or in series with it. The energy dissipation devices themselves can incorporate frictional interfaces, plastic deformation of metals, or devices which extrude softer materials when deformed. Table 7 presents the various proposed analyzed and tested base isolation systems, identifying in each case the components which perform the required functions. Not all systems presented in Table 7 have been implemented. The various systems are discussed in some detail below. For the following discussion, the systems are separated into those based on reinforced elastomers and those based primarily on frictional interface.

5.1 ELASTOMER SYSTEMS

Elastomer is a generic term used to denote rubber-like materials. The two most commonly used elastomers for bearings are natural rubber (polyisoprene) and neoprene (polychloroprene). Elastomers in general are characterized by their high deformability. This is due to their chemical structure which, at the molecular level, comprises long, very regular polymer chains. Elastomers have been in use for over 50 years or so and have a good track record in other applications such as vibration isolation and isolation against thermal movements.

5.1.1 Plain Elastomer Bearing

The simplest isolation device is a plain elastomer bearing block such as shown on Figure 33. Bearings of this type have been used to isolate a three-story building in Skopje, Yugoslavia (Seigenthaler, 1970). In the absence of reinforcing laminates, the elastomer block tends to bulge sideways under vertical load and results in a low vertical stiffness. The shear capacity of an elastomer block is directly proportional to its thickness; the vertical load capacity is inversely proportional to it. Typically, the vertical load on a plain elastomer bearing is about 75 kips.

The disadvantage of the plain elastomer bearing is that for the thickness of elastomer required to obtain isolation from horizontal seismic motion, the vertical stiffness is relatively low--about the same order of magnitude as the horizontal. It is, therefore, difficult to sustain gravity loads of the structure without undergoing significant vertical displacements. Further, because of similar horizontal and vertical bearing stiffnesses, a significant amount of undesirable coupling between the seismic horizontal and rocking motions of the structure is anticipated.

5.1.2 Reinforced Elastomer Bearing

In the mid 1950's, Freyssinet conceived the idea of improving the performance of the elastomer bridge bearings by reinforcing them with steel plates. In this type of bearing, alternating layers of steel plates and elastomer sheets are bonded in a vulcanization process. A typical bearing of this type is shown on Figure 34. Today, such bearings are commonly used in bridges.

As shear capacity of the bearings depends upon the total thickness of the elastomer, these bearings retain the horizontal flexibility while their vertical stiffness is significantly increased by the presence of the steel plates. Again, natural rubber and neoprene are the commonly used elastomers for bearings of this type and typically carry a vertical load in the range from about 500 to 1000 kips.

It is a common practice to designate an elastomer by its hardness. Hardness is related to the resistance encountered when attempting to produce deformation by a specially shaped indentor under a specified load. Hardness is expressed as relative values on a scale having a maximum of 100. Typically, the hardness for elastomers used in bearings ranges from 50 to 70 Durometer A hardness. The engineering properties useful in the design of bearings are shown in Table 8 for two different hardness values.

Additionally, the behavior of elastomers under load is typically non-linear, even for moderate displacements. Consequently, they dissipate energy through hysteresis. If no special compounding is used, elastomer bearings of this type typically provide an equivalent viscous damping of about 5 percent. Such bearings have been used for seismic isolation of several structures. An example is the safety-related structures of the Cruas Nuclear Power Station in Cruas, France (D'Appolonia, 1979).

5.1.3 Reinforced Elastomer Bearings/High Damping Rubber

In several cases, the flexibility of the bearings required to provide seismic isolation results in relatively large relative displacements between the ground and the isolated structure. These displacements may be undesirable from the point of view of design (e.g., rattle space between isolated and non-isolated parts, utility connections, etc.). To control these displacements and limit these to tolerable values, it is necessary to provide means for energy dissipation, in addition to the inherent damping in the elastomer. Special compounding of the elastomer can improve its damping characteristics. For example, the Foothill Community Law and Justice Center building (Way, 1986) has reinforced elastomer bearings constructed of high-damping rubber. A typical bearing under this building is shown on Figure 35.

Damping in an elastomer results from two sources. One is due to the fact that elastomers exhibit non-linear load-deformation characteristics for relatively large strains on the order of 100 percent. The material itself remains elastic and, upon removal of the load, eventually returns to the initial strain. However, the unloading path is different from the loading and, therefore, results in a hysteretic energy loss. The second source of damping results from the visco-elastic nature of elastomers and manifests itself as a lag between stress and strain under dynamic load. This damping is frequency dependent and increases as the frequency of loading increases. Generally, at low frequencies of about 1.0 Hertz, viscous damping is small.

Figure 36 presents the shear load-deflection curve of a high damping (non-linear) rubber bearing installed under the Foothill Community Law and Justice Center Building in San Bernardino, California. It is evident from the figure that the energy loss due to hysteresis is significant for a strain of 40 percent, that could occur during a strong earthquake.

The advantage of this isolation approach is that a significantly high initial stiffness can be chosen to limit displacements for normal environmental loads such as wind or small earthquakes, which have a significantly higher probability of occurrence. When the displacements exceed a certain limit, the non-linear behavior results in a smaller modulus and a displacement-proportional damping. It should be noted that for events producing smaller displacements, the equivalent stiffness is higher and the hysteretic damping is small.

5.1.4 Reinforced Elastomer Bearings with an External Energy Dissipation Mechanism

In this scheme, reinforced elastomer bearings such as shown on Figure 34 are supplemented by special energy absorbing devices. Conceptually, this combination may be as shown on Figure 37. This figure illustrates a mechanical energy dissipator which uses plastic deformations of a torsion bar to absorb energy.

At relatively low displacements, the mechanical energy dissipator remains elastic and its stiffness is relatively high compared to that of the elastomer bearing. Consequently, for ambient loads such as wind and other environmental vibrations such as small earthquakes, the building remains non-isolated and behaves in the same manner as a conventionally-founded building. As the displacement response increases for larger earthquakes, the device yields at a predetermined force, and in this manner acts like a mechanical fuse. The post-yield stiffness, and therefore the frequency, is then determined predominantly by the reinforced elastomer bearing. The yielded mechanical device supplies energy absorption through hysteresis in undergoing inelastic distortions. The dissipator force acts in parallel with the restoring force of the elastomer bearing.

Several specific devices based on plastic deformation of steel and extrusion of lead have been tested (Skinner, et al., 1975; Steimer and Chow, 1984). Some of these devices are illustrated on Figure 38. The energy dissipating characteristics of a torsion bar system, for example, are shown on Figure 39 (Kelly et al., 1972). The torsion bar in this test is 1/2-inch square and one-inch long. For an angle of rotation of about 23 degrees, the energy absorbed per cycle is about 7,500 in pounds per in^3 and the torque developed is about 1,950 in pounds. After about 40 cycles, the developed torque begins to decay. The cumulative energy dissipation of 40 cycles is about 3.4×10^5 in pounds per in^3. Energy absorbing characteristics have also been developed for other devices. Several of such devices have been included in bridges in New Zealand. However, to the authors' knowledge, none so far have been incorporated in a scheme to isolate building structures.

5.1.5 Reinforced Elastomer Bearings with an Internal Energy Dissipating Mechanism

In this concept, the reinforced elastomer bearing includes a cylindrical hole in its center which is filled with lead. A cross-section through the bearing is presented on Figure 40. The reinforced elastomer carries the vertical gravity load and provides the horizontal flexibility. In undergoing shear deformations, the lead plug deforms plastically in shear and results in energy absorption, in addition to that provided by the elastomer itself.

As the lead is contained by the steel and rubber plates, it is forced to deform more or less uniformly in shear. Cyclic shear during earthquakes "hot works" the lead and, consequently, the lead recovers most of its mechanical properties during deformation. A typical shear load deflection curve for a lead-rubber bearing is shown on Figure 41. It is noted that the lead plug contributes significantly to the hysteretic damping; however, the lead plug also increases the horizontal stiffness of the bearing. Because the lead recovers its mechanical properties during deformation, the lead-rubber bearing performs very well under cyclic loading with very little degradation in its performance. When the shear load is removed, the reinforced elastomer provides a restoring force to return the bearing to its original configuration.

The William Clayton Building in Wellington, New Zealand is isolated by bearings of this type (Megget, 1978; Megget, 1984). Several bridges in New Zealand also use such bearings. In addition to sizing the overall bearing dimensions, the design of this type of bearing includes the proper sizing of the lead plug in accordance with the required bearing stiffness and damping for seismic isolation (Kelly et al., 1986).

5.1.6 Reinforced Elastomer Bearings with a Frictional Interface

In this concept, a frictional interface is incorporated between the bearings and the isolated structure they isolate (Richli et al., 1980). The concept is illustrated on Figure 42 which presents the base isolation foundation method used at Koeberg and the Karun River nuclear power plants in South Africa and Iran, respectively.

Each bearing consists of a reinforced elastomer pad mounted with a lead-bronze alloy plate which forms a friction couple with a stainless steel plate stiffened by a concrete slab and embedded in the upper mat of the superstructure. The materials for the friction couple were chosen to provide a constant friction coefficient close to 0.2 under a wide range of vertical bearing pressures and speed of lateral motion.

The behavior of the isolated structure during an earthquake is illustrated qualitatively on Figure 43. During moderate earthquakes, the structure vibrates on the bearings without slip and returns to rest in its original position. During more severe earthquakes with a peak horizontal acceleration exceeding 0.25 or 0.3 g's, the structure vibrates and slips on the frictional surface and eventually comes to rest with some residual displacement. In addition to limiting the seismic forces imparted to the structure, the frictional interface also limits the shear deformation of the bearings, and thus improves the factor of safety against buckling of the bearing under vertical load.

5.2 FRICTIONAL BASE ISOLATION SYSTEMS

5.2.1 The Alexisismon (Ikonomu, 1979)

In this concept of base isolation, the functions of supporting the vertical loads and providing horizontal flexibility are performed by two separate components. The concept is illustrated on Figure 44. It comprises the following primary components:

- A connecting element consisting of a vertical or a horizontally positioned steel bar designed to break at a predetermined force. This acts like a mechanical fuse.
- A supporting element consisting of a combination of rubber and teflon pot bearings.
- A rigid diaphragm which connects all bearing elements.
- An elastomer block which provides a horizontal restoring force on the foundation.

Because the vertical support element is separated from the element providing horizontal stiffness, this concept allows a relatively large horizontal displacement without compromising the vertical load-carrying capacity of the system. This may be comparable to the reinforced elastomer bearing with a frictional interface. When activated, the restoring force of the elastomer is parallel with the horizontal frictional resistance of the pot bearings.

No damping elements in addition to the frictional interface are included. In feasibility studies for bridge applications, lateral displacements of up to 40 inches were anticipated in the design of the system. Fundamental periods of structures isolated with this system are about five seconds. A three-dimensional Alexisismon has been proposed recently which provides for vertical isolation in addition to the principal horizontal directions (Ikonomu, 1984). In this configuration, viscous dampers are used to control vertical displacements.

5.2.2 The Resilient-Friction Base Isolator (R-FBI)

The resilient-friction base isolator is comprised of a stack of flat steel rings which enclose one or more cylindrical rubber cores. The concept is illustrated on Figure 45. Teflon sheets bonded to the steel rigs provide a frictional interface with the steel surface of the adjacent steel ring. Thus, the frictional characteristics of teflon are combined with the resistancy of elastomers. The vertical load is supported primarily by the stack of flat steel rings and the horizontal flexibility is provided by the low frictional coefficient and the enclosed elastomer cores. Damping or energy absorption is supplied through friction.

For wind loads, environmental vibration, and for small seismic ground motion, the friction prevents slippage and the superstructure responds as a non-base-isolated structure. When the ground motion exceeds a certain limit, slip occurs and the structure is isolated to the extent that the force-deformation characteristic of the core and friction will allow.

The static coefficient of friction for the teflon/steel interface is smaller than the dynamic and, also, the friction coefficient increases with sliding velocity. This prevents any potential stick-slip phenomenon. Because the horizontal stiffness is provided primarily by the enclosed elastomer cores, the predominant horizontal mode frequency of the base-isolated structure is more or less independent of the gross bearing dimensions and these dimensions can therefore be chosen to assure safe support of vertical loads regardless of earthquake size.

Although no structures have been built using the R-FBI, this concept has undergone significant testing and analysis (Mostaghel and Flint, 1986; Mostaghel et al, 1986). A representative load-deflection relationship for a test specimen is shown on Figure 45.

5.2.3 The Earthquake Barrier System

This concept relies primarily on sliding friction to provide a force barrier. The barrier physically consists of a friction assemblage with a predetermined friction-slippage level attached to the structure at its base. One suggested assemblage uses two perpendicular sliding rails operating independently. A horizontal diaphragm restrains the torsional motion of building columns which are supported by the friction assemblage. Hydraulic dampers or neoprene shear springs may be added in parallel with the friction assemblage to enhance the performance of the barrier systems. Analytical as well as experimental work has been performed on this system (Caspe and Reinhorn, 1986).

5.2.4 Steel Roller Bearings

Steel roller or ball bearings can provide the vertical load-carrying function with an independent device used to restrain or limit the horizontal motion. A four-story reinforced concrete frame building in Mexico, D.F. is supported on roller bearings. Each bearing consists of 100 one-centimeter diameter steel balls. Horizontal restraints are provided by steel limiting cables (Gonzalez-Flores, 1964). In the event of severe ground motions, the limiting cables may need an additional energy dissipation device.

5.2.5 The Coil Spring Concept

Perhaps comparable in simplicity to the plain elastomer pads, this concept uses coil springs to mount the structure on the foundation soils. This concept was studied to base isolate an existing five-story, wood-frame building at the Naval Post Graduate School in Monterey, California (Reed and Kircher, 1986). The disadvantage of this system is that the horizontal response is strongly coupled with the rocking response of the isolated structure. This concept has been further expanded and studied in the GERB System (Huffmann, 1985).

5.2.6 Friction Pendulum System

The friction pendulum system (FPS) uses the pendulum concept for reducing the frequency of the responding system together with a frictional surface to provide coulomb damping. The building behaves as a non-isolated structure until the frictional resistance is overcome. The isolated frequency of the structure is determined by the radius of curvature of the frictional surface.

Like other isolation systems, the FPS reduces the earthquake forces on the structure with the attendant increase in total displacements. Shaker table tests suggest that the total displacements reflect an effective viscous damping of about 5 to 10 percent. The base-isolated frequency, and consequently the effective isolation, is inversely proportional to the square root of the radius of curvature of the frictional surface. For increasingly larger radius of curvature, the isolating frequency approaches zero. Consistent with this concept, high frequency motions are transmitted during small earthquakes which produce forces insufficient to exceed the friction. Prior to causing slip, the seismic forces may also produce overturning moments on the FPS connections. Like other friction-based systems, the FPS exhibits a permanent set.

Several shaker table tests have proved the feasibility of the concept. In actual practice, however, potential long-term degradation of the frictional surface may be cause for concern. Periodic inspection of the FPS may alleviate some of this concern.

5.3 OBSERVATIONS

Table 9 presents a summary of base isolation components used in some systems. The reinforced elastomer bearing is the common denominator in the more developed schemes. In terms of the additional damping mechanisms, the reinforced elastomer bearings with the frictional interface, special elastomer compounding, and the internal lead plug appear to be the most developed from the point of view of analysis, confirmatory testing, and practical application for the seismic isolation of building structures.

Based on the case histories, test, analytical data, and actual usage, the following observations are made:

- Base isolation is a sound and viable alternative to conventional seismic design.

- Reinforced elastomer bearings with or without:

  - High-damping rubber (e.g., FCLJC building),
  - A frictional interface (EDF system; e.g., Koeberg Nuclear Plant), or
  - A lead plug (DIS system; DIS, 1984).

  have been developed extensively and may be considered for improving the seismic performance of buildings and other structures.

Of the remaining bearing types, a few have been extensively tested (e.g., R-FBI system) and analyzed, but may only lack the advantage and experience of use.

Typically, the bearings are about 15 to 30 inches in plan dimensions and 5 to 15 inches in thickness. The bearings carry a vertical stress of about 400 to 2,000 psi. The vertical stiffness is generally about 500 times the horizontal. The horizontal stiffness of the bearings ranges from about 2.0 kips per inch for light structures to about 40 kips per inch for heavy structures. The number of bearings comprising the isolation system generally results in a base-isolated fundamental frequency ranging from about 0.5 Hertz to 1.0 Hertz. Natural rubber and neoprene are the most commonly used elastomers for these bearings. The material properties of these elastomers, as they apply in the design for base isolation, and the mechanical properties of the finished bearings have been discussed by Vaidya and Bazan (1987).

If the earthquake ground motion is known, the parameters of each of the three bearing types recommended above can be chosen such as to result in the desired structural response. And from this point of view, each is comparable to the others in expected seismic performance. Advantages and disadvantages of the bearing types should, therefore, be considered from the point of view of the inevitable uncertainties in ground motion and structure, the relative cost of bearings to offset the potential effects of these uncertainties and design provisions to obtain a more-or-less uniform factor of safety. Table 10 compares the most important performance characteristics of the bearings. To appreciate the differences, Table 10 should be reviewed along with the following qualitative discussion.

The high-damping bearing and the lead-rubber bearing exhibit a significant non-linearity in their load-displacement relationship. This results from special rubber compounding in the former and a lead plug in the latter. The compounding and the size of the lead plug determines the degree of hysteresis. Compared to the reinforced elastomer bearing alone, the energy adsorption is greatly improved.

The advantage of these systems is that the compounding or size of the lead plug is a design feature and accordingly can vary with a building structure. In the extreme, some bearings under the same structure may not have any additional damping.

The high-damping rubber bearings and lead-rubber bearings are both characterized by high initial stiffness. At small shear strain, however, the stiffness is drastically reduced as shown for example on Figures 36 and 41. The isolation frequency is generally estimated by the secant modulus of the load-deflection curve. Since the secant modulus depends upon the bearing displacement, it follows that larger earthquakes will result in smaller apparent stiffness. For small earthquakes, therefore, the bearings may transmit higher frequency motions to the building structure. Although these are not seen to be a problem for the structure, their impact on light equipment and their supports may need to be reviewed. For large horizontal displacements, the bearings may develop potential instability while supporting large vertical loads.

Although, in general, permanent building displacements are likely to be somewhat smaller than in the EDF system, they may not be zero. In a single bearing which is displaced in the horizontal direction, the forces in the lead plug and the elastomer may be in equilibrium and thus under no external load the bearing may assume a distorted shape. Eventually, however, the restoring force of the elastomer and motions in the tail region of the earthquake time history will return to the bearing. The mechanism by which this takes place is not clearly understood, but apparently depends upon the rubber compounding and the size and stiffness of the lead plug and the magnitude of the restoring force due to the elastomer.

Large horizontal displacements result in significant localized strain fields in the elastomer (especially at the bearing edges). The general failure modes can be a separation of the bond between the elastomer sheets and the reinforcing plates or significant flexural deformation of the reinforcing plates. Following a major event, visual inspection is, therefore, justified to assure the quality and future performance of these bearings.

Elastomer bearings with a frictional interface have been installed at the Koeberg Nuclear Power Station in South Africa. The safe shutdown earthquake (SSE) here is 0.3g, which in principle is the same as the maximum credible earthquake (MCE) in the design of important non-nuclear structures. Licensing experience suggests that a fail-safe mechanism is generally desirable to accommodate displacements larger than those computed for an MCE. These may result from the type and form of ground motions which can potentially differ from those used in the analysis.

The EDF system accommodates larger than anticipated displacements by providing a corresponding larger bearing area from the friction plate located on the underside of the supported structure. Indeed, the larger the bearing area, the greater is the cost, and, accordingly, this feature should be weighed in relation to acceptable risk. It is not just a matter of providing a larger frictional surface, but provisions also need to be included in the structural system to accommodate the resulting eccentric vertical loads and transmit them to the bearings.

The advantage of this system is that large displacements are taken up by the slip on the frictional surfaces. Thus, the elastic shear distortions in the bearing are limited. At the Koeberg and the Karun River Nuclear Power Plants, the elastic distortion was limited to a value equal to the effective elastomer thickness. Even this is larger than normally used by bridge engineers. This restriction increases the factor of safety against bearing instability from vertical loads. Conversely, bearing thickness can be reduced with a corresponding reduction in the cost of bearing fabrication.

Because the system is highly non-linear, especially for a maximum credible earthquake, the slip on the friction surface results in a permanent displacement of the structure. However, sequential jacking of individual bearings against the supported structure can reposition the structure to its original location. This operation can be performed without interruption of facility operation. Repositioning of a nuclear power station with 2,000 bearings and a permanent set of about 12 to 15 cms was estimated to take about three months. Aftershocks following the main event can further aggravate the permanent slip. Therefore, the bearing system must be designed to accommodate the cumulative slip.

Bearing design should be concerned with long-term performance of both the elastomer and the additional damping mechanism. In the authors' opinion, a satisfactory method by which to verify this is not available. The only recourse at the present time is to periodically test the bearings. Since removing bearings from under a structure is relatively expensive and time consuming, witness samples need to be installed in the same environment as the bearings. Periodic tests on these samples is a good way to provide a basis for predicting the in situ bearing properties.

6.0 DESIGN CONSIDERATIONS

The use of reinforced elastomer bearings is relatively commonplace in bridges where they have been used primarily to accommodate thermal movements of the bridge deck. Elastomer bearings have also been used for vibration and noise isolation for buildings and vibrating equipment. Indeed, the first such use dates back about 30 years. However, seismic application is relatively recent. Because of the particular dynamic interaction between building structures and ground motions, this application deserves a careful consideration from the point of view of the overall system performance. The following paragraphs discuss some of the more common design-related issues that need attention in a base-isolated structure. Tentative requirements for design proposed by the Structural Engineers Association of Southern and Northern California (SEAOSC and SEAONC) are also discussed.

The structural response of base-isolated structures is significantly different from that of conventionally-founded structures. Vertical load-carrying members, such as building columns and load bearing walls, are discrete structural elements tied together horizontally at floor elevations by the floor diaphragms. The lateral loads induced during earthquakes due to the structure mass are transmitted to the foundation by braced frames or shear walls which themselves contain the vertical load-carrying elements. Generally, lateral load resisting elements may also be discontinuous and discretely located within the plan dimensions of the building. The floor diaphragms are designed to appropriately collect the distributed inertia forces and transmit them to these discrete lateral load resisting elements.

The structural system must distribute both vertical gravity loads and horizontal seismic forces uniformly to the base isolation bearings. Most base isolation designs distribute the bearings in accordance with the arrangement of the vertical load-carrying elements such as column and load bearing walls. Usually, a rigid diaphragm is therefore imperative at the first floor to distribute the horizontal seismic loads to the discrete bearing elements. If the diaphragm is not adequately rigid, some of the bearings may be loaded beyond their design capacity. This issue is similar to conventional design where floor diaphragms have to be rigid enough to appropriately collect and distribute inertia forces and transmit them to discrete lateral load-resisting elements such as in a distributed shear wall system.

Special cases may require close attention to the design of the first floor diaphragm. Consider, for example, interior core walls such as those enclosing elevator shafts. These may form the principal lateral load resisting elements. In such a case, the first floor diaphragm should be able to distribute overturning moments, in addition to the horizontal base shear. For relatively large aspect ratios, the overturning moments and the resulting tension on at least some bearings may preclude the use of base isolation.

In most cases, vertical ground accelerations are smaller than the horizontal and are predominantly high frequency motions. Most prevalent base isolation systems isolate only the horizontal ground motions and provide no isolation against vertical ground motion. In cases where significant vertical motion is expected, the effect of this vertical motion on the horizontal response should be investigated.

In general, the response of base-isolated structures to vertical ground motion can lead to a non-uniform distribution of vertical load on the bearings during an earthquake. This results in variation in the horizontal load deflection characteristic of the bearings and can, therefore, affect the horizontal displacements. However, the variation of vertical load on the bearings is relatively rapid (corresponding to a vertical frequency of 10 Hz or more) in comparison to the horizontal response which is a low frequency phenomenon (about 1 Hz). This leads to a decoupling effect resulting in a relatively insignificant interaction between the horizontal and vertical motions. For most cases, therefore, the vertical ground motions have a minor effect on the horizontal response.

Accidental eccentricities may cause a torsional response resulting in larger displacements at the edges of the isolated building. For usual building proportions, accidental eccentricity of about 5 percent does not lead to severe torsional response. However, when the building dimensions are large, about 300 to 400 feet (on the order of a seismic wave length), propagation of surface waves may induce a torsional input. Similarly, for asymmetric buildings, the horizontal loads on the bearings must consider potential torsional response due to non-uniform mass distribution. Because most base isolation systems incorporate bearings with strongly non-linear load-deflection characteristics (for improved damping), prediction of asymmetric response requires the proper spatial representation of the bearing elements. If the building is more-or-less symmetric, the discrete bearing elements may be lumped together into a single base isolation element in the mathematical model.

Similarly, foundation conditions are important also. Soft soil conditions may result in some detrimental soil-structure interaction in the seismic response. In addition, building settlements may result in a non-uniform vertical load distribution. Again, the load-deflection characteristics of bearings depend upon the axial load on the bearing. In general, as the axial load decreases, the effective bearing stiffness increases and effective damping or energy absorption decreases. Variable soil conditions are especially difficult for which to reliably predict seismic response.

When a building foundation incorporates piles to a competent substratum, the seismic response may also be affected by the pile-soil-base-isolated structure interaction. In usual cases, the lateral stiffness of a pile foundation is relatively large so that its dynamic characteristics do not interact with the base-isolated structure. However, the top of all

When a building foundation incorporates piles to a competent substratum, bearings may be placed at the top of pile caps, and in this case the seismic response will be determined by taking account of the pile-soil-base-isolated structure interaction. The subject of pile-soil-structure interaction is still not completely understood. For example, the response is generally affected by the soil stiffness, pile design, pile type and stiffness, and whether load is transferred through point-bearing or side friction. In usual cases, however, the lateral stiffness of the pile foundation is relatively large so that its dynamic characteristics do not interact with the base-isolated structure. However, just as in the conventional construction, the top of all piles under a structure must be tied together to impart a more or less coherent horizontal ground motion to all parts of the base-isolated structure.

At the present time, longevity and long-term performance of elastomers is still not completely defined. Properties important to base isolation are the bearings' vertical load-carrying capacity and their capacity to deform in shear. Any potential stiffening of the elastomer with age obviously results in a degradation in the level of isolation provided. Although most long-term field observations suggest that the two most commonly used elastomers, namely neoprene and natural rubber, perform well, a systematic study of the degree of stiffening with age, environment, sustained loads, etc. is lacking. Therefore, maintaining access to bearings for inspection and potential replacement is a prudent practice to follow.

Bearing access is usually provided by installing base isolation bearings below the first floor in a basement area. Most utilities, pipes, sewer lines, etc. crossing the isolation boundary need special attention. Expandable or articulated joints should be installed in pipes to accommodate large relative displacements (on the order of about 10 to 15 inches). In general, a gap of about 10 to 15 inches is required between the base-isolated and the non-base-isolated building components, such as elevator shafts and stairwell walls, perimeter walls, etc.

components. These building motions may also become the more critical from the operability standpoint, if the building houses equipment which is particularly sensitive to high frequency motion.

Several older structures use masonry walls which are particularly sensitive to high frequency vibrations. Seismic upgrading of such structures should also carefully consider effects of small earthquakes for which base isolation systems may transmit higher frequency motions to the building components. Similarly, modern construction, which includes extensive use of glass, may also be susceptible to the high frequency motions that can produce non-structural damage.

Most problems associated with detail design such as discussed above are just that--problems of good detailing specific to the particular building under consideration and the particular design concept chosen. These issues can generally be resolved satisfactorily on a rational basis. In the final design stages, other issues such as the effect on the seismic response due to eccentricity, settlement of foundation soils, effects of vertical and rocking input motion, etc., should be investigated quantitatively to confirm the design assumptions. In most cases, these issues are secondary (Vaidya and Plichon, 1986), but depending upon the structure, they may be relatively important.

The components of the specific base isolation system, namely, the bearing and the energy dissipating device, should be carefully reviewed. Effects of manufacturing tolerances of bearings should be examined. Finished bearings should be tested to confirm their mechanical properties. Although elastomers such as neoprene and natural rubber have proven their durability in other applications, the long-term effects of the environment and aging remain ill-defined. Although accelerated aging tests on bearing samples provide guidelines and are usually required, these may not truly represent long-term behavior of installed bearings. Some means of inspection and testing of in-place bearings is desirable.

Actual experience with systems in place will, with time, supply required performance data to improve or refine the base isolation systems. Newer applications will perhaps present other issues that have not been discussed, but it appears that most issues can be solved on a rational basis.

6.1 TENTATIVE SEISMIC ISOLATION DESIGN REQUIREMENTS (SEAONC, 1986)

The SEAONC provides "general design requirement applicable to a wide range of possible seismic isolation schemes." The undertone of these guidelines appears to be simplicity in application. The engineering parameters that characterize the isolation schemes such as horizontal bearing flexibility, damping, vertical load capacity, etc. are required to be verified by mandatory tests.

For simple, uniform structures, the design displacement is described as a function of site and building characteristics.

$$D = \frac{10ZNST}{B} \tag{1}$$

where: Z = seismic zone coefficient (e.g., 0.3 for Zone 3, and 0.4 for Zone 4,
S = soil type coefficient (i.e., 1.0 for soil type S_1, 1.5 for soil type S_2, and 2.0 for soil type S_3),
T = isolated building period,
N = near-field coefficient (i.e., 1.0 for sites greater than 10 km from an active fault, 1.2 for sites within 10 km of of an active fault, and 1.5 for sites within 5 km of an active fault),
B = damping coefficient (e.g., 1.0 for 5 percent damped systems, 1.2 for 10 percent damped systems, and 1.5 for 20 percent damped systems).

and the minimum lateral force on the structural elements above the isolation interface are given by:

$$V_S = \frac{2k_{max}D}{R_w} \tag{2}$$

where: k_{max} = maximum effective stiffness of the isolation system determined by test,
D = design displacement, and
R_w = numerical coefficient specified in Table 1-G of the SEAOC (1985) for various types of structural systems, except R_w shall not be taken greater than a value of 8. (See Table 6).

For complex structures and exceptionally flexible buildings, more rational analyses are required. The rational analyses include response spectrum and time history methods. The design requirements represented by Equations 1 and 2 or by a rational analysis are based on ground motions representative of a severe seismic event with a return period of approximately 500 years.

The remainder of the "Tentative Seismic Isolation Design Requirements" is devoted to describing the required tests to confirm the isolation system parameters assumed in the design, and additional design requirements for the isolation system, the structural system, and the non-structural components.

The simplicity embodied in the SEAONC requirements should encourage the use of base isolation, and the committee has performed an excellent attempt at crystallizing several issues that affect the design of base-isolated structures. It is hoped that a designer will use the simple expressions with a clear understanding of the underlying principles.

The peak design displacement is described for a range of applicable values of T. However, for increasing values of T, the spectral displacement should approach the maximum ground displacement. The nearness factor N represents increased values of displacement close to a fault. This factor is based on typical ad/v^2 ratios for rock and soil sites combined with a correlation of v/a values with distance (Donovan, 1986), where a, v, and d are peak ground accelerations, velocity and displacements, respectively. The factor S in Equation 1 is included to represent potential amplification due to soft sites.

For sites close to a fault, the vertical accelerations and displacements are relatively significant and may be important for the design, particularly of base-isolated structures. Seismic analysis and design of the base isolation system and the structure should examine this issue.

Some inelastic deformations of the structural elements above the base isolation system are permitted by the SEAONC requirements. Accordingly, the base-isolated loads are further reduced by a modified ductility factor. This issue should be approached with some caution. First, by very design, base isolation attempts to preclude inelastic structural behavior, and hence, the mobilization of ductility. Second, inelastic structural behavior changes significantly the fundamental period of the structure and consequently, its dynamic interaction with the base isolation system. Third, depending on the pattern of inelastic distortion, the center of rigidity may shift and result in a relatively significant torsional response.

At the present state of development, it is a prudent practice to confirm simplified base-isolated design on the basis of a rational analysis. Indeed, SEAONC requires a rational analysis for a complex structure or an exceptionally flexible structure. Similarly, potential inelastic structural behavior should also require a rational analysis.

In addition to a testing program prior to the installation of the base isolation system, its components should be periodically inspected and tested for signs of deterioration and stiffening.

In most cases where the use of base isolation is justified, some manner of energy dissipation device is provided in addition to the reinforced elastomer bearing. This results in a strongly non-linear load-deflection behavior for the isolation system. The magnitude of displacements for which the system remains linear is small (on the order of about one inch or two). Unless it can be demonstrated that the entire system remains within the elastic limits, linear analysis methods, such as the response-spectrum technique, should be reinforced by time history methods which consider non-linear system behavior.

Recentering capability of the base isolation system should not be a requirement. This unnecessarily eliminates an important class of isolation systems which include a frictional interface. Indeed, frictional interface is provided to limit the forces in the structure as well as prevent bearing instability under excessively large horizontal displacements. Recentering is required so that the isolation system can accommodate potential aftershocks. Depending on the acceptable risks, the base isolation system can be designed to accommodate aftershocks in the time period required to recenter by external means. Eliminating one system for its lack of recentering capability and at the same time not requiring a corresponding reduction in the risk of bearing instability in other systems appears unjustified.

6.2 DRAFT GUIDELINES FOR THE DESIGN OF BUILDINGS WITH BASE ISOLATION (SEAOSC, 1987)

The Adhoc Committee of the Structural Engineers Association of Southern California has prepared Draft Guidelines consisting of requirements for the base isolation design of buildings. The guidelines discuss requirements related to the selection of seismic input, mathematical modeling, analytical procedures to compute seismic response, design, performance testing of base isolators, structural requirements, and in-service inspection of the base isolation system.

The undertone of these guidelines is caution. They are not as easy to apply to design as the requirements suggested by the SEAONC, but for this very reason, they are also quite straightforward and free from potential misinterpretation. At the outset, they appear to embody quite complex analysis not heretofore required of conventional buildings, and in this respect they penalize the base isolation concept. Upon some consideration, however, it is realized that:

- Base isolation is a potentially non-linear system, and therefore its proper non-linear representation in the mathematical model is necessary. Methods to linearize non-linear systems are available, but more development needs to be performed to study the methods and the sensitivity of the resulting response to parameters characterizing the non-linearity. It is indeed of benefit to incorporate the components that result in the non-linearity since they contribute to energy absorption and the attendant reduction of seismic response. From this point of view, the non-linear methods are justifiable, although more time consuming.

- Because base isolation reduces the participation of the isolated structure in the seismic response, generally it is sufficient to represent the structure only by a single mass on an appropriate non-linear representation of the base isolation system. This simplifies the mathematical model quite significantly, and the subsequent seismic analysis using non-linear time histories is simpler than appears at the outset.

- Especially flexible structures on base isolation systems or base-isolated structures in which inelastic response is allowed are discouraged. Degrading the stiffness of the structure, and therefore, its consequent interaction with the dynamics of the base isolation system are likely to make the results of a seismic analysis more difficult to interpret.

The requirements of seismic input recognize that a site may experience both near-field and distant earthquakes. Depending on the tectonic setting, one or the other will be the more predominant. Minimum relative displacements for design will be those caused by site-specific ground motions based on maximum credible earthquakes. "The time histories used in the analysis shall be selected and scaled to fit the site and source specific response spectra." Generally, a 20 to 30 seconds time history should suffice in the analysis, unless it can be shown that the maximum response displacement occurs at values less than 20 seconds. On the other hand, large distant earthquakes may produce significant motion of one to two minutes duration (e.g., some records of the 1985 Mexico earthquake). If such motion is expected, time histories of a longer duration may be necessary.

It is recognized that the proper performance of base isolation systems depends both on their components and the components of the structure which must distribute vertical and horizontal loads to the base isolation components. The guidelines discuss issues of structural considerations, e.g., a rigid diaphragm, effects of a fail-safe system, equipment anchorage, etc. The guidelines also outline a pre-installation testing program for the components of the base isolation system and an ongoing, post-installation inspection program. Field experience suggests that reinforced elastomer bearings (a component in most prevalent base isolation systems) perform very well with no sign of degradation. However, this experience is of a relatively short duration, and the bearing performance has not been subject sufficiently to the test of time. Hence, it is prudent to institute an in-service inspection program.

7.0 CONCLUDING REMARKS

This report has discussed the concept of base isolation for use in upgrading the earthquake performance of building structures. In this concept, the entire building structure is supported on flexible bearings located at the base of the structure, which separate the structure and the foundation soils.

Based on the design philosophy, case histories, and the base isolation system characteristics, qualitative criteria for selecting this strategy of seismic design have been developed. Although additional work is required to quantify the benefits and cost of base isolation versus the conventional design, it is possible to define potentially favorable applications. Conversely, situations where base isolation is not recommended can also be identified.

Although the most obvious benefit of base isolation can be achieved for new construction, the concept may also be applied to existing buildings. In cases where conventional seismic upgrading would be logistically difficult or would mar the architectural integrity of an essential existing building, base isolation could prove to be a viable alternative, and could, in fact, be the only means to improve the seismic performance of the facility. The potential for base isolating an existing structure, however, may be constrained by available space, the existing structural system, and materials of construction.

Most prevalent base isolation systems isolate the structure from horizontal ground motion only. Some base isolation schemes have been proposed in which isolation from both horizontal and vertical ground motion is provided.

Compared to conventional design, the effects of base isolation are to cause the structure to act as a rigid body during an earthquake and to reduce the following:

- The fundamental mode frequency.
- Forces and accelerations transmitted to the structure.
- Overturning moments on the foundation.
- Interstory displacements in the structure.
- In-structure forces on equipment and non-structural components.

In addition, the chances of structural and non-structural damages are also reduced.

Several structures have been constructed on base-isolated foundations for seismic protection. The characteristics of some of these structures have been discussed. Most of these structures are in the three- to ten-story range and use braced frame or shear wall construction. If they were conventionally-founded, they would be considered as relatively rigid (fixed base frequencies in the range from about three Hertz to ten Hertz). Analysis has shown that these structures benefit significantly from the reduced seismic response. Increased relative displacements due to base isolation were easily accommodated by design features.

Most base isolation schemes use reinforced elastomer bearings to support the vertical gravity load and to provide the horizontal flexibility for isolation. These reinforced elastomer bearings are comprised of either natural rubber or neoprene reinforced with steel plates. The plan dimensions of the bearings typically range from 15 to 30 inches and have a thickness from about 5 to 15 inches. The bearings are usually designed to support a vertical load of about 1,000 to 1,500 kips. Most systems include an energy-absorbing device, in addition to the elastomer bearings. Systems which have been implemented have incorporated one or more of the following forms of energy-absorbing devices:

- A frictional interface;
- A lead plug embedded in the bearing; and
- Special compounding of the elastomer.

The above systems have undergone numerous tests. Shake table tests have demonstrated their ability to effectively isolate the structure and to support its vertical load. Material tests, both during development and for specific projects, have demonstrated the performance characteristics of the bearings. It is concluded that natural rubber and neoprene possess excellent properties for this application, and their use in bridge bearings for more than 30 years demonstrate their long-term performance.

The relevant issues that must be considered in selecting this strategy have been discussed. The decision to select base isolation depends primarily upon the degree of risk the owner is willing to take--the risk of structural damage and of interrupting operation. Generally, when the ductility is acceptable a conventional design will compete well with base isolation design. Thus, in low seismic zones where no special design features need to be incorporated to increase ductility, conventional design will predominate. Similarly, in zones of higher seismic hazard, facilities which can tolerate damage and shutdown may not justify the additional cost of base isolation.

After reviewing important issues in the selection process such as seismic hazard, site conditions, subsurface conditions, architectural features and building use, structural systems used, and potential inelastic response, it is concluded that, on a qualitative basis, base isolation should be considered for the following situations:

- The site is located in a zone of high seismic hazard.

- The structure is founded on stiff soil or rock. Base isolation with pile foundations is feasible but requires careful analysis.

- The building is of low to medium height, i.e., no uplift should be permitted.

- The building has a relatively low shape factor ($H/L \leq 1$).

- The contents of the building are sensitive to high frequency vibration.

- The lateral load resisting system results in a rigid structure.

The response of the base-isolated structure depends significantly on the energy absorption qualities of the isolation system. The one-cycle hysteresis defined by the non-linear load-deflection characteristics represents the energy absorbed per cycle. In strongly non-linear systems, the degree of isolation is determined by the effective stiffness of the bearing element which in turn depends on the extent of displacement in the isolation system. Smaller displacements result in greater effective stiffness and smaller damping.

Specific design considerations arising due to the particular response characteristics of base-isolated structures are briefly discussed. It is suggested that these are problems of good detailing and are just as important as in conventional design. Provided that these are paid the attention they deserve, base isolation is a viable alternative to upgrading seismic performance of building structures.

Based on this research, the following areas are identified for further work, which should be directed towards developing quantitative guidelines for use in preliminary evaluation of base isolation as an alternate seismic design strategy:

- Quantify on a more uniform basis the characteristics and effects of linear bearing systems.
- Investigate effects of building types and structural systems on their base-isolated response.
- Investigate potential inelastic action in the base-isolated structure and torsional effects.
- Quantify effects of variability in earthquake ground motion and the variability in base isolation characteristics.
- Quantify risk of damage to facilities and operations so that representative values of safety factors may be assigned to their design.

The concept of base isolation is an important development in seismic design. Its use in the design of earthquake protection of structures, particularly those that tolerate minimal damage, should be considered at an early stage of building design along with other conventional methods.

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TABLES

TABLE 1

GENERAL CHARACTERISTICS OF EXISTING BASE-ISOLATED STRUCTURES

BUILDING	LOCATION	YEAR OF COMPLETION	NUMBER OF STORIES	PLAN DIMENSIONS	STRUCTURAL SYSTEM	SEISMIC DESIGN CRITERIA	ISOLATION SYSTEM
Heinrich Pestalozzi School	Skopje, Yugoslavia	1969	3	60 m x 13 m	Concrete frame		Unreinforced blocks of natural rubber
Elementary School	Lambesc, Marseilles, France	1978	3	77 m x 26 m	Precast concrete panels	Intensity VIII M.M.	Reinforced natural rubber bearings
Office Building	Athens, Greece	1980	3	14.5 m x 15.6 m	Reinforced concrete frame non-structural masonry walls	Time histories, peak ground acceleration .34 to 1.24 g	(Alexisismon) Pot bearing and elastomer block
William Clayton Building	Wellington, New Zealand	1983	4	97 m x 40 m	Reinforced concrete frame	El Centro, 1940 Earthquake .523 g peak	Reinforced natural rubber bearings with lead plug
Nuclear Power Plant	Koeberg, South Africa	1984		150 m x 100 m (Nuclear Island)	Reinforced concrete shear walls	0.3 g Site-specific spectrum	Reinforced neoprene bearings with frictional interface
Union House	Auckland, New Zealand	1983	14	25.5 m x 24.45 m	Braced reinforced concrete frame	El Centro, 1940 Earthquake	Long (12 m) flexible piles inside steel casings

TABLE 1
(Continued)

BUILDING	LOCATION	YEAR OF COMPLETION	NUMBER OF STORIES	PLAN DIMENSIONS	STRUCTURAL SYSTEM	SEISMIC DESIGN CRITERIA	ISOLATION SYSTEM
School Building	Mexico D.F., Mexico	1974	4	10 m x 30 m	Reinforced concrete frame with infilled masonry walls	0.1 g	Steel balls between steel plates
Demonstration Building	Crimea, USSR	1975 (est.)	7		Reinforced concrete frame		Steel ovoids
Foothill Communities Law and Justice Center	San Bernardino, California	1986	4	127 m x 34 m	Steel-braced frame, reinforced concrete shear wall	0.4 g site-specific spectrum	Reinforced high damping natural rubber bearings
Radioactive Waste Building	Torillon, France	1985	3	24 m x 13 m	Reinforced concrete frames and shear walls	0.3 g Site-specific spectrum	Reinforced natural rubber bearings
Nuclear Power Plant	Cruas-Meysse, Rhone Valley, France	1985		150 m x 100 m (Nuclear Island)	Reinforced concrete shear walls	0.2 g Site-specific spectrum	Reinforced neoprene bearings
Office Building	Athens, Greece	1985 (est.)	6		Steel MRSF	Time histories, peak ground acceleration .34 to 1.24 g	Pot bearing and elastomeric block

TABLE 1
(Continued)

BUILDING	LOCATION	YEAR OF COMPLETION	NUMBER OF STORIES	PLAN DIMENSIONS	STRUCTURAL SYSTEM	SEISMIC DESIGN CRITERIA	ISOLATION SYSTEM
Oiles Technical Center	Kanagawa, Japan	1986	5	36 m x 30 m	Reinforced concrete frame	Base Seismic coefficient 0.2	Reinforced rubber bearing with lead plug
Funabashi Taketomo Dormitory	Chiba, Japan	N.A.	3	15 m x 48 m	Frame and shear walls		Laminated elastomer bearings with viscous shear shear damper
Tohokn University Experimental Building	Miyagi, Japan	N.A.	3	6 m x 10 m	Shear walls	El Centro 1940, Taft 1952 EW	Laminated elastomer bearings with oil damping
High-Tech R&D Center	Tokyo, Japan	N.A.	6	16 m x 22 m	Frame	El Centro 1940, Taft 1952, Hachinohe 1968	Laminated elastomer bearings with hysteretic damper
Technical Research Institute	Tokyo, Japan	N.A.	2	12.5 m x 13.5 m		El Centro 1940, Taft 1952, Tokyo 1956, Sendai 1978	Laminated elastomer bearings with hysteretic damper
Miki Sawada Memorial Hall	Kanagawa, Japan	Under Construction	3	Hexagonal 226 m^2	Frame and shear walls	0.3 to 0.45 g	Laminated elastomer bearings with coil-type damper

TABLE 1
(Continued)

BUILDING	LOCATION	YEAR OF COMPLETION	NUMBER OF STORIES	PLAN DIMENSIONS	STRUCTURAL SYSTEM	SEISMIC DESIGN CRITERIA	ISOLATION SYSTEM
Tsukuba Research Institute Control Building	Ibaraki, Japan	N.A.	4	18 m x 19 m	Concrete frame and shear walls	0.3 to 0.45 g	Laminated elastomer bearings with coil-type damper
3 Residential Buildings	Saint Martin de Castillon, France	N.A.			Masonry walls		15 cm ø natural rubber pads
3 Residential Buildings	Beijing, China	1977, 1986	1		Masonry walls		Sliding friction on specially screened sand

TABLE 2

PEAK GROUND MOTION PARAMETERS

RECORD	MAGNITUDE OF EVENT	DISTANCE FROM SITE (km)	PEAK GROUND MOTION PARAMETERS ACCELERATION (cm/sec^2)	VELOCITY (cm/sec)	DISPLACEMENT (cm)
EL Centro, 1940, S00E	6.5	10	341.7	33.4	10.9
Taft, 1952, S69E	7.2	43	175.9	17.7	9.2

TABLE 3

SPECTRUM AMPLIFICATION FACTORS FOR HORIZONTAL ELASTIC RESPONSE

DAMPING	ONE SIGMA (84.1%)			MEDIAN (50%)		
% CRITICAL	A	V	D	A	V	D
0.5	5.10	3.84	3.04	3.68	2.59	2.01
1	4.38	3.38	2.73	3.21	2.31	1.82
2	3.66	2.92	2.42	2.74	2.03	1.63
3	3.24	2.64	2.24	2.46	1.86	1.52
5	2.71	2.30	2.01	2.12	1.65	1.39
7	2.36	2.08	1.85	1.89	1.51	1.29
10	1.99	1.84	1.69	1.64	1.37	1.20
20	1.26	1.37	1.38	1.17	1.08	1.01

Reference: Newmark and Hall, 1982.

TABLE 4

FUNDAMENTAL FREQUENCY VERSUS BUILDING HEIGHT

BUILDING[1]	FUNDAMENTAL PERIOD (SEC.)		
	MODE 1	MODE 2	MODE 3
7-Story Building	0.88	0.288	0.164
30-Story Building	3.0	1.0	0.56

Reference: Navfac P-3551 (1986)
1. Both buildings have reinforced concrete space frames.

TABLE 5

DUCTILITY DEMAND RATIOS OF STRUCTURAL COMPONENTS

BUILDING SYSTEM	ELEMENT	ESSENTIAL	HIGH RISK	OTHERS
Steel DMRSF	Beams	2.0	2.5	3.0
	Columns(1)	1.25	1.5	1.75
Braced Frames	Beams	1.5	1.75	2.0
	Columns(1)	1.25	1.5	1.75
	Diag. Braces(2)	1.25	1.5	1.5
	K-Braces(3)	1.0	1.25	1.25
	Connections	1.0	1.25	1.25
Concrete DMRSF	Beams	2.0	2.5	3.0
	Columns(1)	1.25	1.5	1.75
Concrete Walls	Shear	1.25	1.5	1.75
	Flexure	2.0	2.5	3.0
Masonry Walls	Shear	1.1	1.25	1.5
	Flexure	1.5	1.75	2.0
Wood	Trusses	1.5	1.75	2.0
	Columns(1)	1.25	1.5	1.75
	Shear Walls	2.0	2.50	3.0
	Connections (other than nails)	1.25	1.50	2.0

Reference: Army Corps of Engineers, 1986.

1. In no case will axial loads exceed the elastic buckling capacity.
2. Full panel diagonal braces with equal number acting in tension and compression for applied lateral loads.
3. K-bracing and other concentric bracing systems that depend on compression diagonal to provide vertical reaction for tension diagonal.

TABLE 6

SUMMARY OF SEISMIC RESISTING BUILDING SYSTEMS

	RESISTING SYSTEMS	
Conventional Systems (Resistance by Fracture or Yielding)	Timber Framing	7 to 9
	Masonry Bearing Walls (unreinforced)	1.5*
	Steel Framing with Masonry Wall Infill	2* to 8
	Steel Framing with Concrete Wall Infill	8
	Concrete Bearing Walls	6
	Concrete Non-Ductile Frame	2*
	Steel Braced Frame (SBF)	6
	Steel Ductile Moment Frame (SDMF)	12
	Concrete Ductile Moment Frame (CDMF)	12
	Dual Wall and DMF	12
	Dual Brace and DMF	9 to 10
Controlled-Yield Mechanisms (Resistance by Limited Yielding)	Progressive Resistance Systems (Wall/Brace/Frame)	6 to 12
	Coupled Concrete Shear Walls	6 to 8
	Slitted Concrete Shear Walls	6
	Precast Concrete Walls with Yield Connections	6
	Eccentric Steel Braced Frame (ESBF)	10
	Rocking Base (Walls or Frames)	--
	Yielding Steel Braces	--
High Damping Mechanisms (High Energy Dissipation)	Cable-Anchor Systems	--
	Friction Damped Braces	--
	Sliding Friction Base Systems	--
	Damped Cladding	--
Base Isolation Mechanisms (Decoupling Systems)	Soft Soil Mechanisms	--
	Soft Story Structures	--
	Torsion Bars or Flexure Beams	--
	Elastomeric Bearings	--

* not allowed

Reference: SEAOC, 1985

TABLE 7

BASE ISOLATION TYPES

BASE ISOLATION TYPE (Reference)	ELEMENT SUPPORTING VERTICAL LOAD	ELEMENT PROVIDING HORIZONTAL FLEXIBILITY	DAMPING ELEMENT	DAMPING CLASS	ANALYSIS	TESTS	IMPLEMENTED
High Damping Rubber Bearing	Reinforced Elastomer Bearing	Reinforced High Damping Elastomer Bearing	Special Compounding of Rubber	Hysteretic	yes	yes	FCLJC California
EDF (Koeberg)	Reinforced Elastomer Bearing	Reinforced Elastomer Bearing and Friction Plates	Friction Plates	Coulomb Friction	yes	yes	Koeberg, South Africa
EDF(Cruas)	Reinforced Elastomer Bearing	Reinforced Elastomer Bearing	Elastomer	5% Viscous	yes	yes	Cruas NPP, France
Lead-Rubber Bearing	Reinforced Elastomer Bearing	Reinforced Elastomer Bearing	Lead Plug	Hysteretic	yes	yes	William Clayton Building
Base Isolator Resiliant-Friction	Stack of Flat Plates	Elastomer Plug	Friction	Coulomb Friction	yes	yes	---
Alexisismon	Pot Bearing	Elastomer Block	Friction	Coulomb Friction	yes	--	Office Buildings, Athens, Greece
Earthquake-Barrier	Friction Assemblage	Elastomer Block	Friction	Coulomb Friction	yes	--	---
Low-Friction System	Steel Roller Bearings	Steel Roller Bearings	Steel Limiting Cables	Hysteretic	N.A.	N.A.	School Building in Mexico, D.F.
GERB	Helical Springs	Helical Springs	Viscous Dampers	Viscous	N.A.	N.A.	---
Seismafloat,	Reinforced Elastomer Bearings	Reinforced Elastomer Bearings	Elastomer	Hysteretic	N.A.	N.A.	Pestalozzi School, Yugoslavia

N.A. = Not Available

TABLE 8

ENGINEERING PROPERTIES OF ELASTOMERS

PROPERTY	ELASTOMER HARDNESS	
	60 DUROMETER	70 DUROMETER
Young's Modulus	540 psi	320 psi
Bulk Modulus	290,000 psi	290,000 psi
Shear Modulus	140 psi	95 psi
Elongation at Break		
Natural Rubber	450%	600%
Neoprene	350%	400%

TABLE 9

APPLICATIONS OF BASE ISOLATION ELASTOMERIC BEARINGS

STRUCTURE	BEARING SIZE	K_H (k/in)	K_V (k/in)	f (Hz)	VERTICAL STRESS (psi)	MATERIAL
Koeberg NPP	27.5 in. x 27.5 in.	25 - 40	13500	0.9	800	Laminated Neoprene and Friction Plates
Cruas NPP	20 in. x 20 in.	24.0	12500	1.0	1100	Laminated Neoprene and Friction Plates
FCLJC	30 in. diameter	6.0 - 9.0	3000 - 6000	0.5	1000 - 2000	Laminate Rubber (high damping)
William-Clayton Building	24 in. x 24 in.	8.0	3300	0.5	500	Laminated Rubber (with lead plug)
Experimental Building, Japan	18 in. diameter	4.0	4000		430	Laminated Rubber
Radioactive Waste Building, France	16 - 20 in. diameter	1.5 - 1.6	880 - 1300	1.3	700 - 800	Laminated Rubber
Oiles Technical Center, Japan	28 in. diameter	21	21000	1.1	830	Laminated Rubber (with lead plug)

Notes: K_H = Initial Horizontal Stiffness

K_V = Vertical Stiffness

f = Frequency

TABLE 10

COMPARISON OF PERFORMANCE CHARACTERISTICS OF SELECTED BEARINGS

	DAMPING	SHAKE TABLE TESTS DATA	BEARING TESTS	USE	BEARING STABILITY (P-Δ)	HORIZONTAL DISPLACEMENT CAPACITY	PERMANENT SET	POTENTIAL FOR BEARING REPLACEMENT
High-Damping Rubber	Provided by Rubber Hysteresis	Extensive	Extensive	Installed Under FELJC Building	Potential for Instability at Large Horizontal Displacements	Limited by Elastomer Tension	Small(4)	Possible
Elastomer Bearings with Friction Plates	Provided by Friction During Slip	Extensive(1)	Extensive(1)	Installed Under Structures of Two Nuclear Power Stations	Instability Avoided by Slip or Friction Plates	Virtually Unlimited	Moderate(3)	Demonstrated
Lead-Rubber Bearings	Provided by Lead Plug	Extensive	Extensive(2)	Installed(2) Under Three Buildings	Potential Instability at Large Horizontal Displacements	Limited by Elastomer Tension	Small(4)	Possible

1. Has undergone licensing review by nuclear regulatory authorities.
2. Has also been installed under several bridges.
3. Design allows centering capacity.
4. Centering may not be required if permanent set is small.

FIGURES

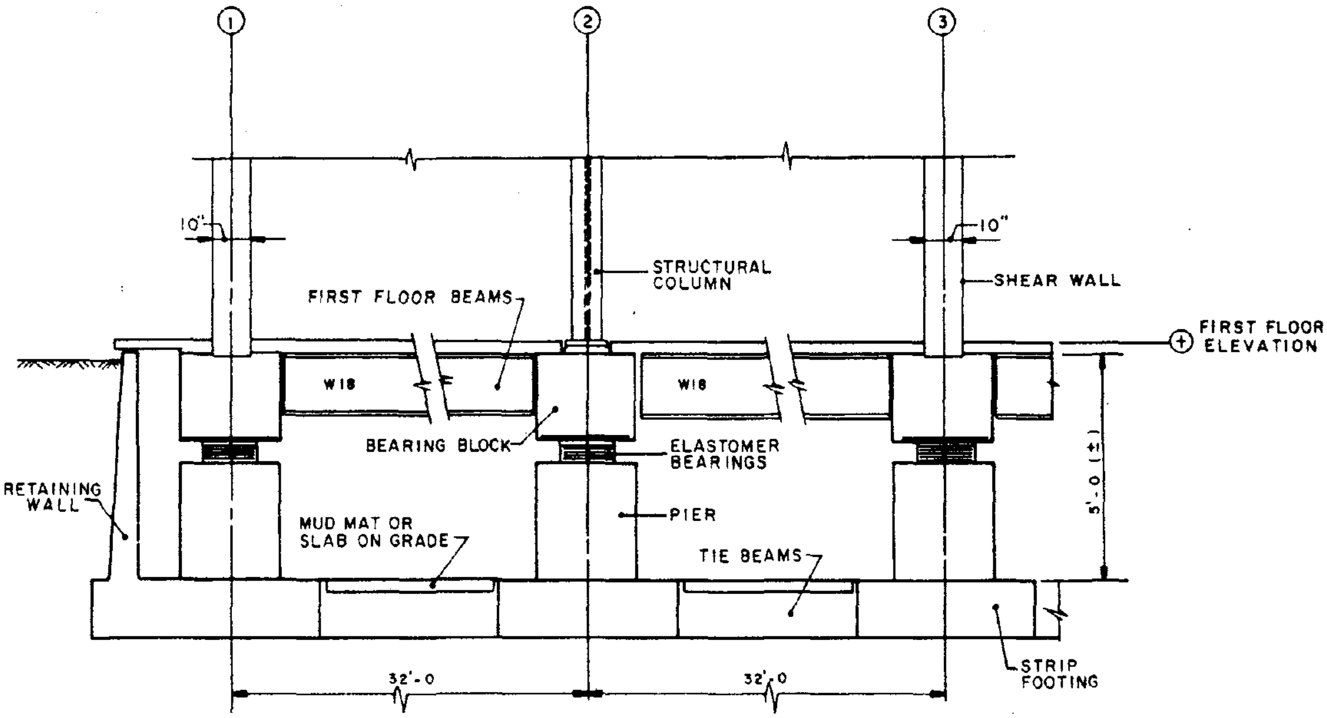
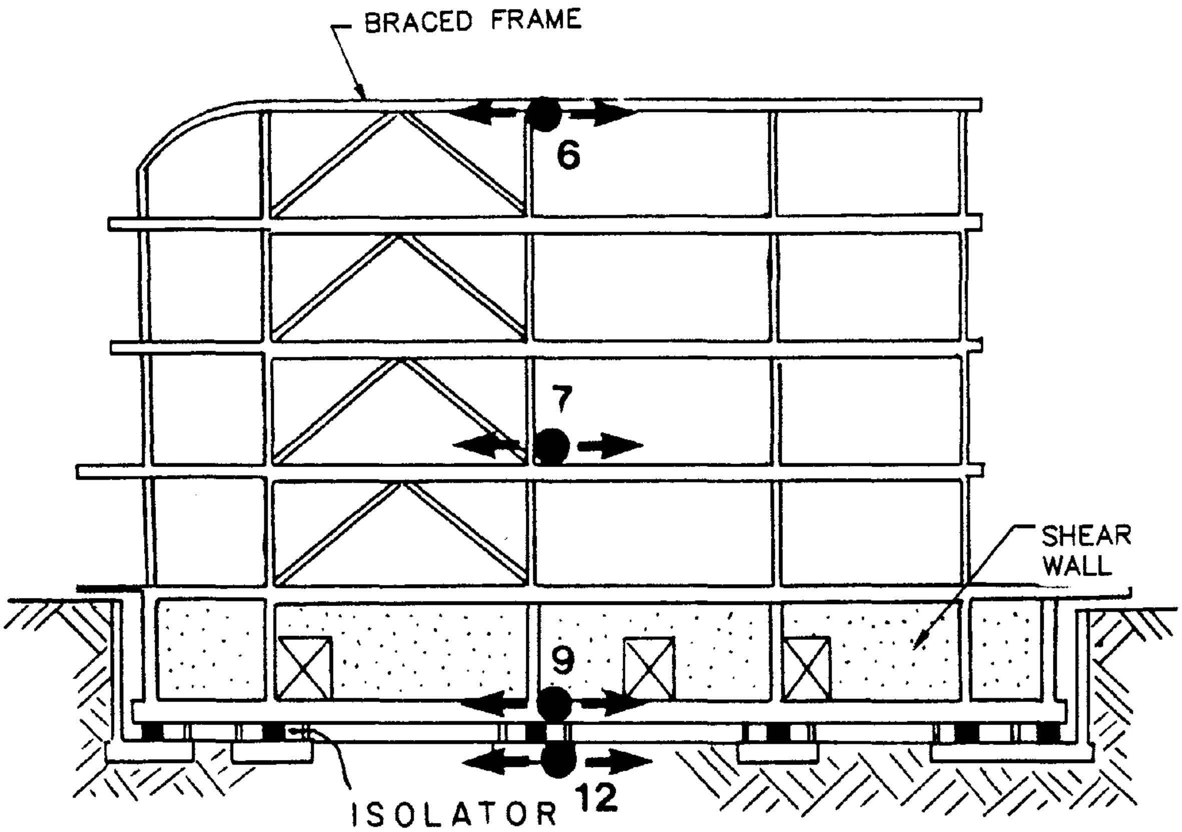
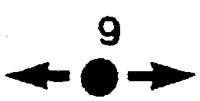


FIGURE 1
CONCEPTUAL DESIGN OF BASE ISOLATION

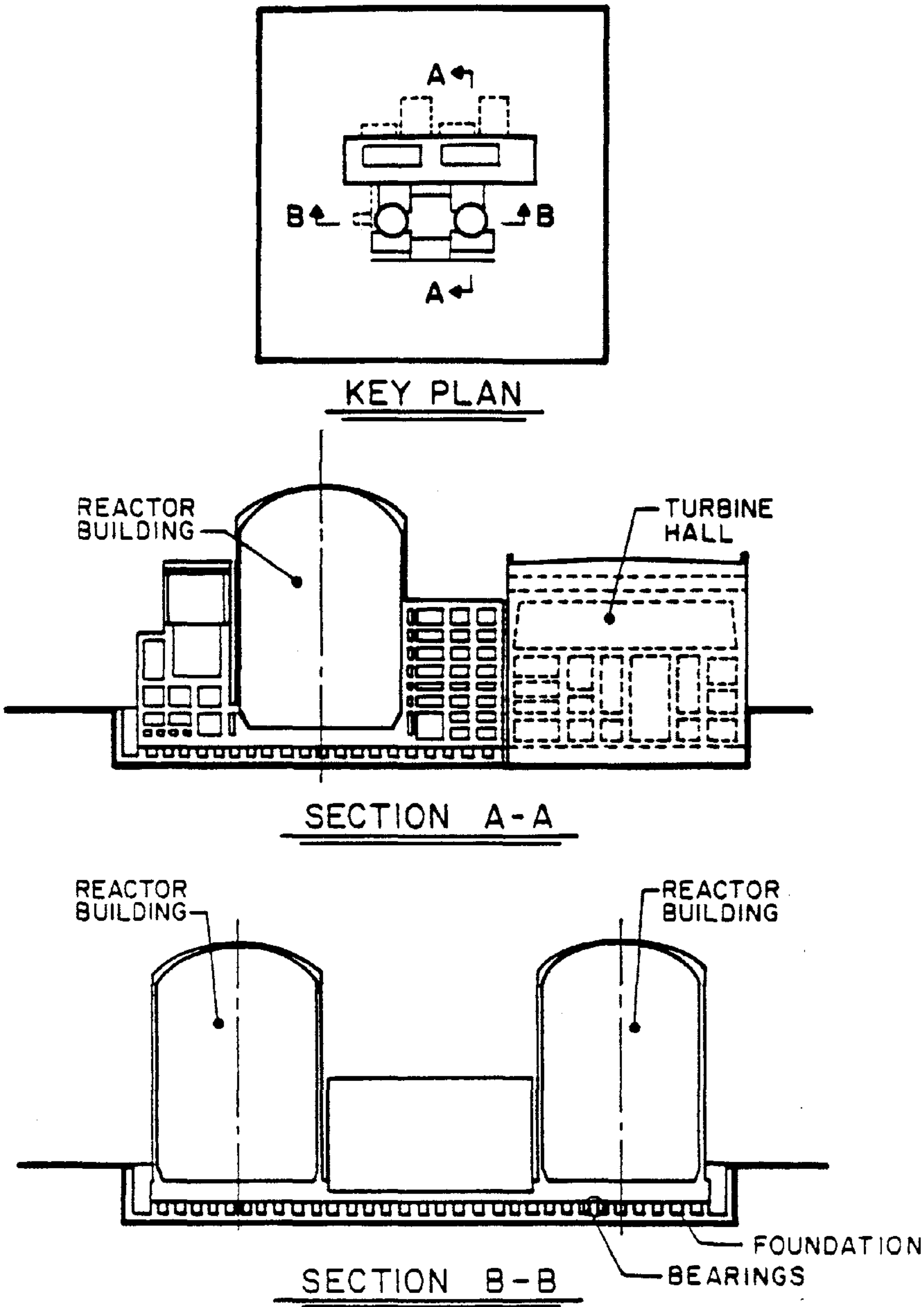


REFERENCE: HUANG ET. AL., 1986



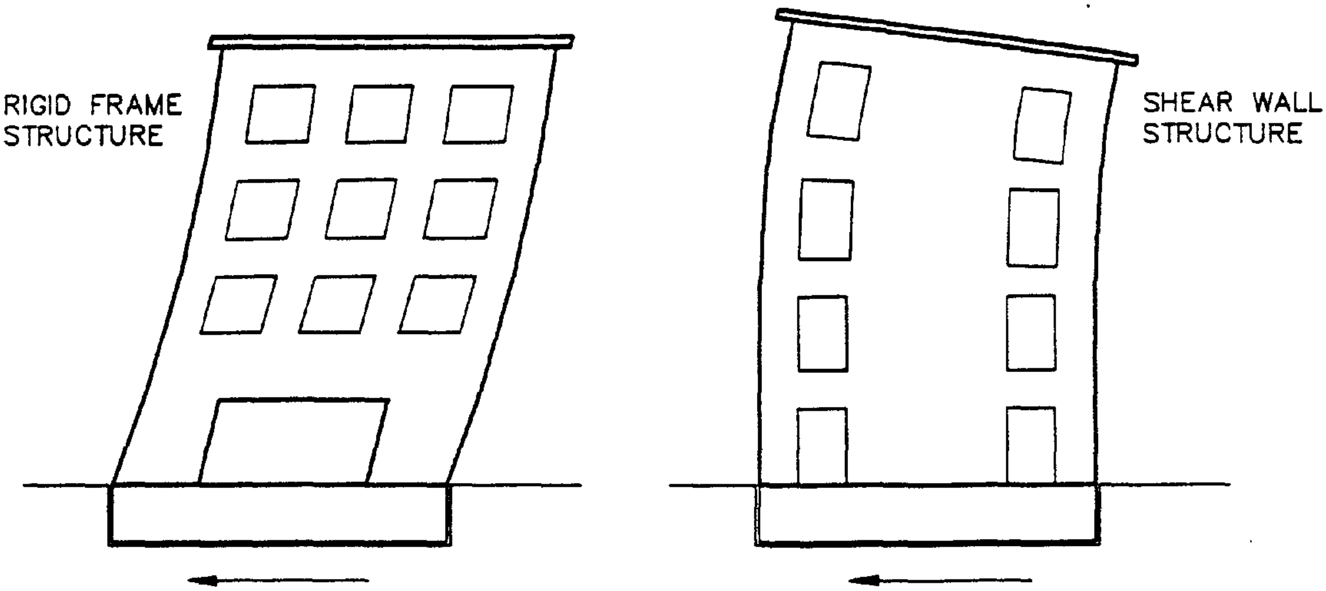
: LOCATION OF ACCELEROMETER

FIGURE 2
CROSS–SECTION OF THE FOOTHILLS COMMUNITY LAW AND JUSTICE CENTER BUILDING



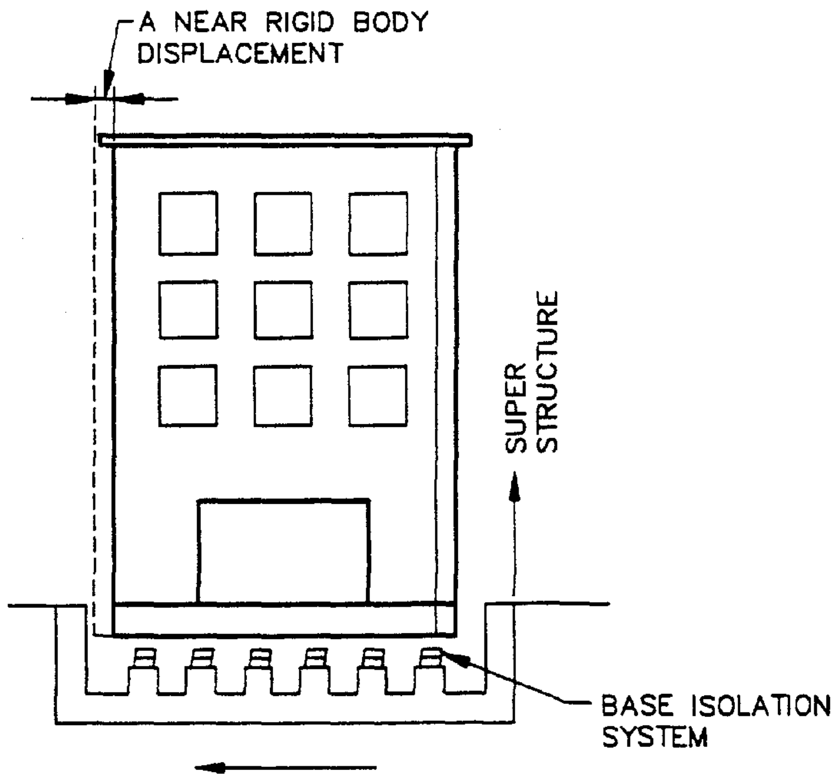
REF. JOLIVET & RICHLI, 1977

FIGURE 3
BASE ISOLATION OF THE KOEBERG NUCLEAR ISLAND



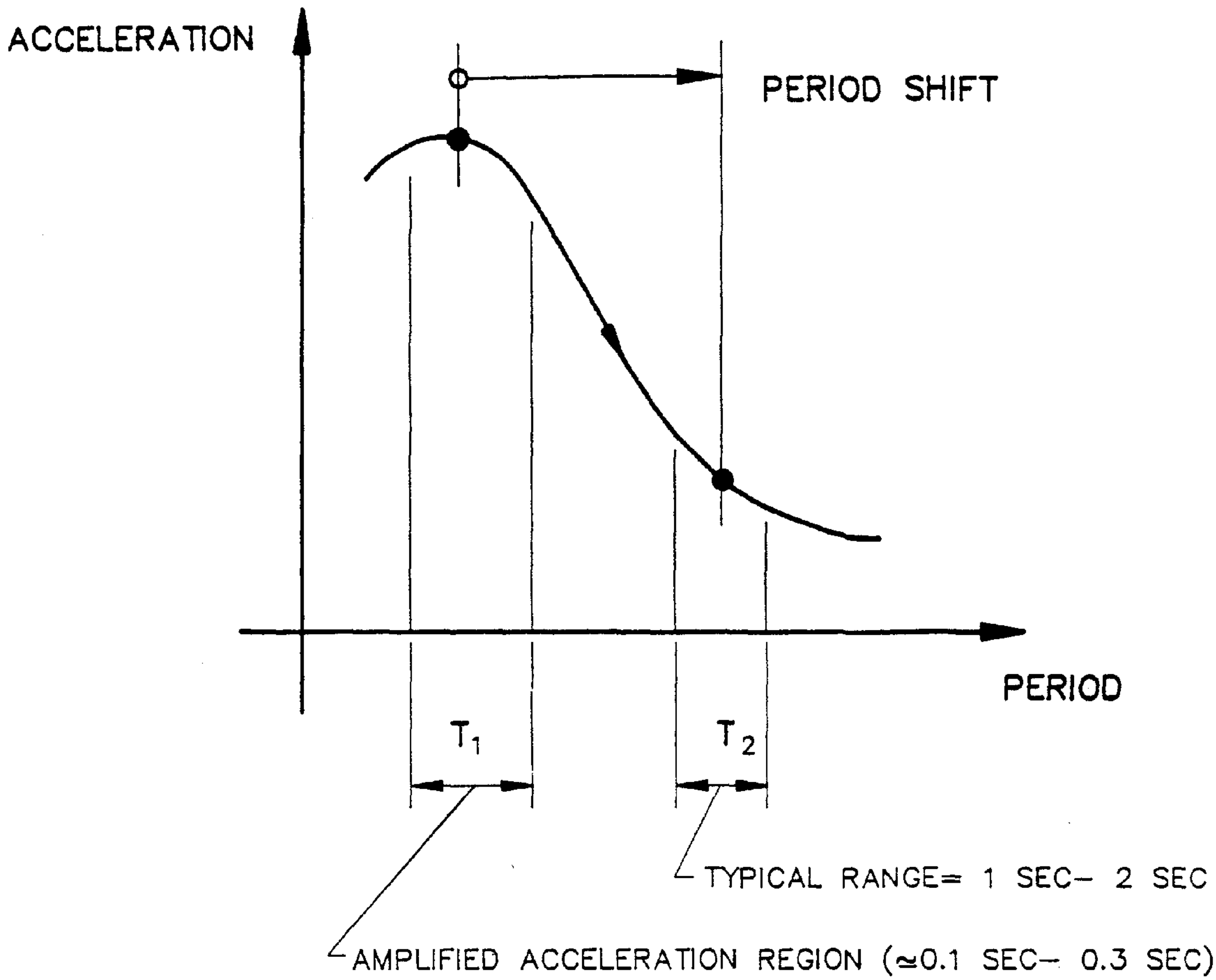
SHEAR DEFORMATION   BENDING DEFORMATION

RESPONSE OF CONVENTIONAL STRUCTURE



RESPONSE OF BASE ISOLATED STRUCTURE

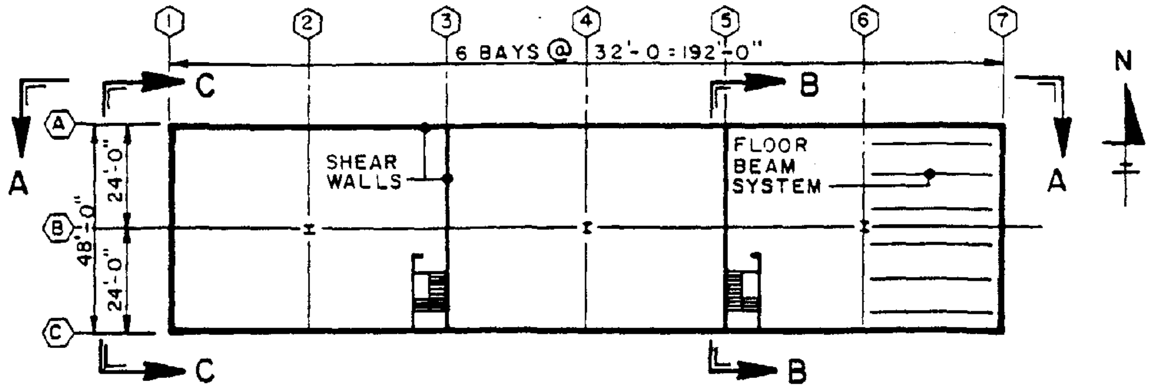
FIGURE 4
RESPONSE OF STRUCTURES TO EARTHQUAKE GROUND MOTIONS



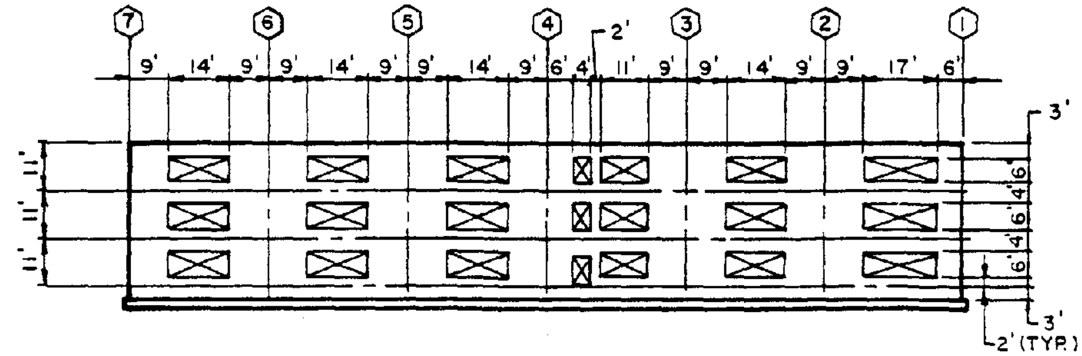
NOTES:

T_1 = PERIOD OF FIXED BASE STRUCTURE
T_2 = PERIOD OF STRUCTURE ON BASE ISOLATION

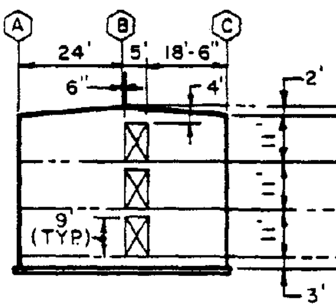
FIGURE 5
IDEALIZED SEISMIC GROUND RESPONSE SPECTRUM

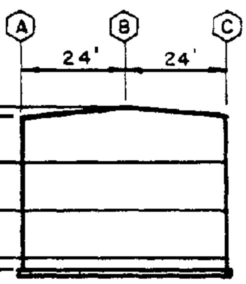


2nd & 3rd FLOOR PLAN



ELEVATION A-A

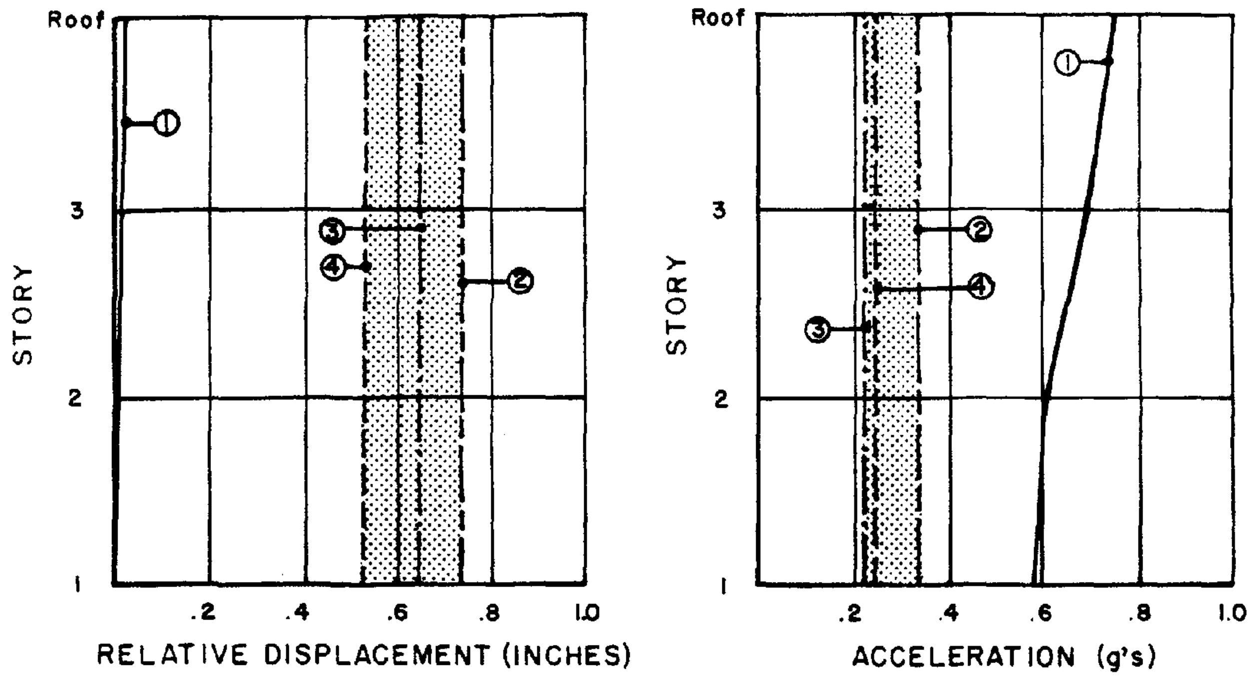




SECTION B-B ELEVATION C-C

REFERENCE — NAVFAC P-355 (1973)

FIGURE 6
STRUCTURAL CONFIGURATION FOR CASE STUDY 3-STORY STRUCTURE WITH SHEAR WALLS



LEGEND

CONVENTIONAL BUILDING

① LINEAR ELASTIC ANALYSIS(1)

BASE-ISOLATED BUILDING(1)

② 5% DAMPED BEARING

③ 5% DAMPED BEARING WITH FRICTIONAL INTERFACE

④ 15% DAMPED BEARING

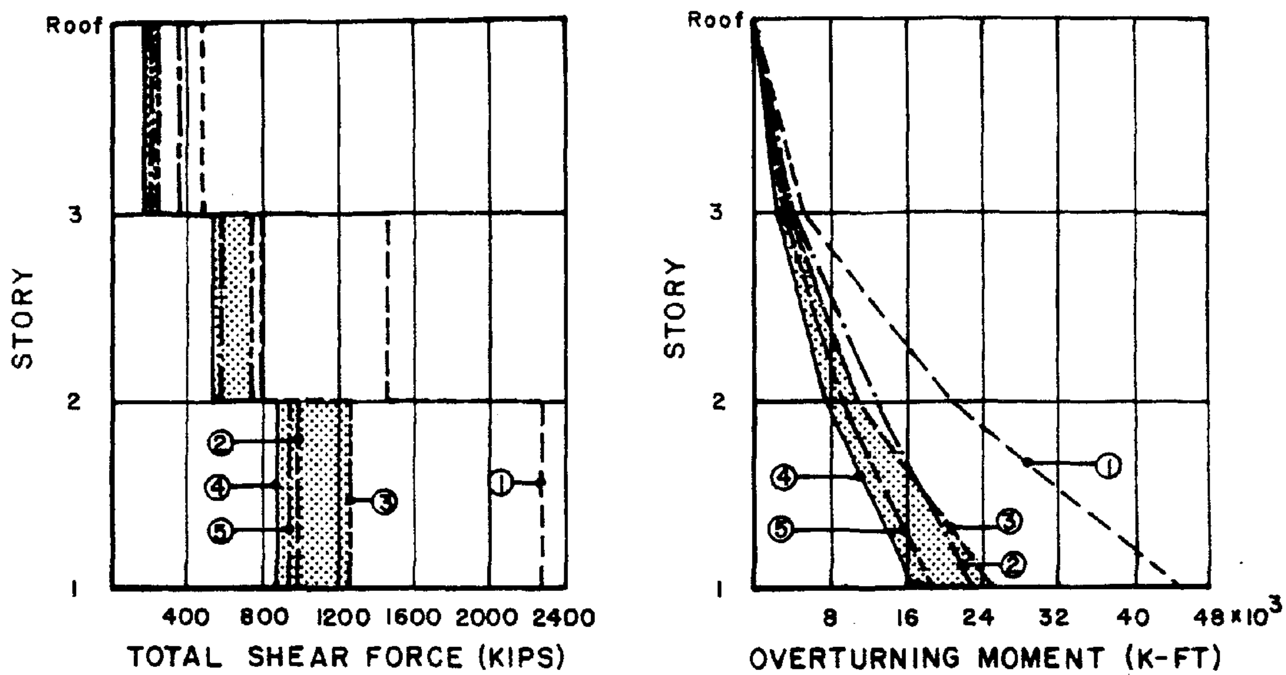
---

(1) BASED ON EL CENTRO 1940 S00E RECORD SCALED TO 0.6g PEAK.

REFERENCE: VAIDYA AND EGGENBERGER, 1984

FIGURE 7

COMPARISON OF BASE-ISOLATED AND CONVENTIONAL RESPONSE DISPLACEMENTS AND ACCELERATIONS



LEGEND

CONVENTIONAL BUILDING

① LINEAR ELASTIC ANALYSIS(1)(2)

② ZONE 4 (SEAOC, 1974)

BASE-ISOLATED BUILDING(2)

③ 5% DAMPED BEARING

④ 5% DAMPED BEARING WITH FRICTIONAL INTERFACE

⑤ 15% DAMPED BEARING

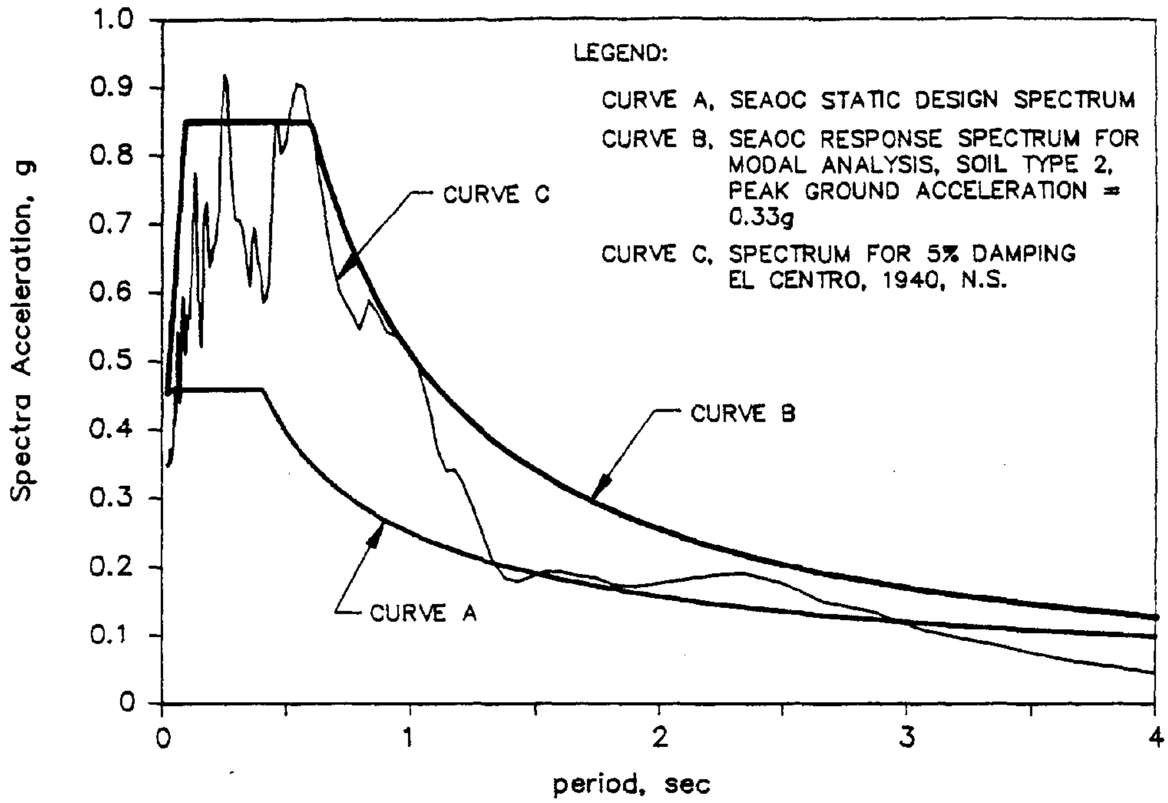
---

(1) PROVIDED AS AN APPROX. MEASURE OF LINEAR ELASTIC DISPLACEMENTS.

(2) BASED ON EL CENTRO 1940 S 00 E RECORD SCALED TO 0.6g PEAK.

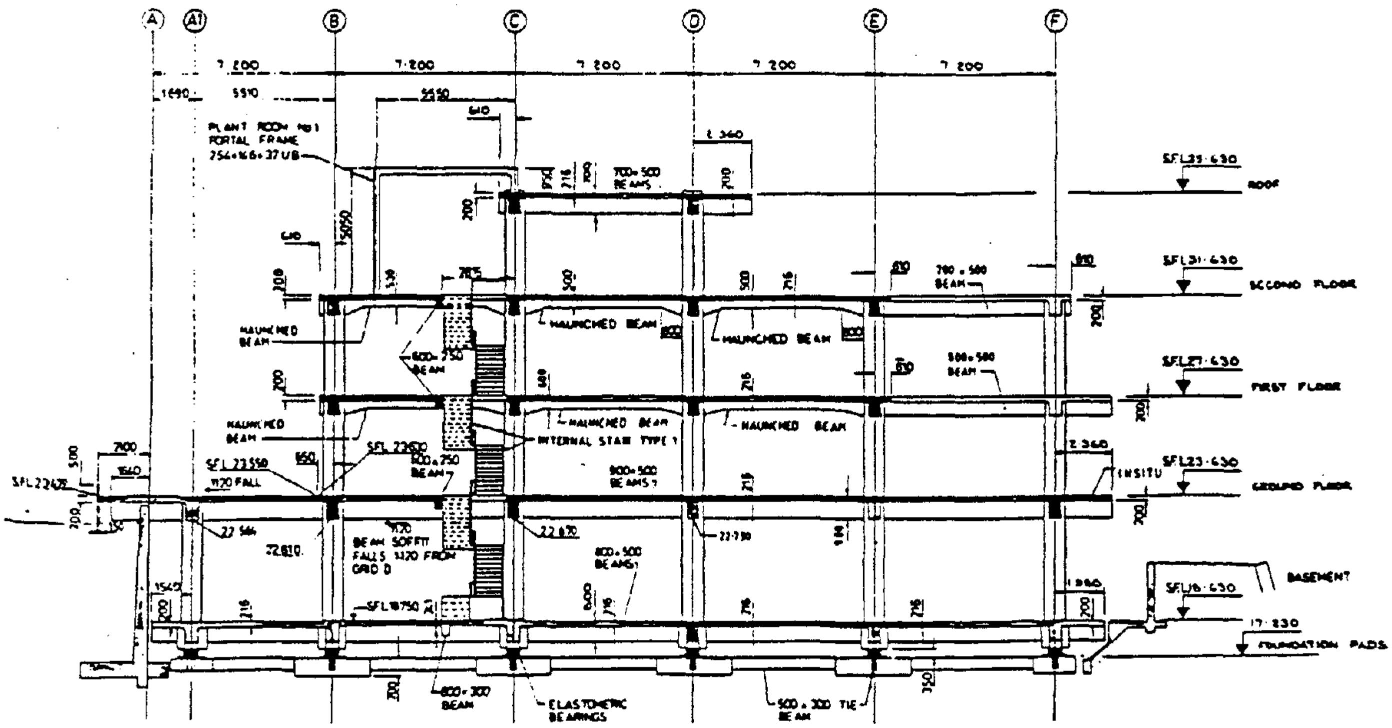
REFERENCE: VAIDYA AND EGGENBERGER, 1984

FIGURE 8
COMPARISON OF BASE-ISOLATED AND CONVENTIONAL RESPONSE
STORY SHEARS AND OVERTURNING MOMENTS



REFERENCE: SEAOC, 1985

FIGURE 9
SEAOC SEISMIC DESIGN RESPONSE SPECTRA



REFERENCE: MEGGET, 1978

FIGURE 10
TRANSVERSE SECTION, WILLIAM CLAYTON BUILDING
WELLINGTON, N.Z.

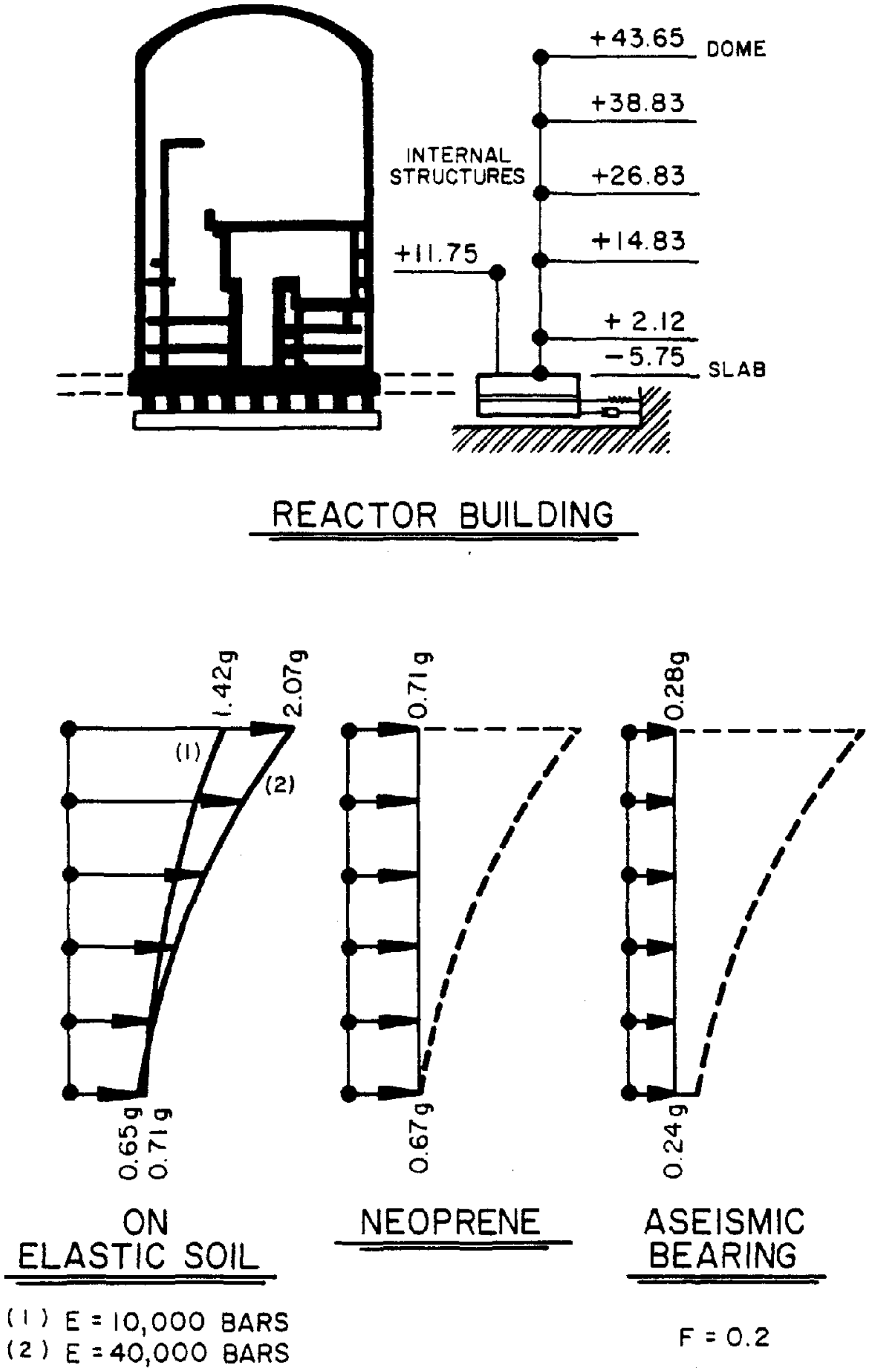


FIGURE 11
ACCELERATION LEVELS IN REACTOR BUILDING
UNDER 0.6g EL CENTRO EARTHQUAKE

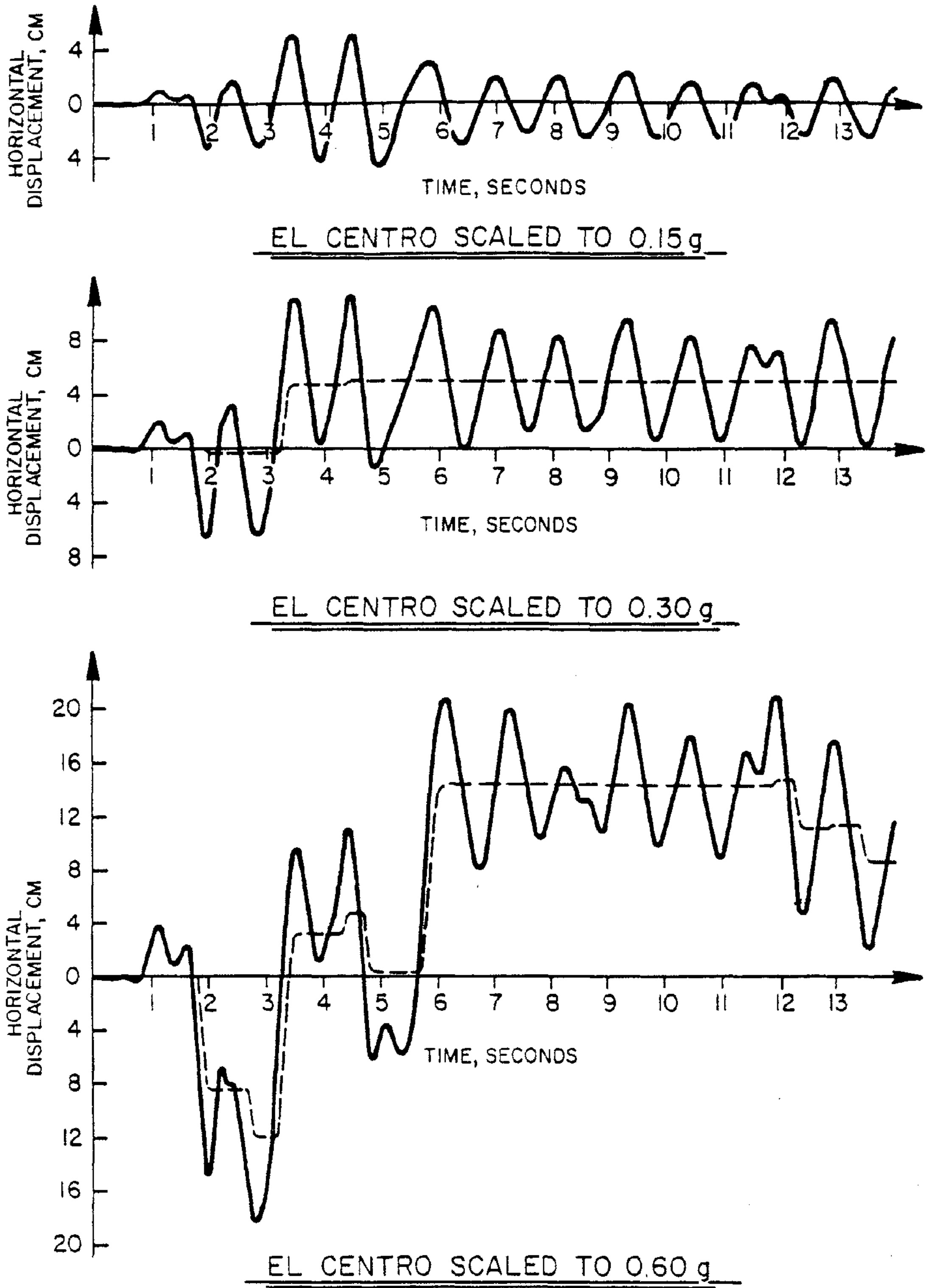
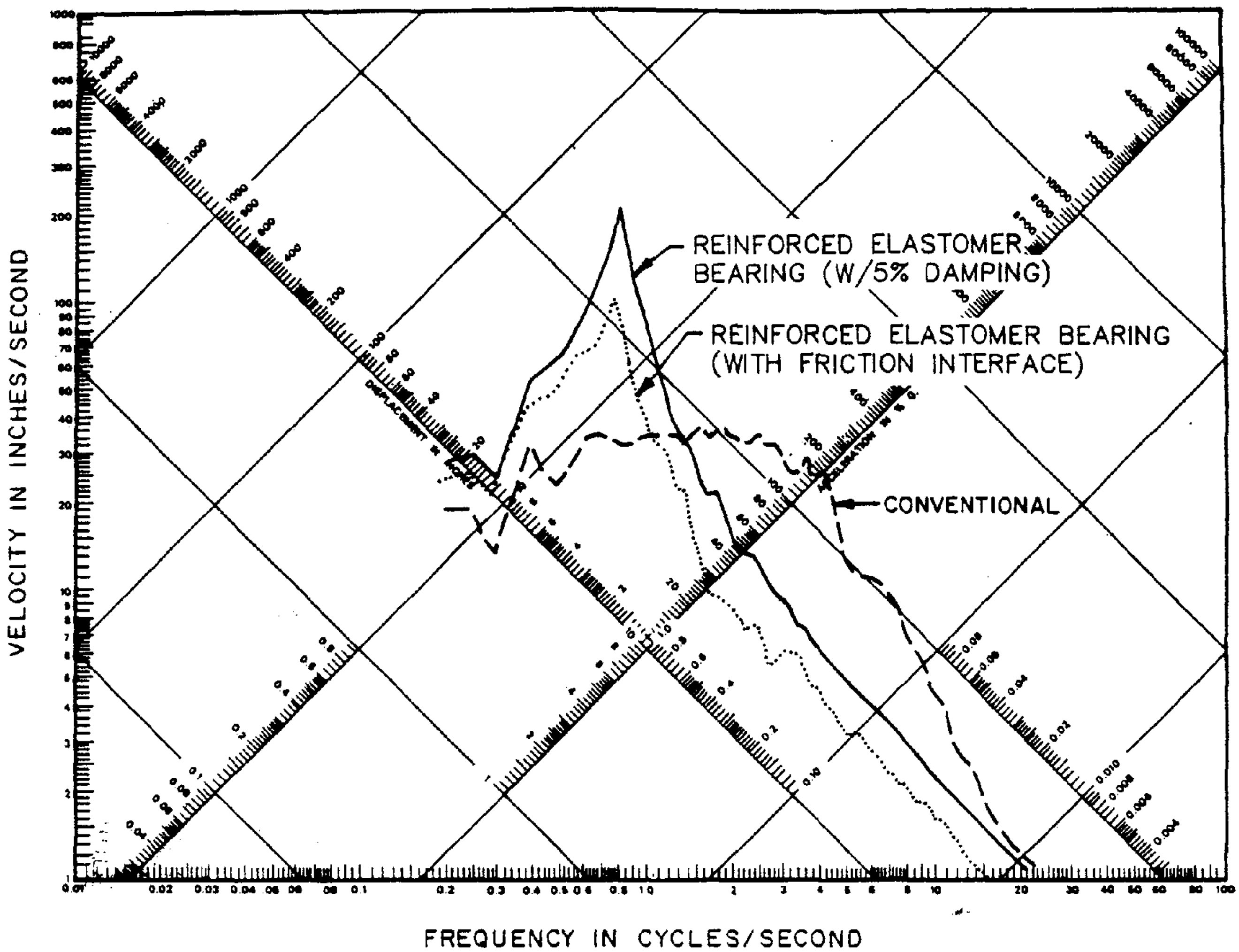
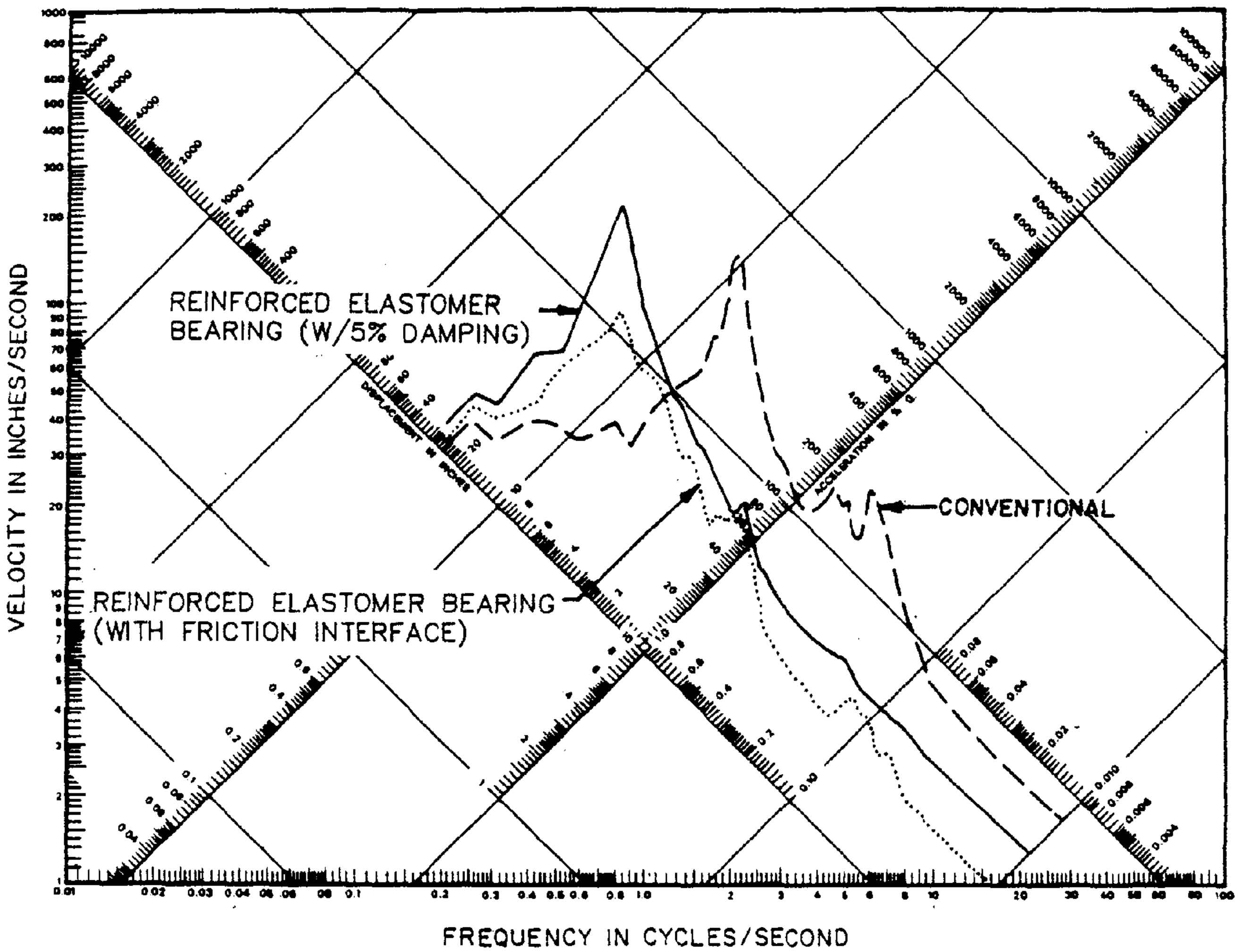


FIGURE 12
RELATIVE DISPLACEMENT BETWEEN THE UPPER
AND THE LOWER RAFT



EQUIPMENT DAMPING = 4%
PEAK GROUND ACCELERATION = 0.3g

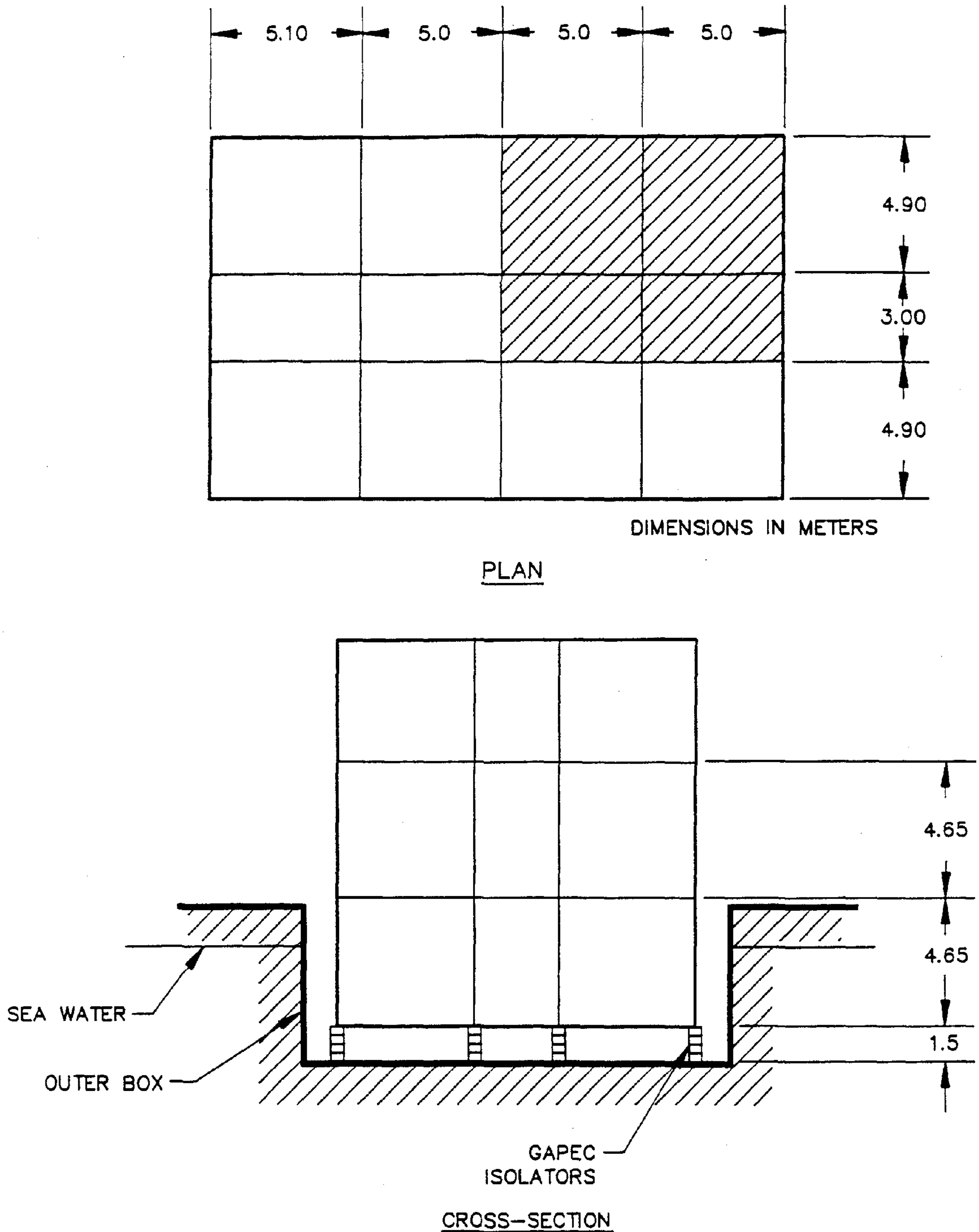
FIGURE 13
HORIZONTAL FLOOR RESPONSE SPECTRUM AT BASE OF REACTOR BUILDING



EQUIPMENT DAMPING = 4%
PEAK GROUND ACCELERATION = 0.3g

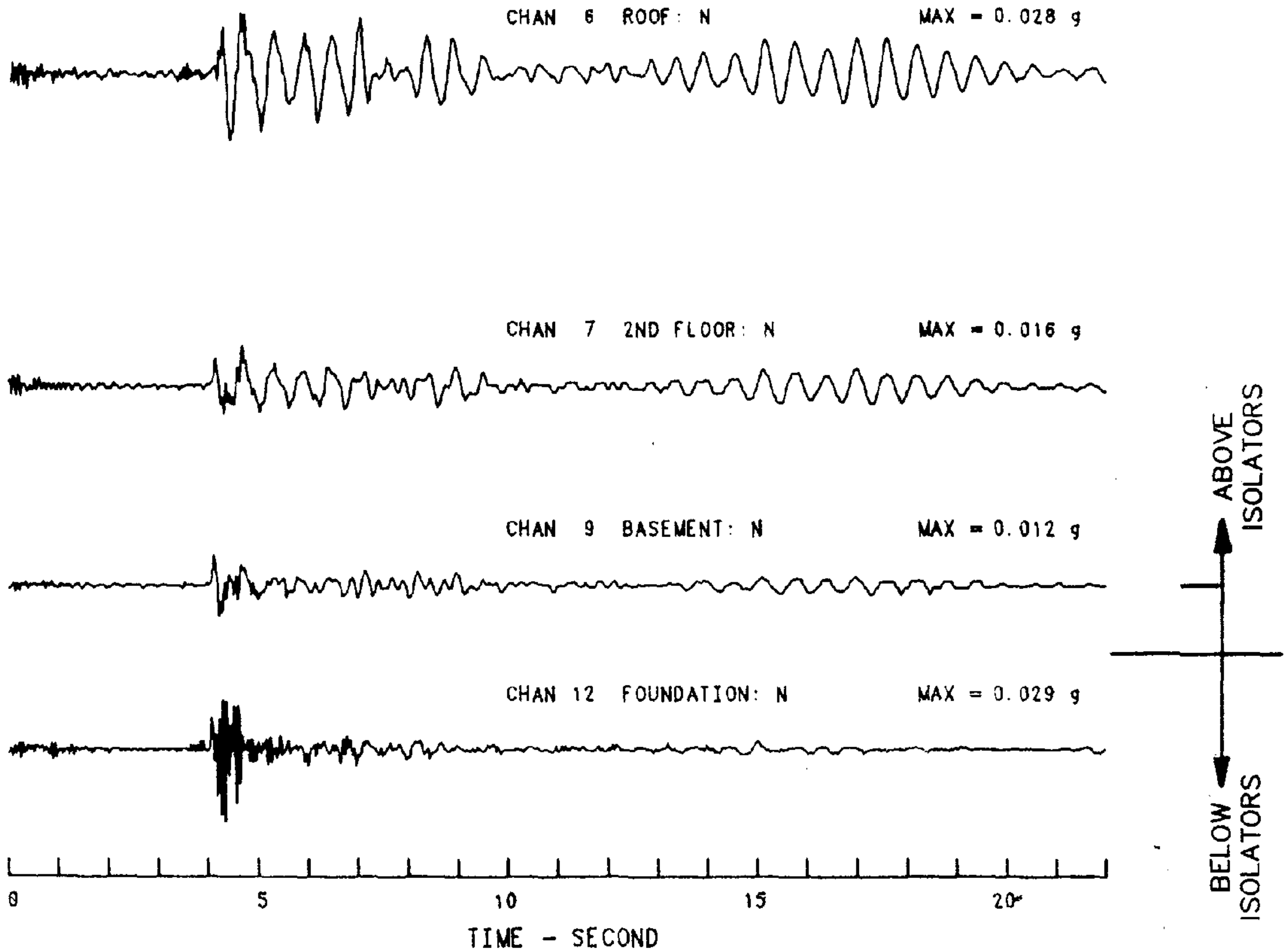
FIGURE 14
HORIZONTAL FLOOR RESPONSE SPECTRUM AT BASE OF REACTOR BUILDING

86-246-A4



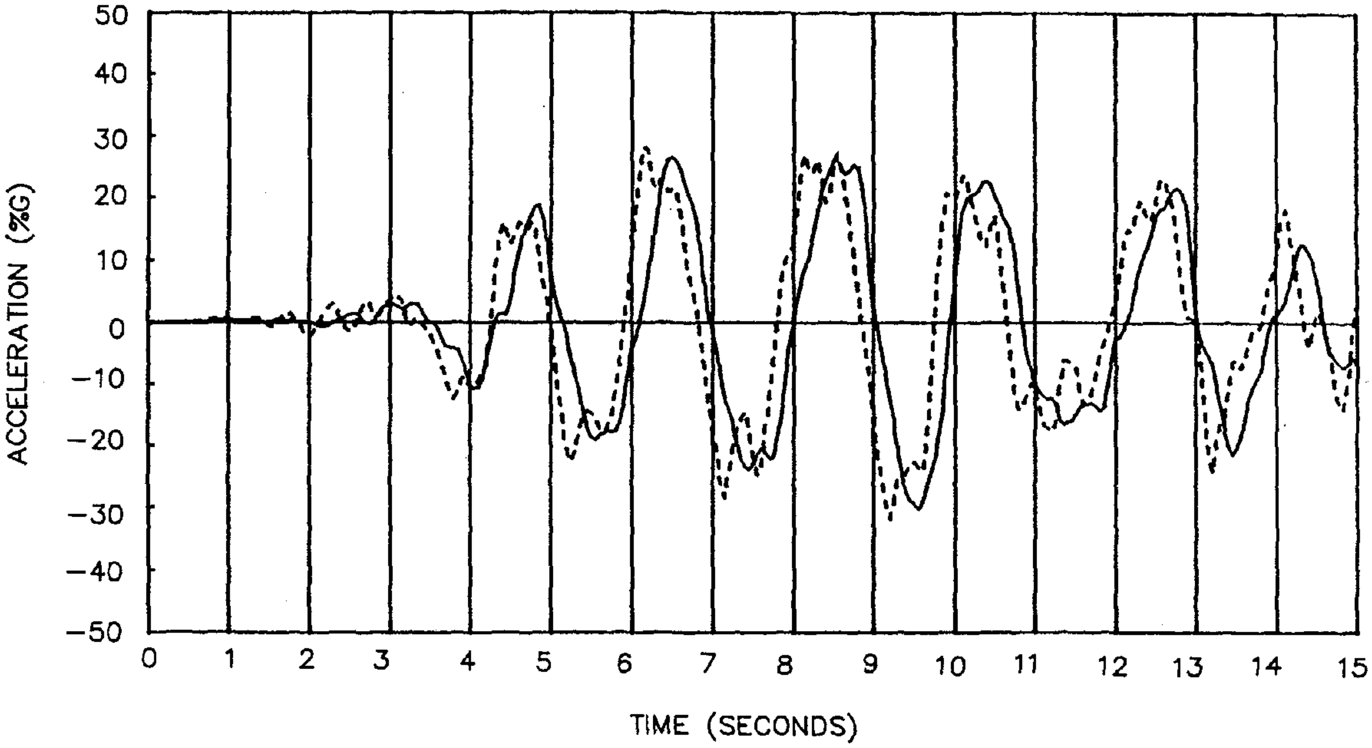
REFERENCE: DELFOSSE AND DELFOSSE, 1986

FIGURE 15
SCHEMATIC PLAN AND CROSS-SECTION OF RADIOACTIVE WASTE BUILDING IN FRANCE



REFERENCE: HUANG ET. AL., 1986

FIGURE 16
ACCELEROGRAMS OBTAINED IN THE FCLJC BUILDING DURING THE REDLANDS EARTHQUAKE OF OCTOBER 2, 1985



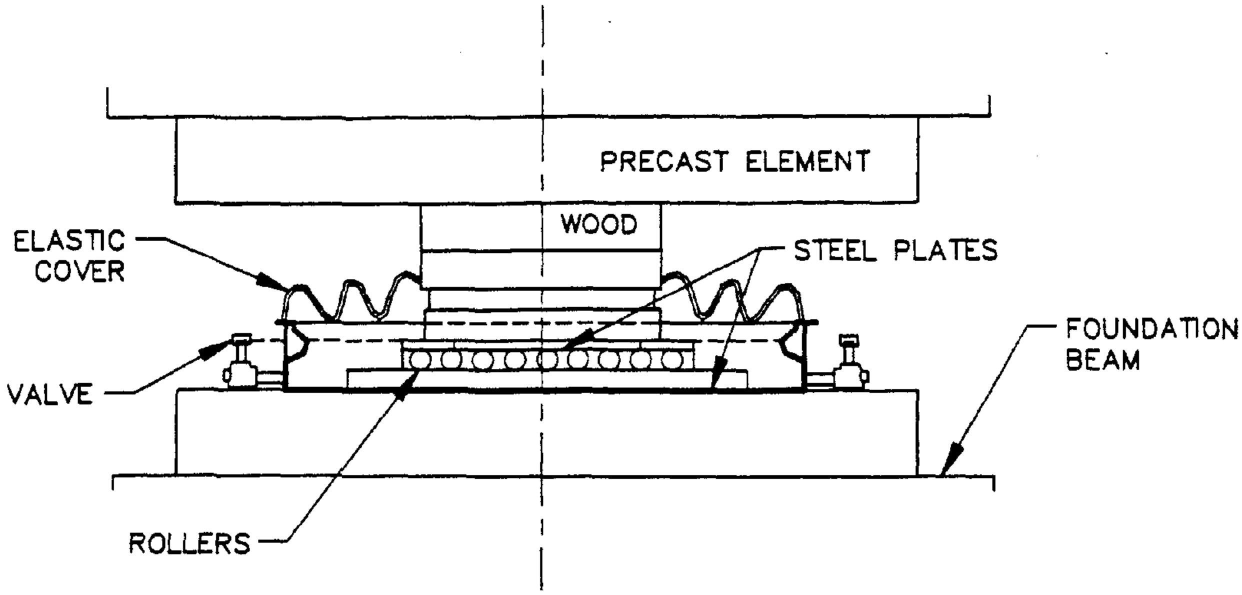
BASEMENT ACCELERATION

ROOF ACCELERATION

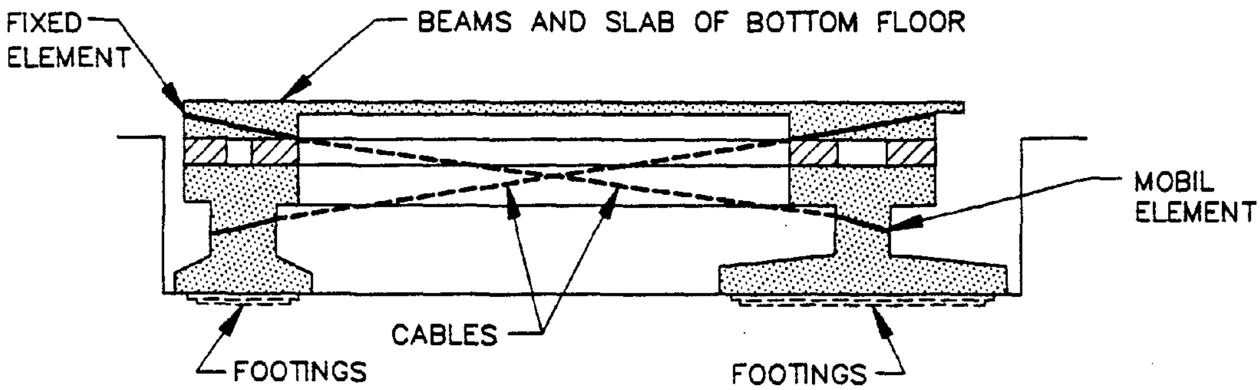
REFERENCE: REID AND TARICS, 1983

FIGURE 17
COMPUTED ACCELERATION TIME HISTORIES
FOOTHILL COMMUNITY LAW AND JUSTICE CENTER BUILDING

86-246-A36



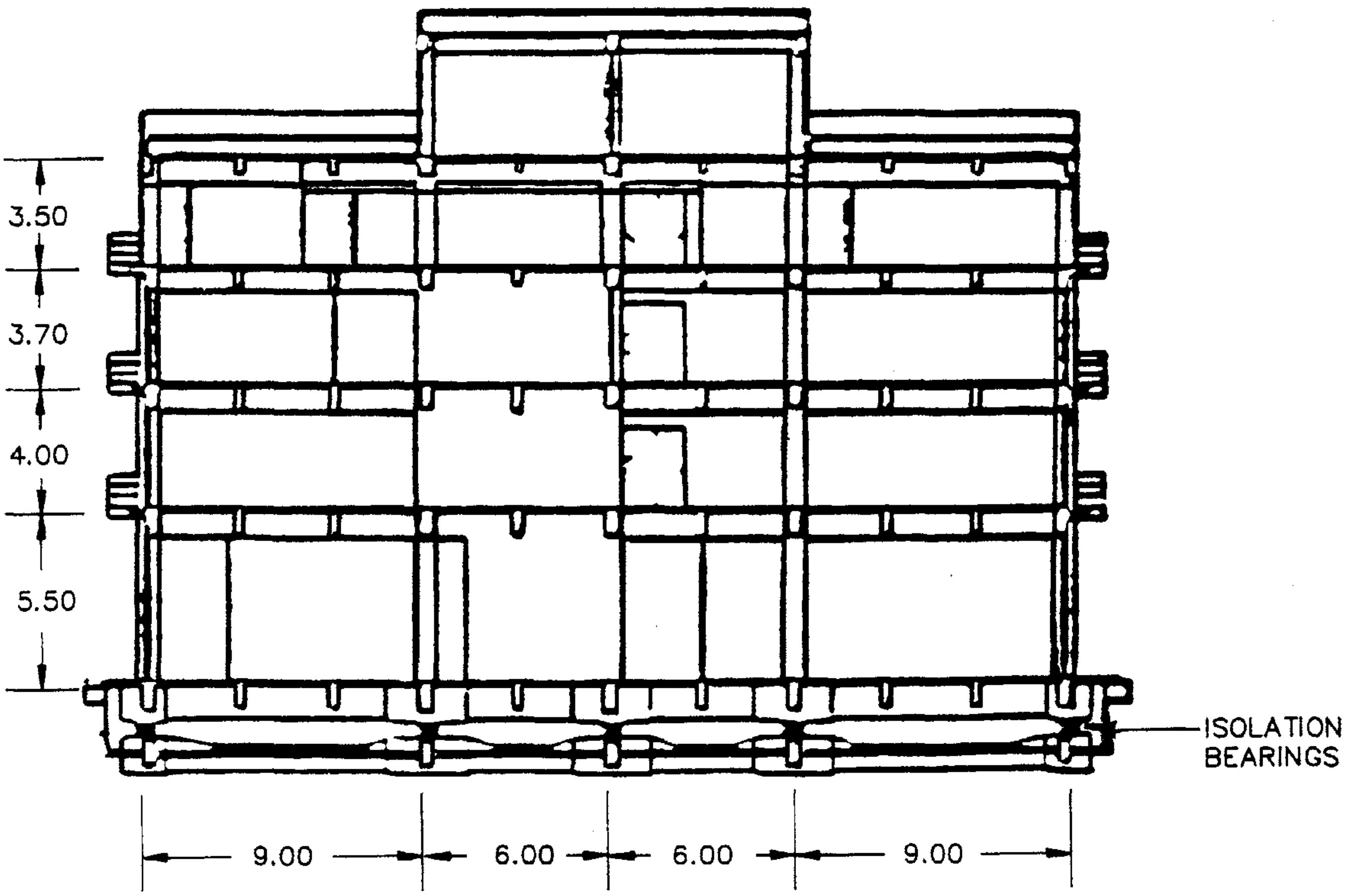
a) SEISMIC UNIT



LIMITING DISPLACEMENT SYSTEM

REFERENCE: GONZALEZ FLORES, 1964

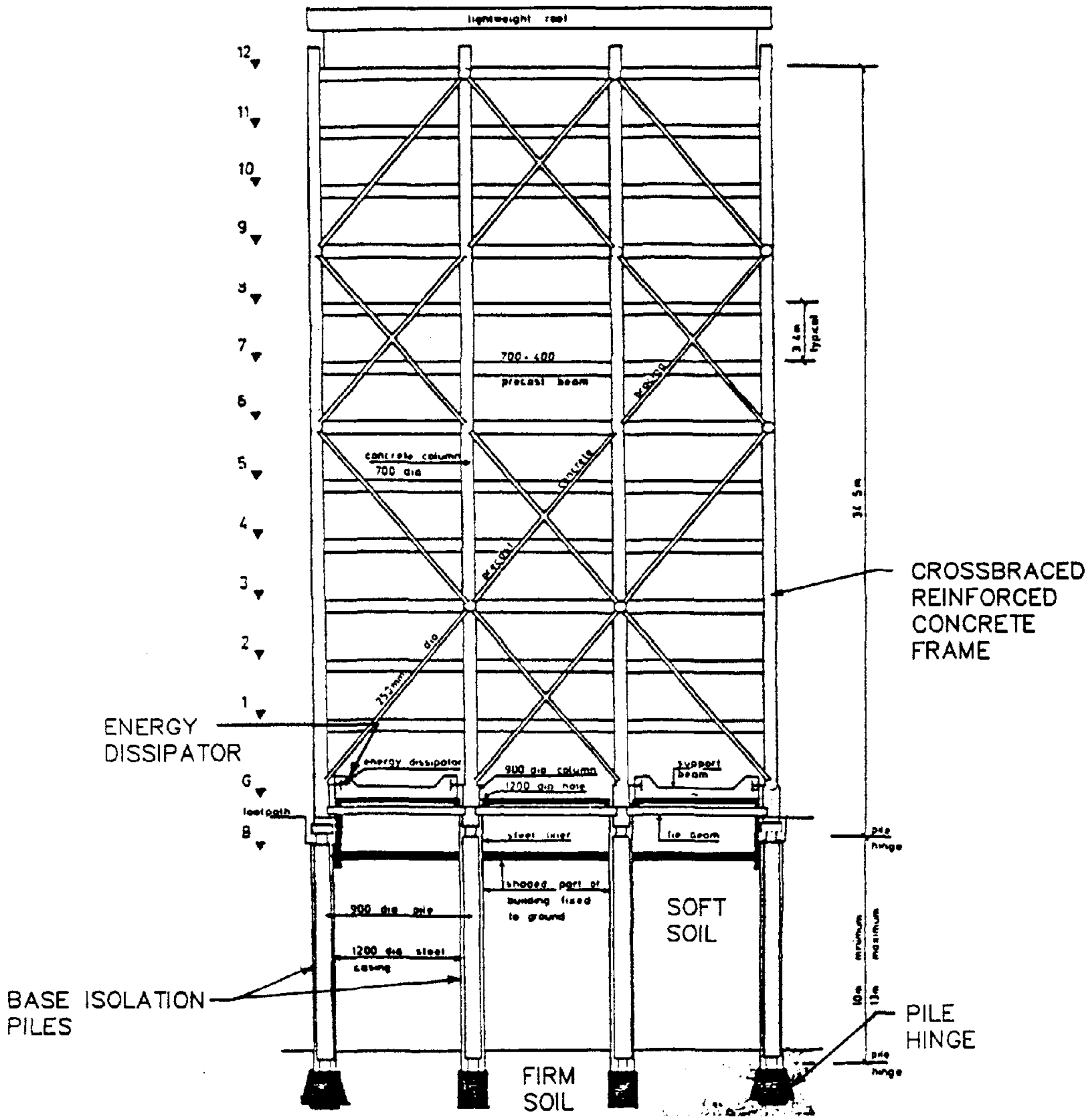
FIGURE 18
STEEL–ROLLER ISOLATION SYSTEM USED FOR A BUILDING IN MEXICO D.F.



DIMENSIONS IN METERS

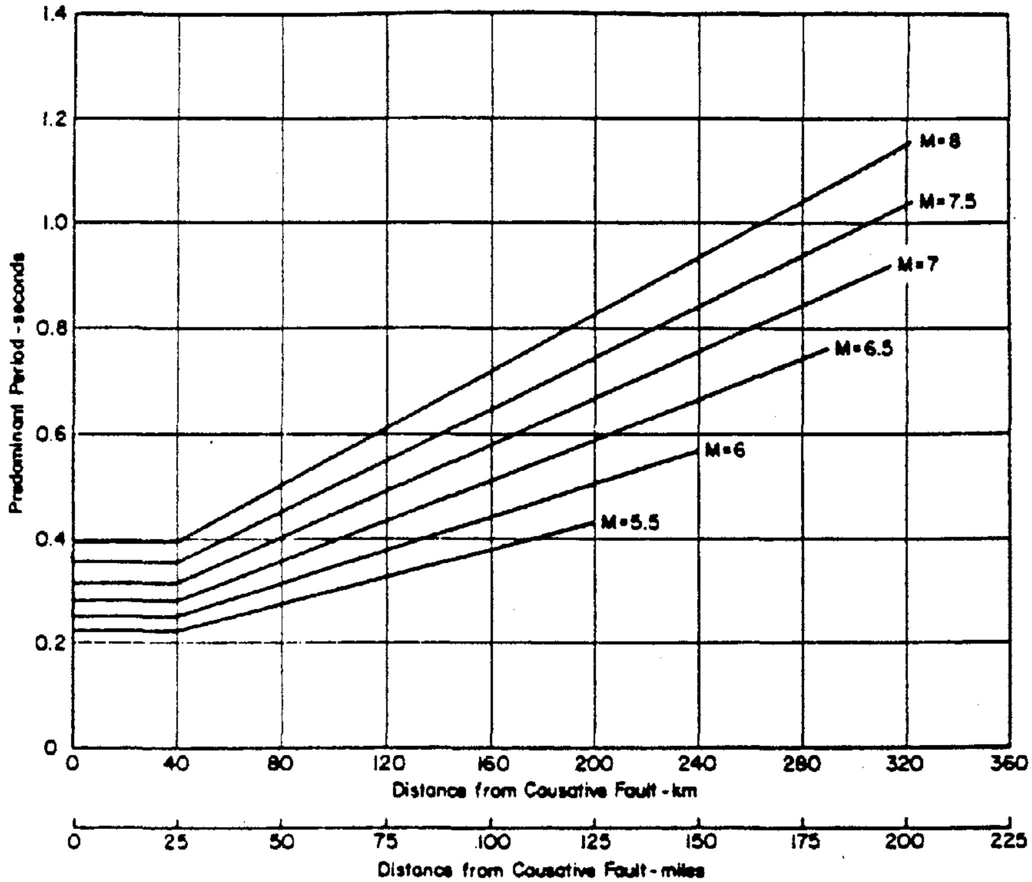
REFERENCE: MIYAZAKI ET. AL., 1986

FIGURE 19
SECTION OF THE OILES TECHNICAL CENTER BUILDING, JAPAN



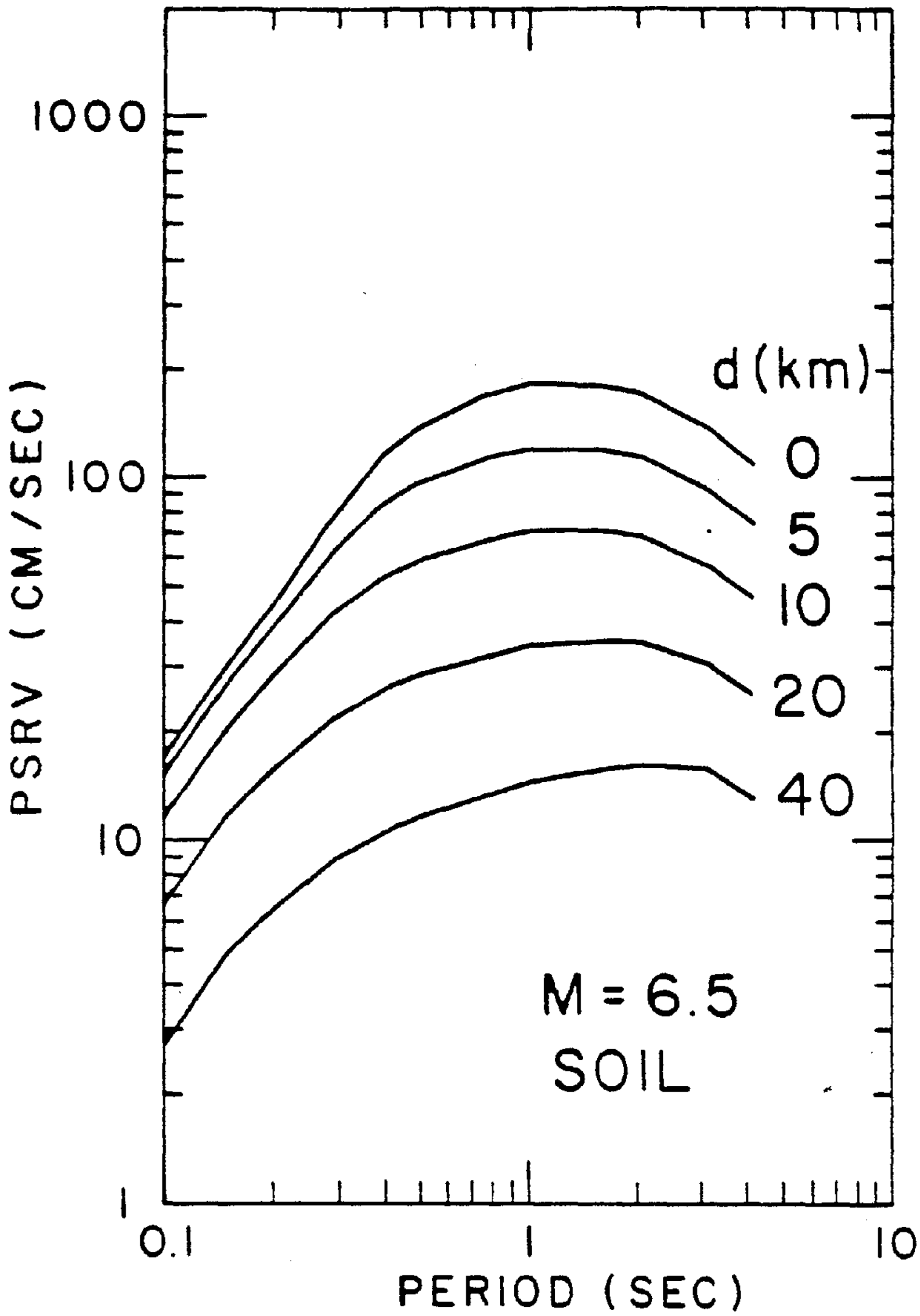
REFERENCE: BOARDMAN ET. AL., 1983

FIGURE 20
ELEVATION OF UNION HOUSE BUILDING
AUCKLAND, N.Z.



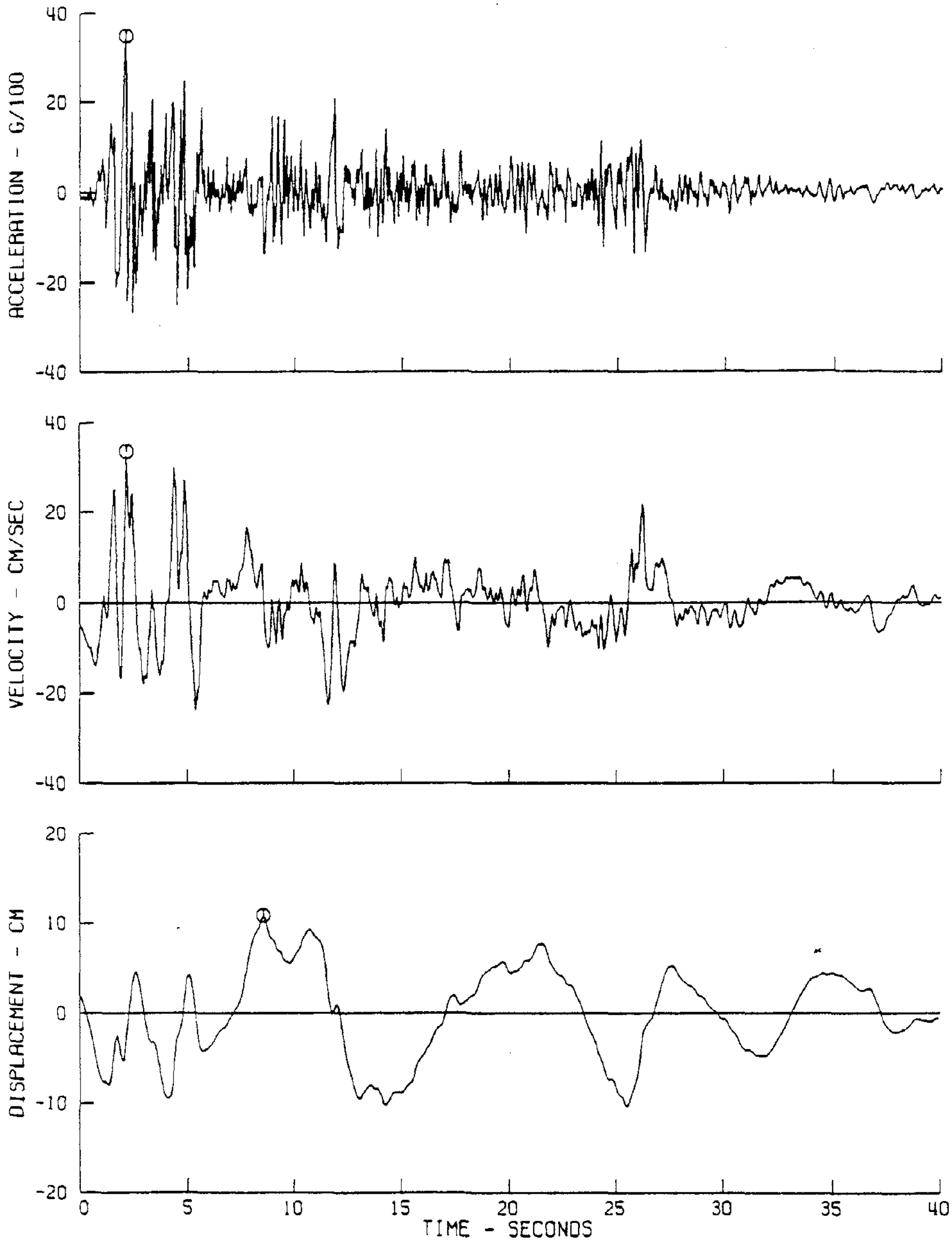
REFERENCE: SEED ET. AL., 1968

FIGURE 21
PREDOMINANT PERIODS FOR MAXIMUM ACCELERATION IN ROCK



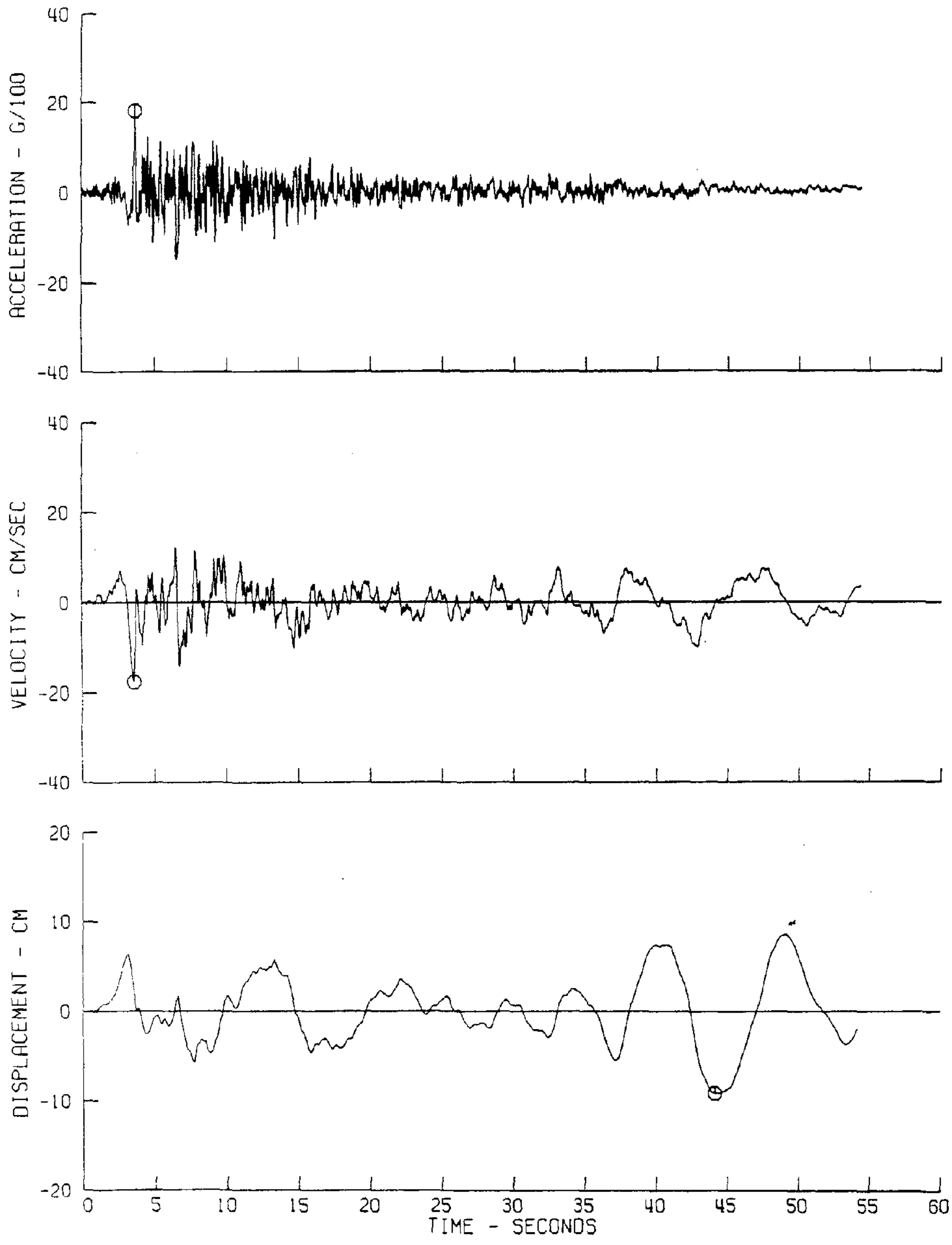
REFERENCE: JOYNER AND BOORE, 1982
MAGNITUDE = 6.5
SOIL SITE
SPECTRA CORRESPOND TO LARGER OF
TWO HORIZONTAL COMPONENTS.

FIGURE 22
PSEUDO-VELOCITY RESPONSE SPECTRA AT
VARYING DISTANCE FROM EPICENTER



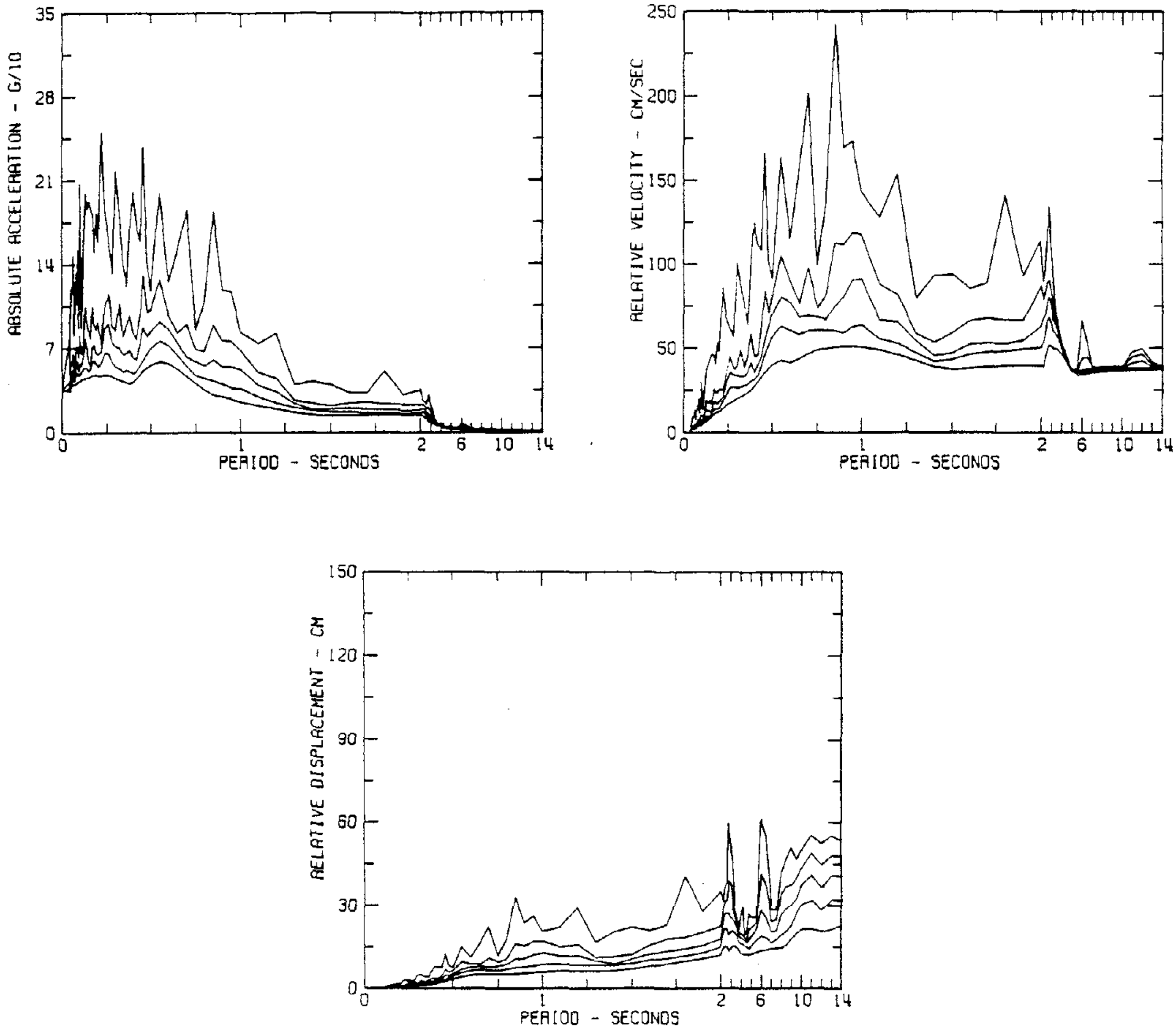
REFERENCE: LEE ET. AL., 1980

FIGURE 23
IMPERIAL VALLEY EARTHQUAKE, MAY 18, 1940
GROUND MOTION TIME HISTORIES, SOOE



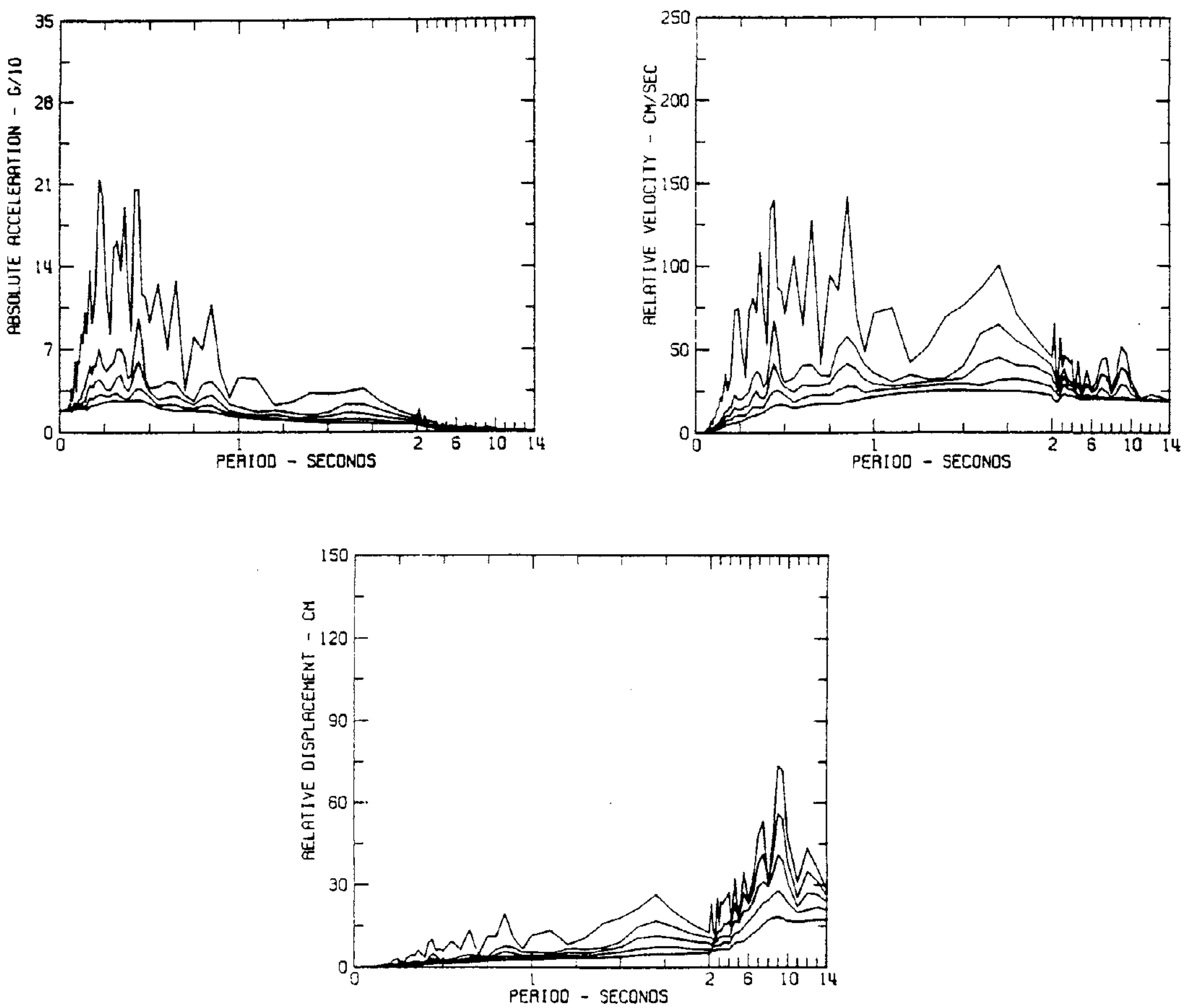
REFERENCE: LEE ET. AL., 1980

FIGURE 24
KERN COUNTY, CALIFORNIA EARTHQUAKE, JULY 21, 1952
GROUND MOTION TIME HISTORIES, S69E



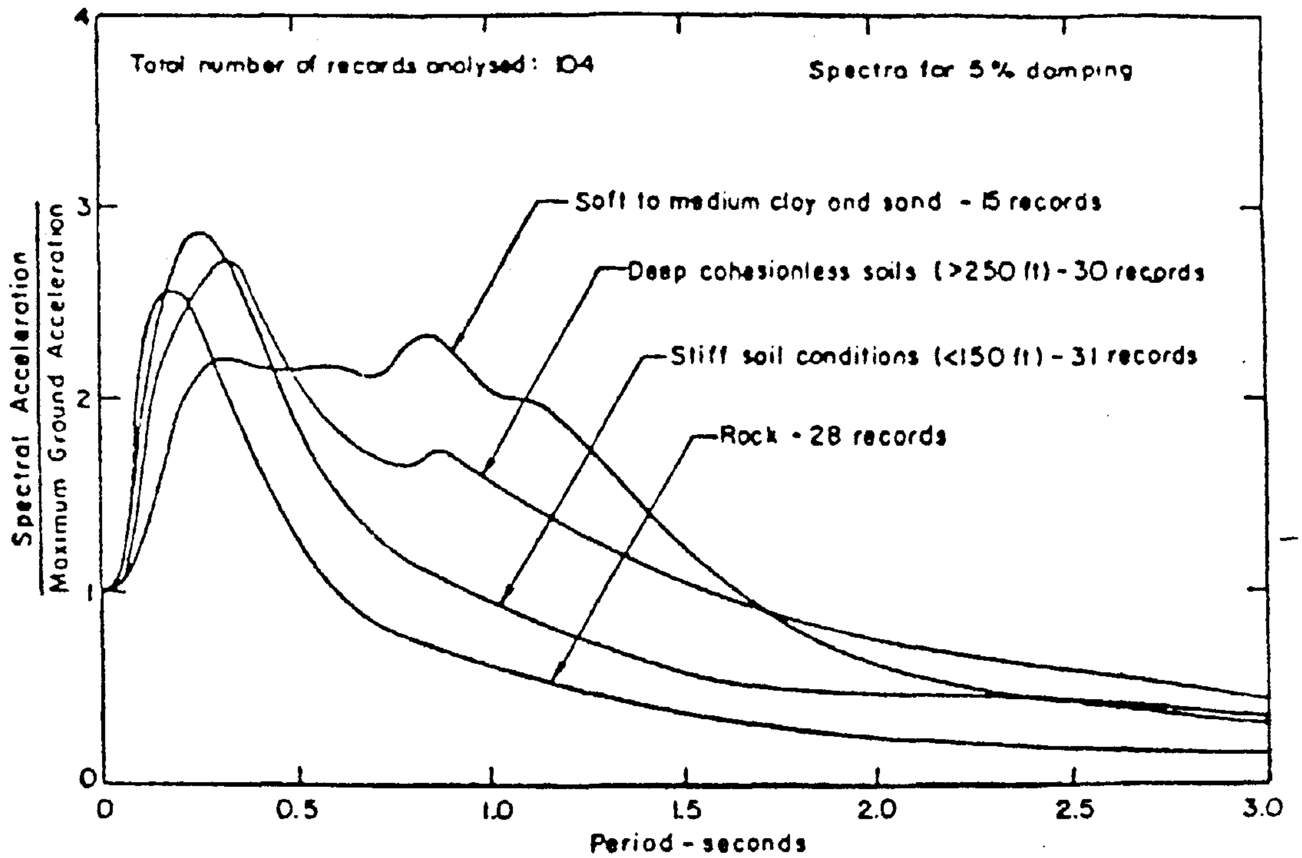
DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL
REFERENCE: LEE ET. AL., 1980

FIGURE 25
RESPONSE SPECTRA FOR THE EL CENTRO 1940 N–S EARTHQUAKE RECORD



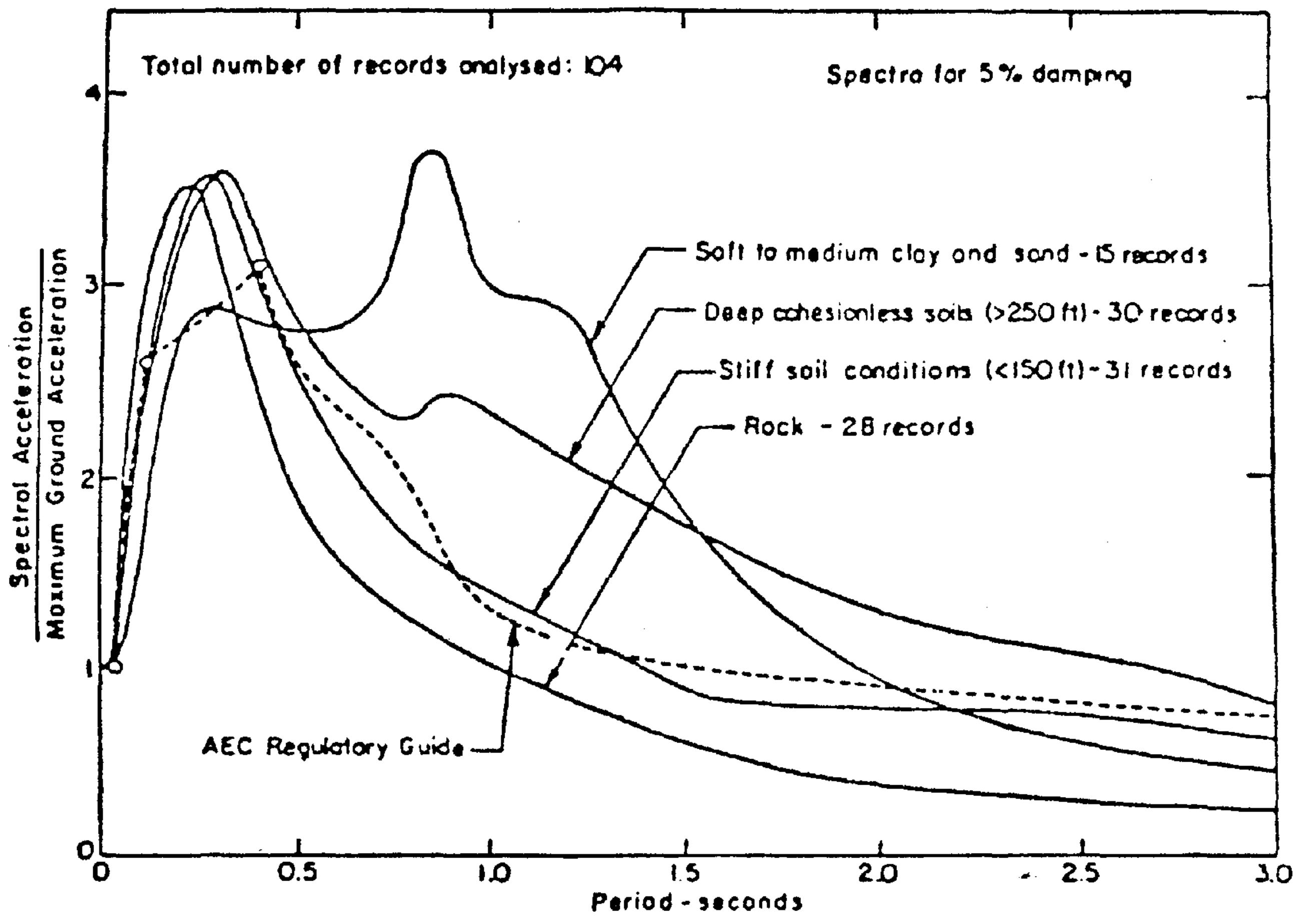
DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL
REFERENCE: LEE ET. AL., 1980

FIGURE 26
RESPONSE SPECTRA FOR THE KERN COUNTY 1952 SOOE
EARTHQUAKE RECORD



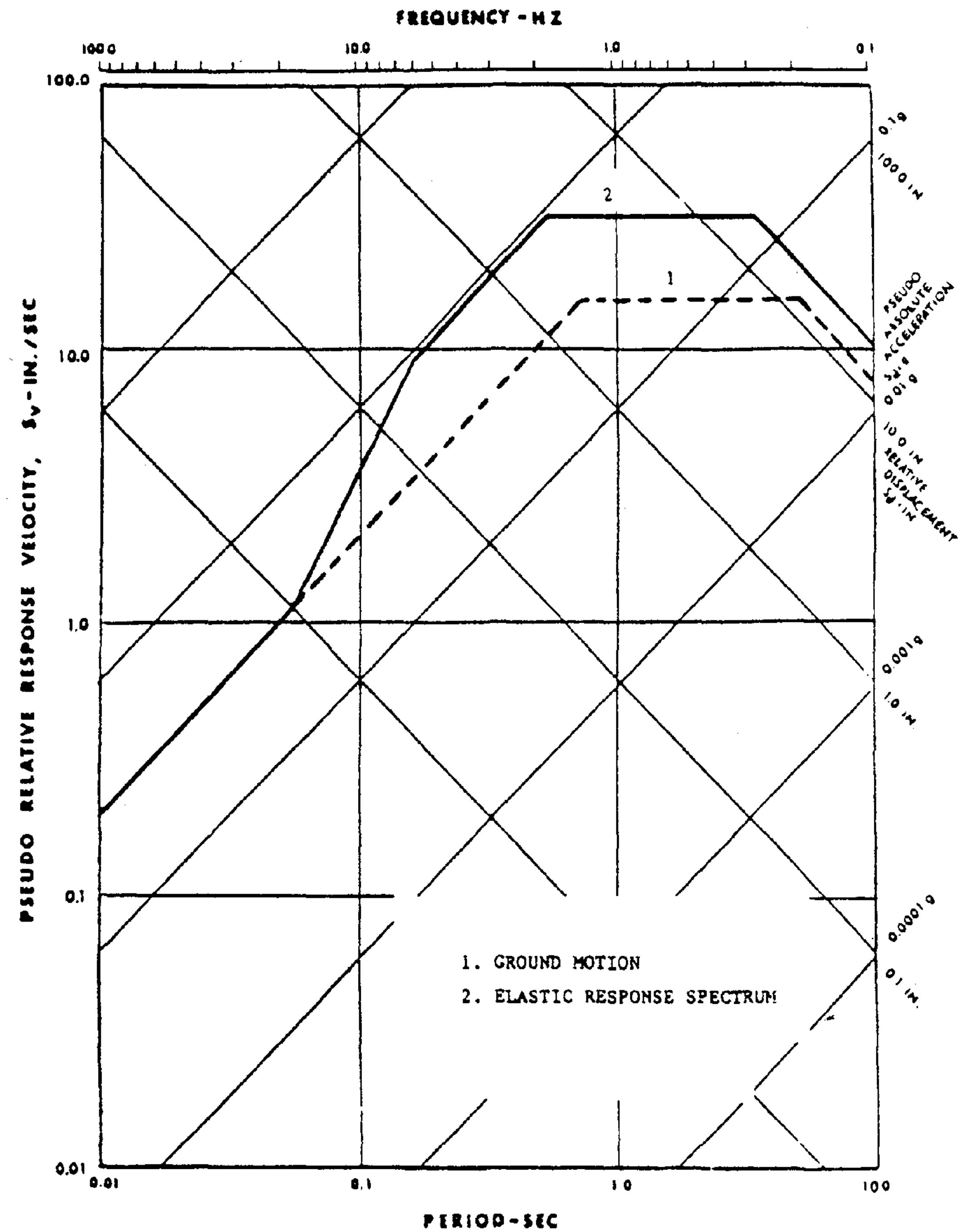
REFERENCE: SEED ET. AL., 1974

FIGURE 27
AVERAGE ACCELERATION SPECTRA FOR DIFFERENT SITE CONDITIONS



REFERENCE: SEED ET. AL., 1974

FIGURE 28
84 PERCENTILE ACCELERATION SPECTRA FOR DIFFERENT SITE CONDITIONS



REFERENCE: NEWMARK AND HALL, 1982

FIGURE 29
NEWMARK–HALL SPECTRUM

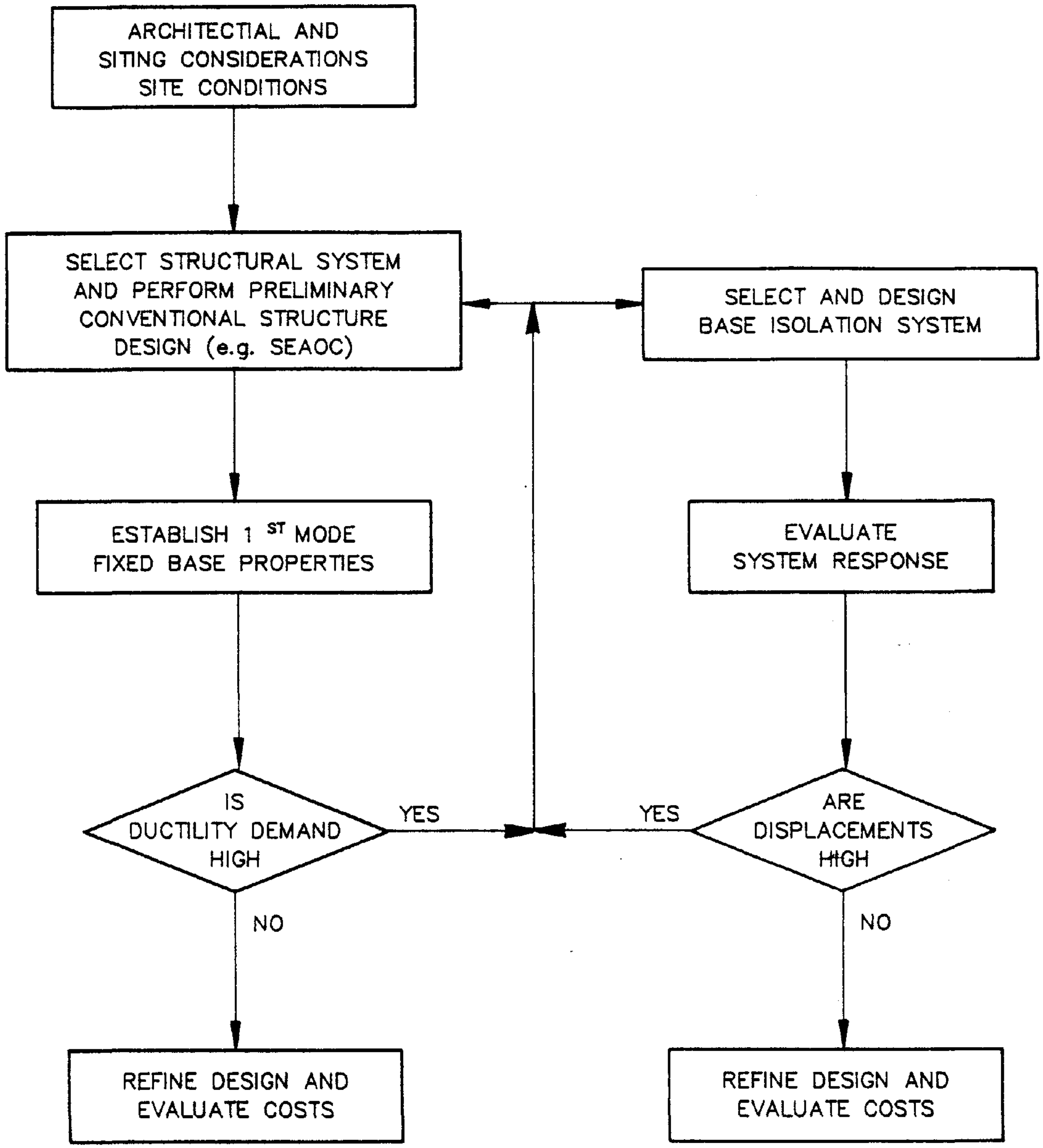


FIGURE 30
CRITERIA FOR SELECTING BASE ISOLATION

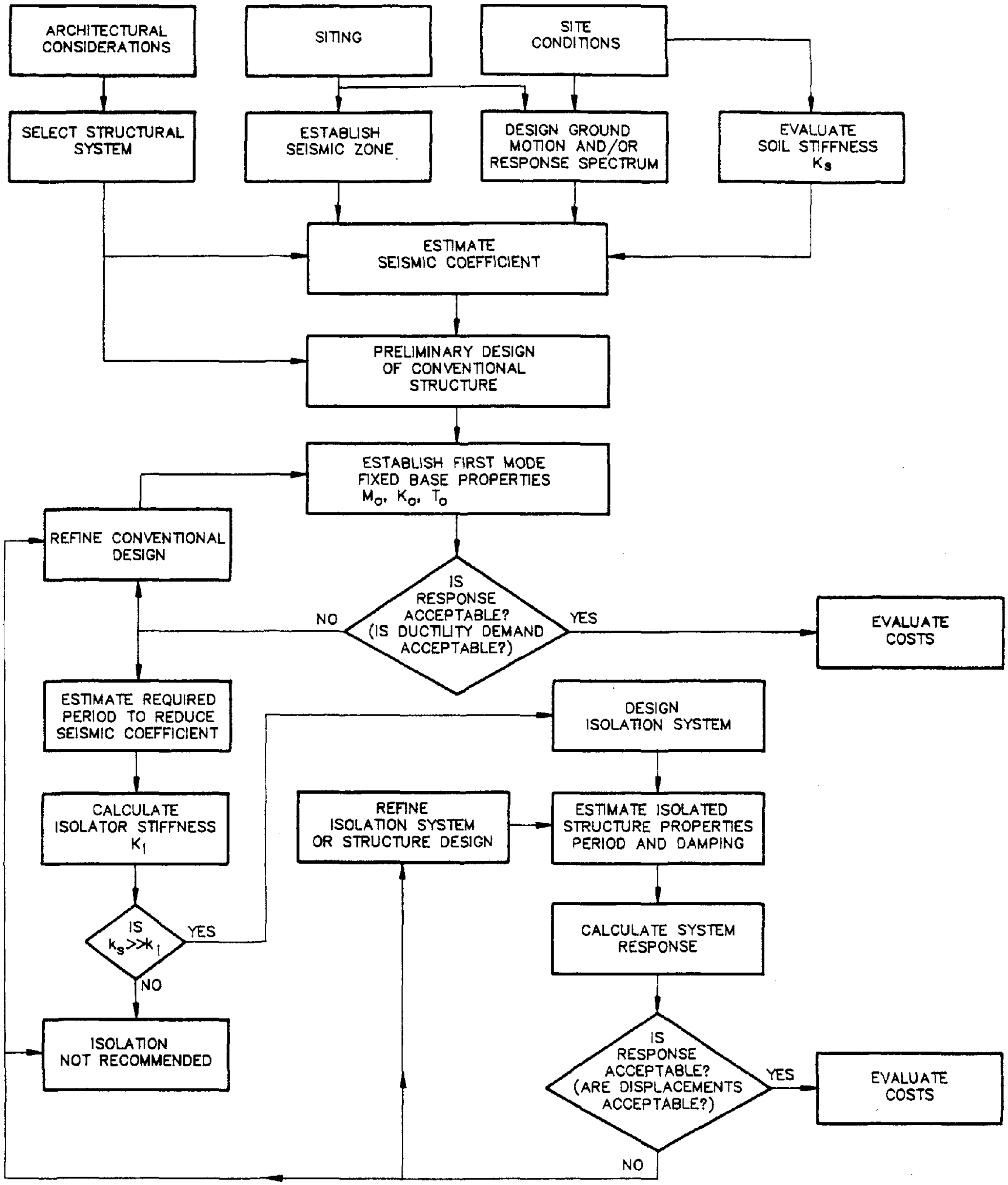


FIGURE 31
STEPS IN THE SELECTION OF SEISMIC DESIGN STRATEGY

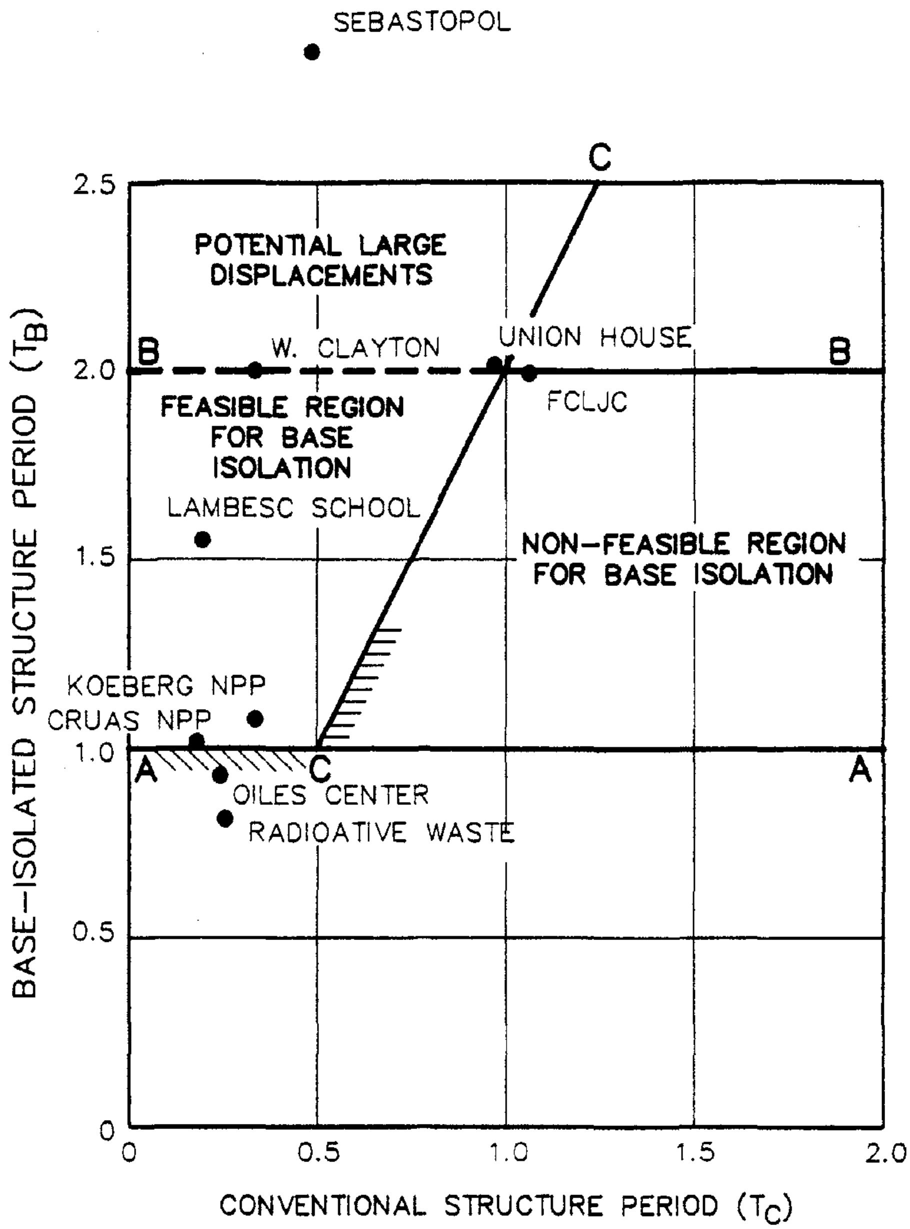


FIGURE 32
FEASIBLE REGION FOR BASE ISOLATION

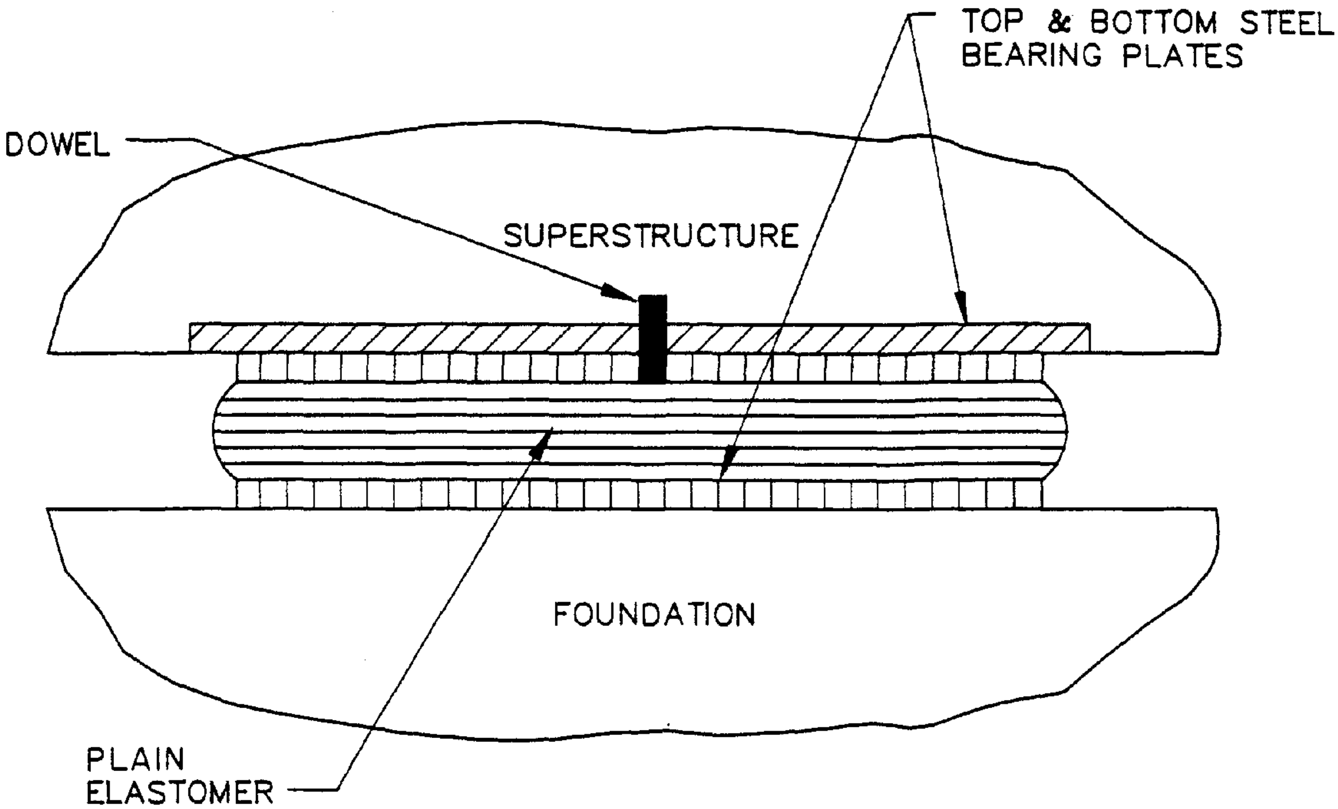


FIGURE 33
PLAIN ELASTOMER BEARING

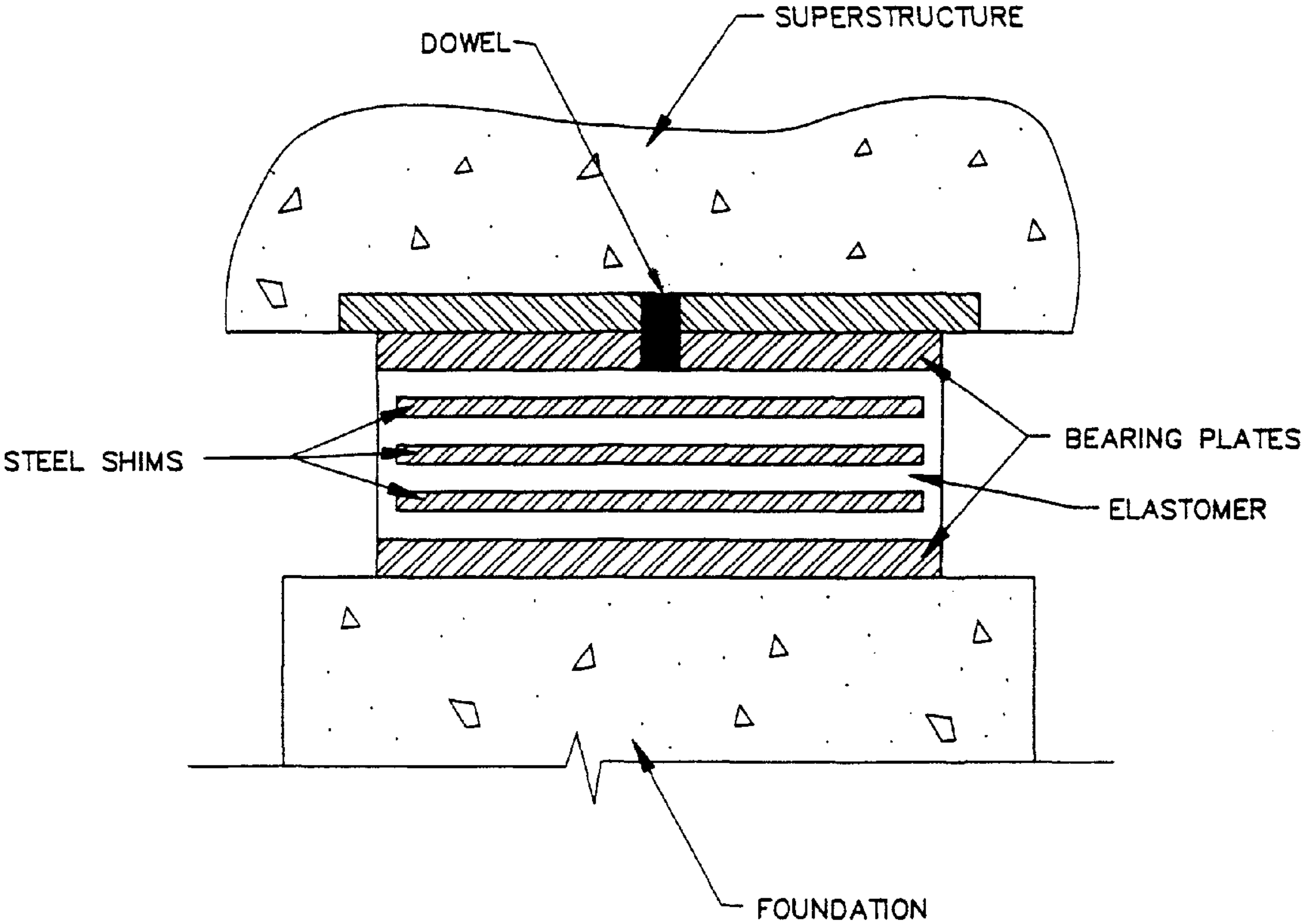
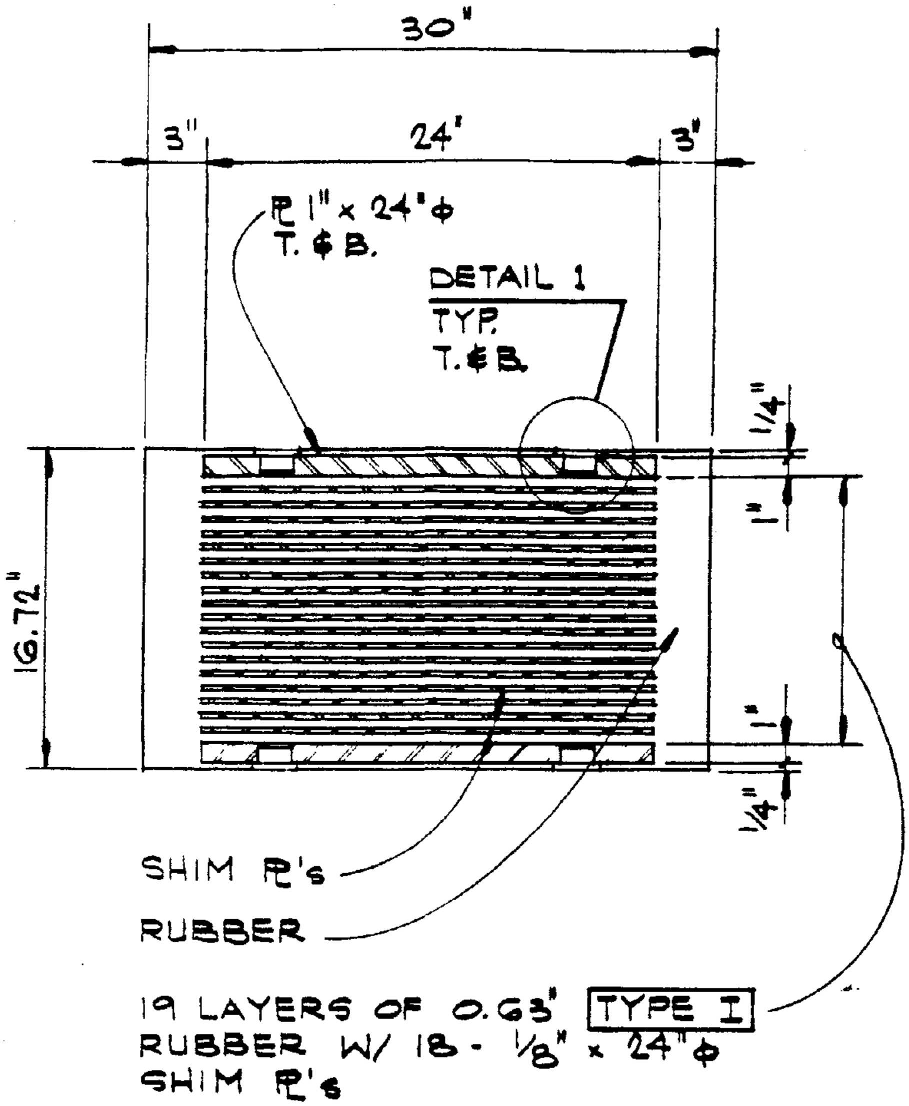


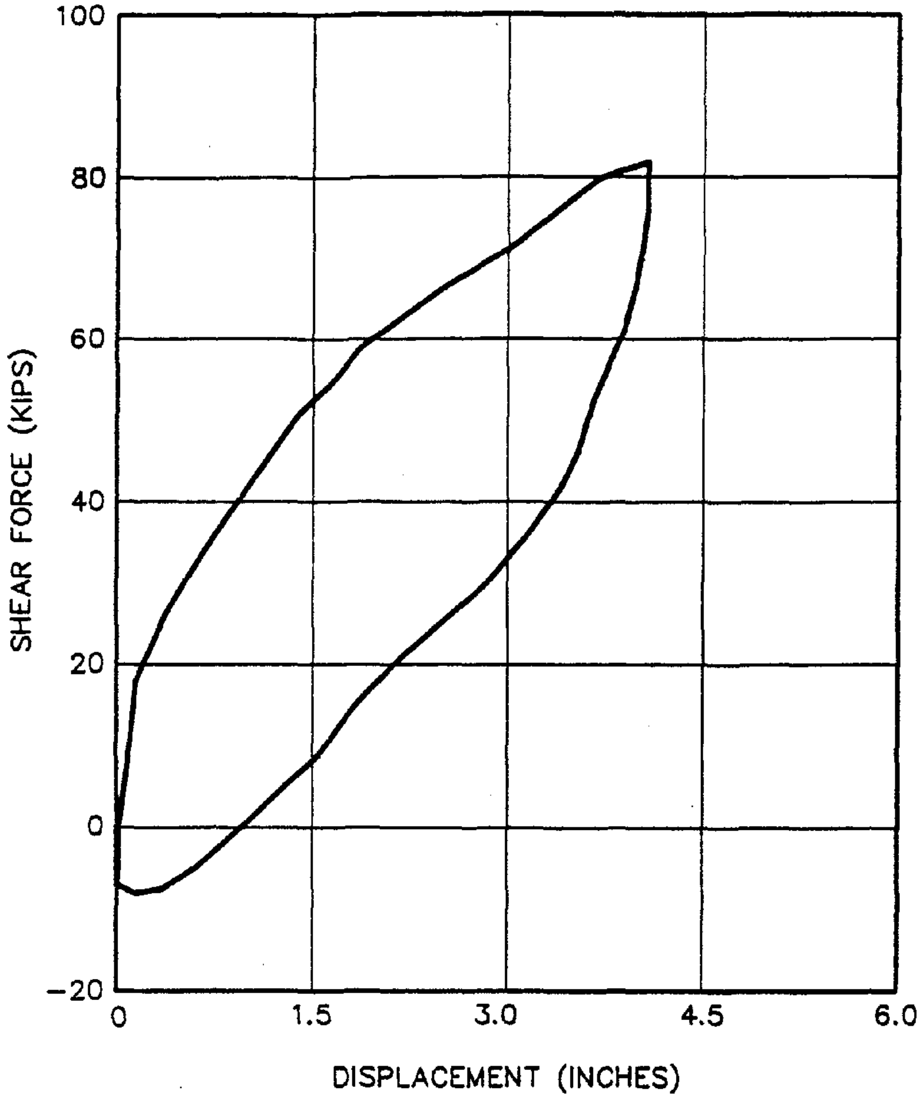
FIGURE 34
REINFORCED ELASTOMER BEARING



REFERENCE: REID AND TARICS, 1983

FIGURE 35
TYPICAL HIGH–DAMPING RUBBER BEARING IN THE
FOOTHILLS COMMUNITY LAW AND JUSTICE CENTER BUILDING

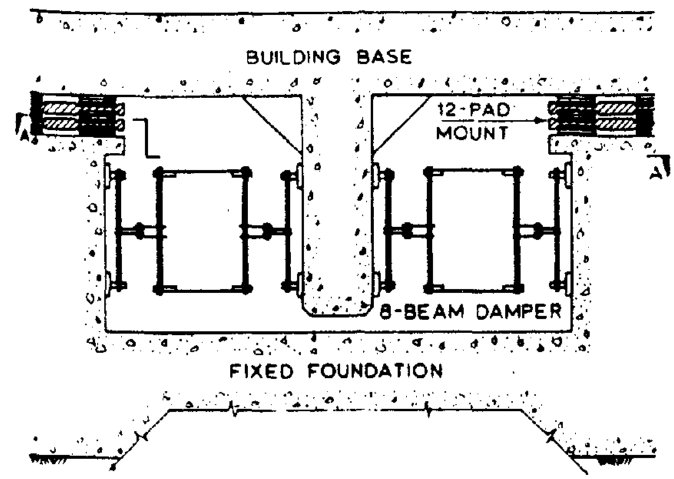
86-246-A9

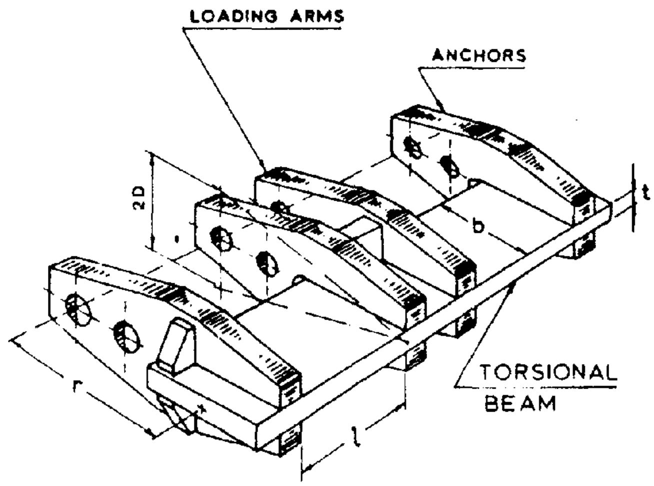


NOTE:
MAXIMUM STRAIN = 40 PERCENT
VERTICAL LOAD = 100 KIPS
BEARING DIA. = 30 INCHES
ERT = 12.0 INCHES

REFERENCE: REID & TARICS, 1983

FIGURE 36
SHEAR LOAD-DEFLECTION RELATIONSHIP OF HIGH DAMPING RUBBER BEARING USED IN FCLJC BUILDING

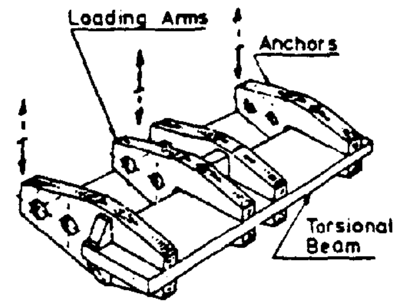




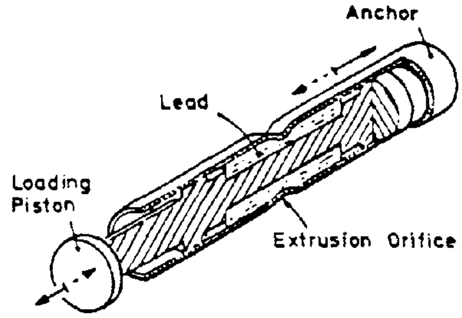
DETAIL—TORSIONAL BAR ENERGY ABSORBER

REFERENCE: SKINNER ET. AL., 1976

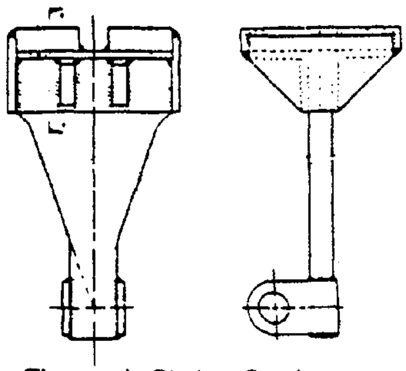
FIGURE 37
REINFORCED ELASTOMER BEARING WITH
EXTERNAL ENERGY DISSIPATING DEVICE



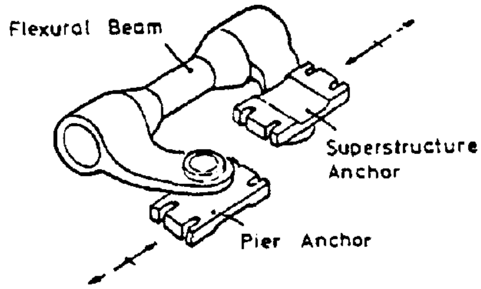
Torsional Beam Device



Lead Extrusion Device

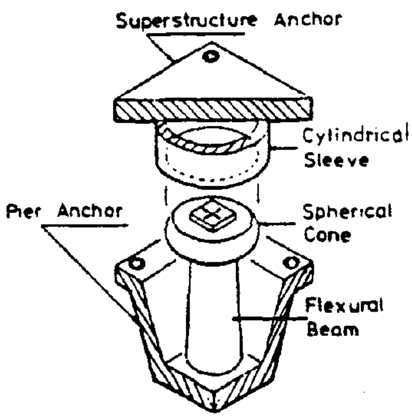


Flexural Plate Device

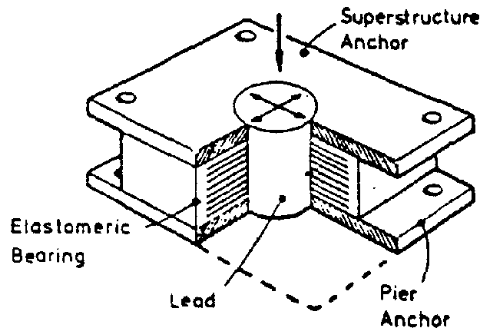


Flexural Beam Device

(a) Uniaxial Action



Flexural Beam Device

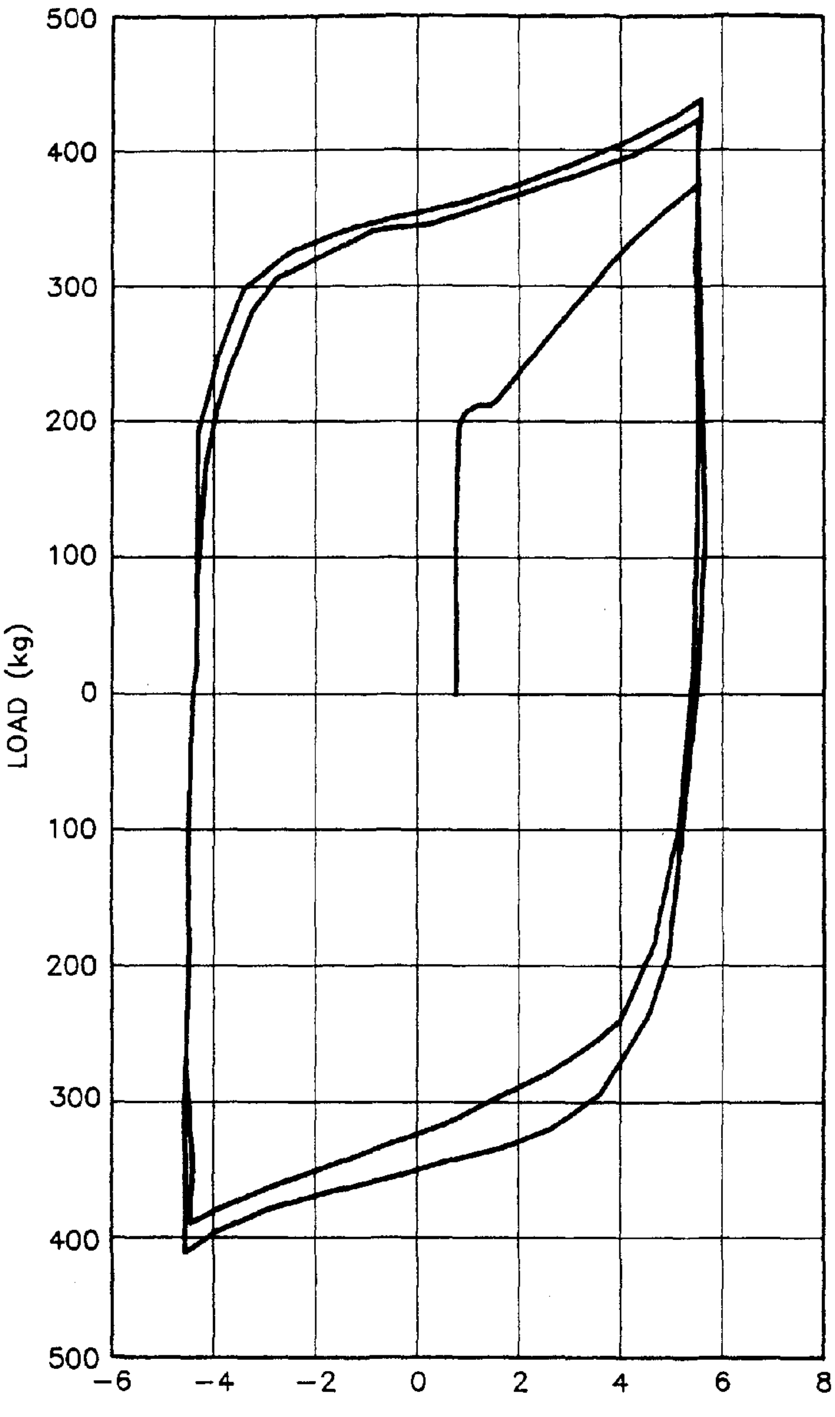


Lead—Rubber Device

(b) Omnidirectional Action

REFERENCE: BLAKELEY ET. AL., 1979

FIGURE 38
MECHANICAL ENERGY DISSIPATING DEVICES

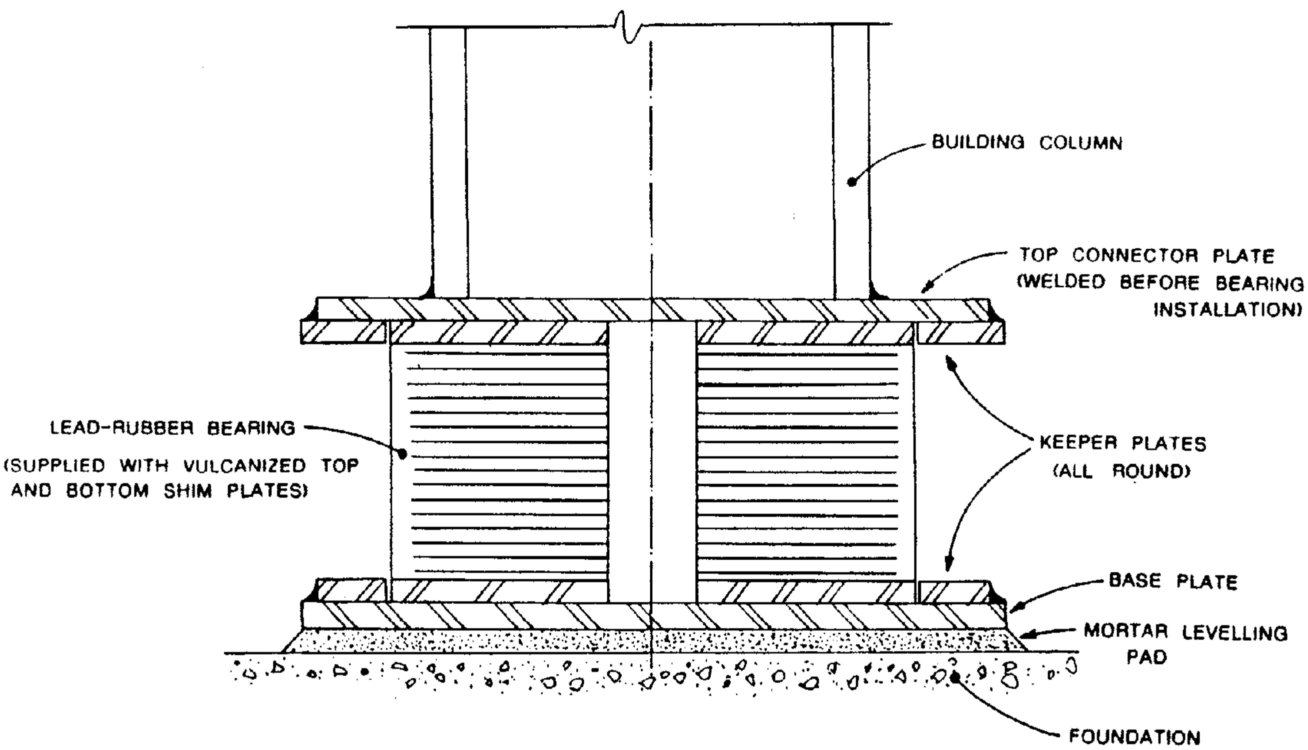


REFERENCE: KELLY ET. AL., 1972

NOTES:

MATERIAL: HOT ROLLED MILD STEEL
BAR SIZE: 1/2" SQUARE, 1" LONG
ENERGY ABSORBED PER CYCLE: 7500 IN−LB/IN^3
STROKE: ± 5 CM

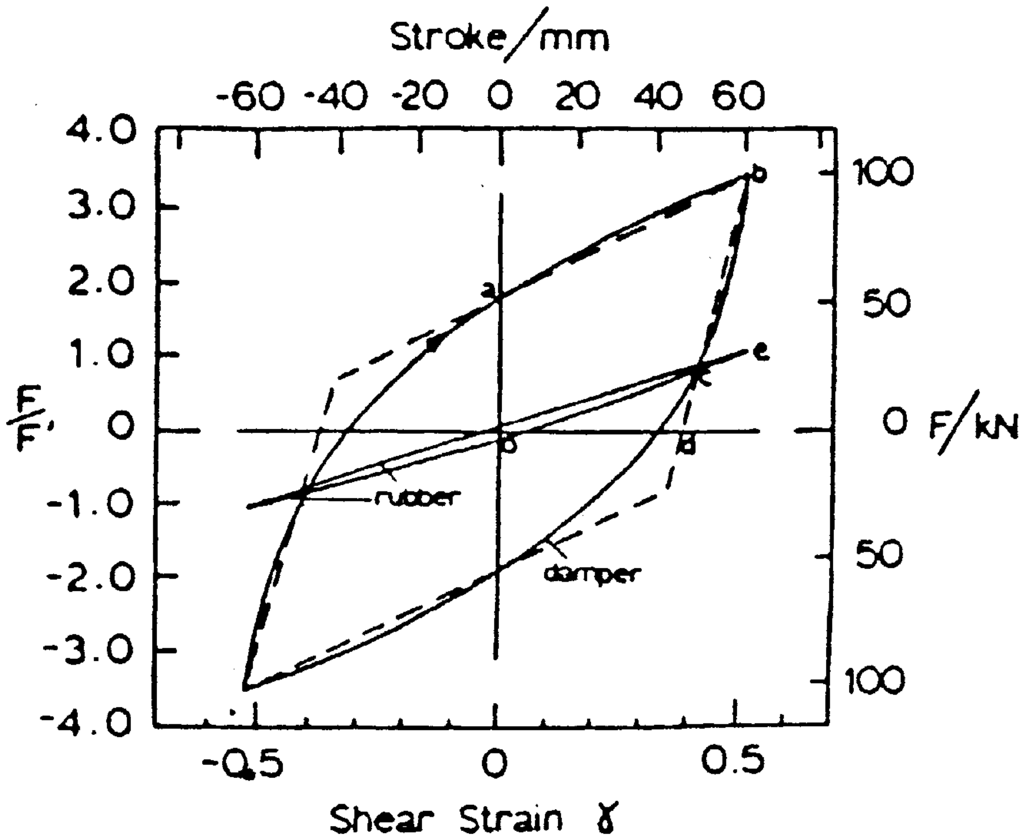
FIGURE 39
HYSTERIS LOOPS FOR A TORSION BEAM ENERGY DISSIPATING DEVICE



NOTE:
MORE COMMONLY A TYPICAL INSTALLATION INCLUDES DOWELS THROUGH THE TOP AND BOTTOM PLATES.

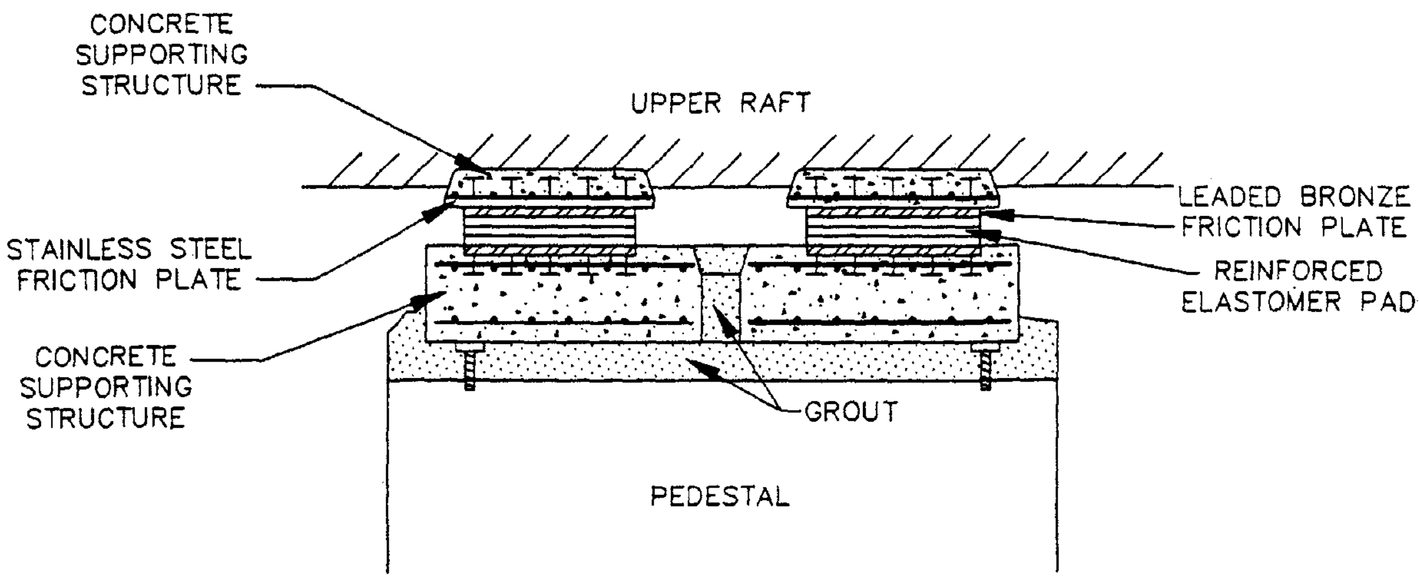
REFERENCE: DYNAMIC ISOLATION SYSTEMS, INC., 1984

FIGURE 40
LEAD–RUBBER BEARING INSTALLATION USING KEEPER PLATES



REFERENCE: ROBINSON AND TUCKER, 1977

FIGURE 41
TYPICAL FORCE–DEFLECTION HYSTERESIS LOOP FOR LEAD/RUBBER DEVICE



REFERENCE: PLICHON AND JOLIVET, 1979

FIGURE 42

FOUNDATION METHOD AT THE KOEBERG POWER PLANT, SOUTH AFRICA

86-246-A20

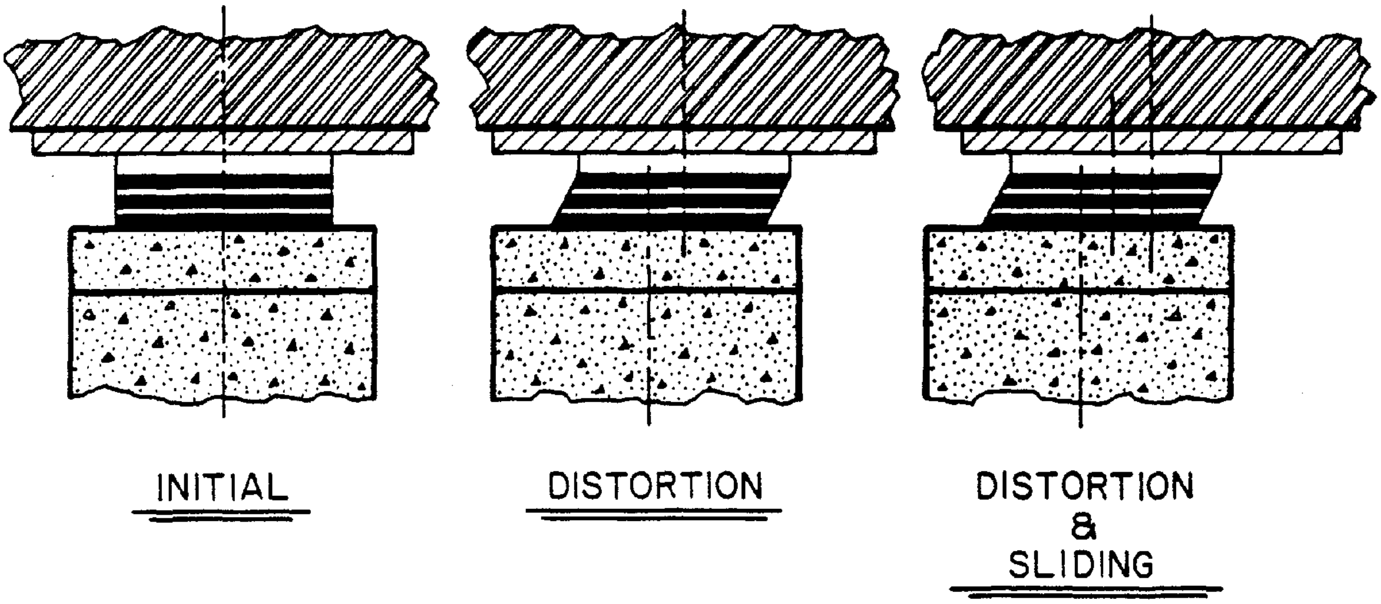
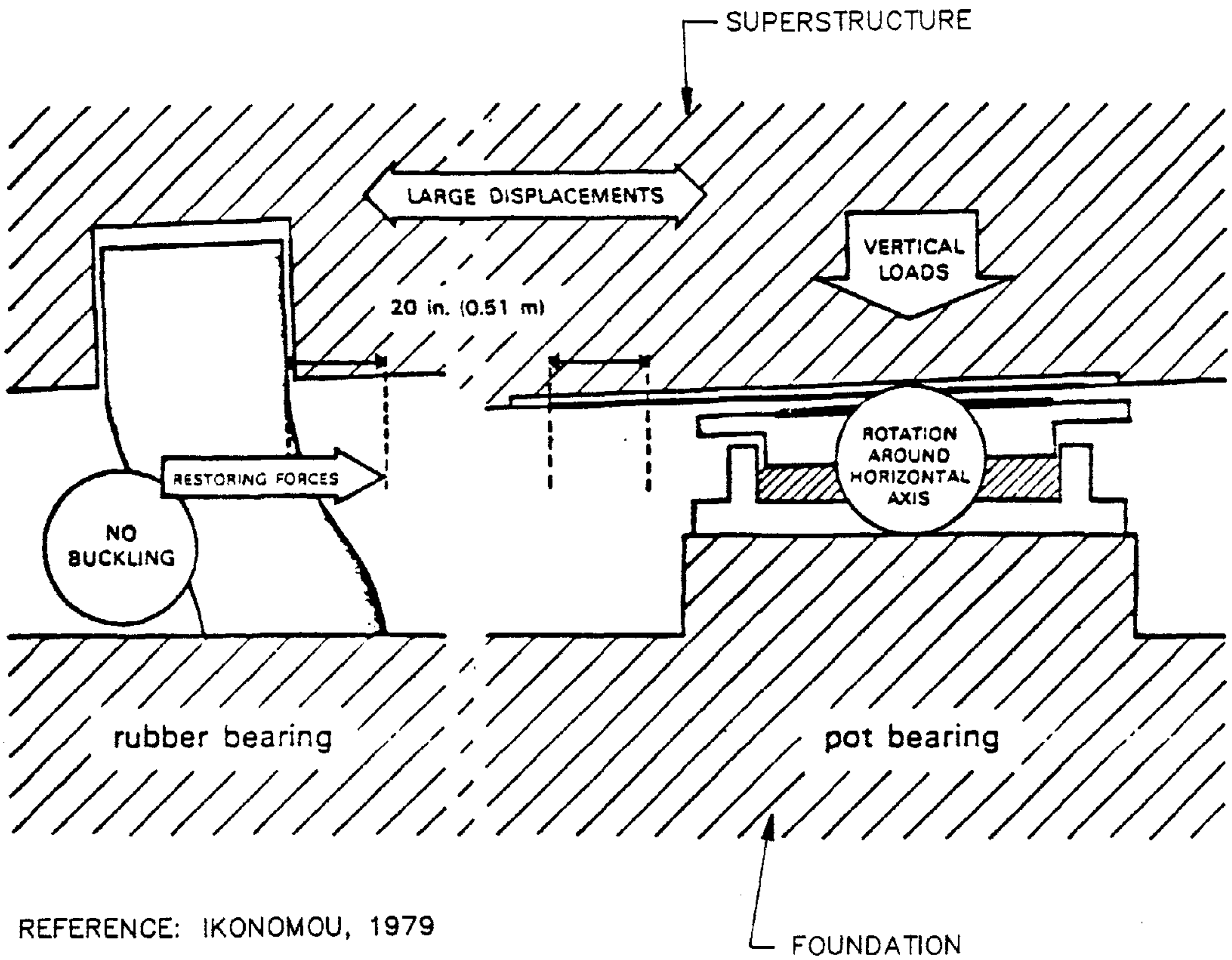


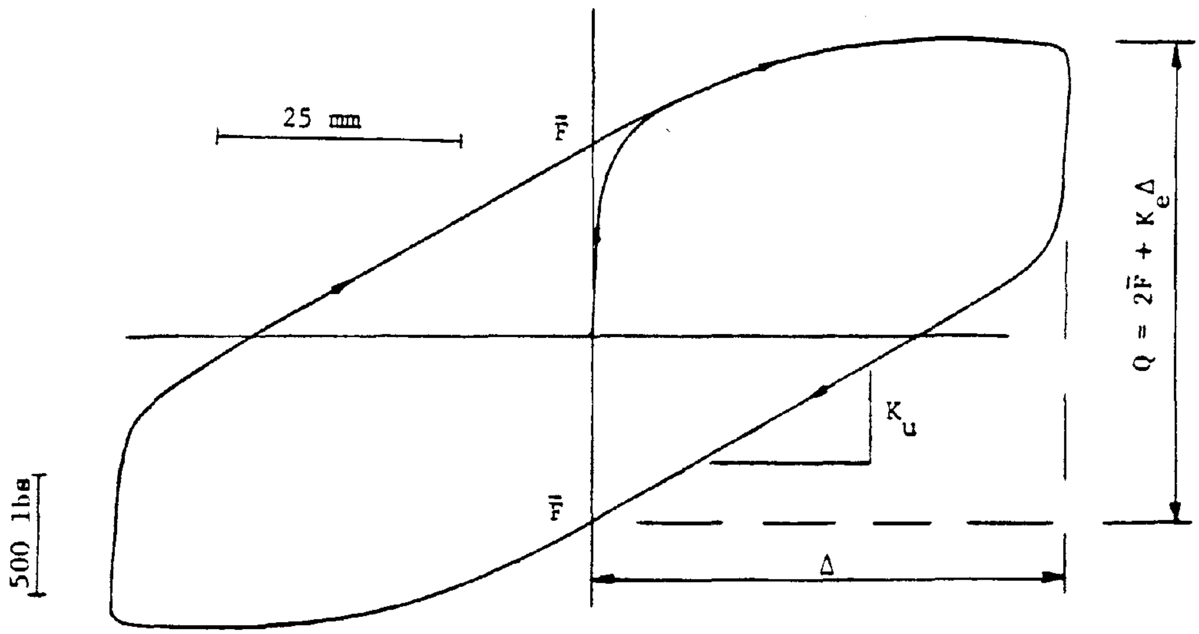
FIGURE 43
BEHAVIOR OF SUPERSTRUCTURE ON THE BEARINGS WITH
FRICTION INTERFACE

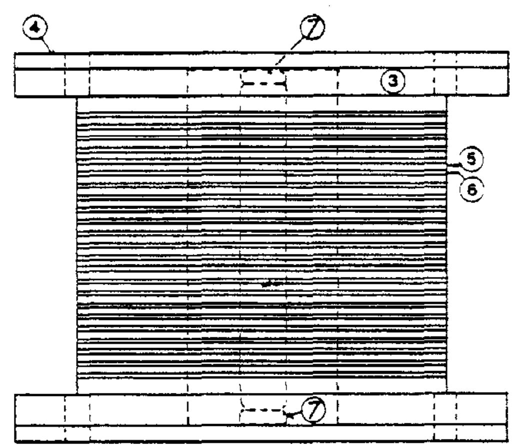
86-240-A26



REFERENCE: IKONOMOU, 1979

FIGURE 44
THE ALEXISISMON SYSTEM





REFERENCE: MOSTAGHEL AND FLINT, 1986

FIGURE 45
SHEAR LOAD–DEFLECTION RELATIONSHIP FOR
RESILIENT–FRICTION BASE ISOLATOR