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FEASIBILITY EVALUATION OF BASE ISOLATION FOR THE ASEISMIC DESIGN OF STRUCTURES

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16. Abstract (Limit: 200 words)

The development of base isolation for use in seismic protection of building structures is examined and the base-isolated design philosophy is discussed. The shape, size, and number of bearings used to isolate a structure are evaluated in terms of the structure's aseismic design requirements. The equations of motion for structures of base-isolation bearings are described, along with procedures for the step-by-step integration of the nonlinear equations of motion. Three case studies are presented to illustrate the base-isolated response of structures. It is concluded that base isolation is most effective in reducing seismic forces on rigid structures, with fundamental frequencies in the range of about 3 to 10 Hz. It is recommended that variability of building material properties and the dynamic characteristics of the building be included in a probabilistic analysis to develop risk assessment for base-isolated designs.

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REPORT

FEASIBILITY EVALUATION OF BASE ISOLATION FOR THE ASEISMIC DESIGN OF STRUCTURES

1.0 INTRODUCTION

Traditional methods of aseismic design rely on the strength and ductility of the structural elements comprising the building structure. Horizontal forces resulting from seismic ground motions are resisted by lateral load resisting systems such as ductile moment resisting space frames, braced frames, shear walls, or combinations thereof.

In general, the lateral load resisting systems are designed for prescribed minimum lateral loads (e.g., SEAOC, 1974; UBC, 1979), the philosophy being that the structure should, in general, be able to:

- Resist minor earthquakes without damage.
- Resist moderate earthquakes without structural damage, but with some nonstructural damage.
- Resist major earthquakes without collapse, but with some structural as well as nonstructural damage.

In recent years, an alternate method of aseismic design has been considered to satisfy the requirement of the above philosophy, namely that of base isolation. In this method of aseismic design, the entire structure is founded on several reinforced elastomer bearings which isolate the structure from severe horizontal seismic ground motions. As a result, not only are the seismic forces on the structural elements reduced but, more important, the motions and forces imparted to equipment and other nonstructural parts are also limited thus decreasing the potential for their damage and the resulting loss of function. The concept is illustrated in Figure 1 which shows cross sections through the base-isolated structures of the Koeberg Nuclear Power Plant in South Africa.

To date, a few structures around the world use base isolation for seismic protection (Kelly, 1979). These range from a simple two-story school building (Delfosse, 1977) to the more complex structures of a nuclear power plant (Jolivet and Richli, 1977). In the United States, with the exception of a 230-kv circuit breaker in California (Kircher, et al., 1980), there is no other significant structure which currently uses base isolation for seismic protection. The new Law and Justice Center Building in San Bernardino County, California is scheduled to be built on reinforced elastomer bearings for added seismic protection (ENR, 1983).

There is ample evidence of increased interest in this method from engineers, developers, and owners in the United States. This interest is a direct result of the reduction in the seismic risk and the potential cost savings which are offered by this method of aseismic design.

Base isolation in its present form uses the reinforced elastomer bearings. The reinforced elastomer bearing has been in use for the last 15 years or so, most commonly in bridges. Its use for seismic isolation of building structures is looked upon by some with some skepticism and, perhaps, justifiably so. Concern over the seismic response and long-term bearing performance are some of the reasons. Due to its recent origin, the profession has not had the opportunity to observe the behavior and examine the performance of base-isolated structures during earthquakes--a learning process that has contributed so significantly to understanding and rationalizing the behavior of conventionally founded structures.

It is generally accepted that mathematical models and modern analytical techniques can be used to reasonably predict the seismic response of building structures (Sharpe, et al., 1973). It is felt that this is particularly true for a base-isolated structure because its seismic response demands very little participation from the structures, it's structural details, and the nonstructural elements.

A significant amount of experimental data has been generated from shaker table tests (Kelly, 1979; Pavot and Polus, 1979) on base-isolated structural models. These tests have shown good correlation with analytically predicted response and they further demonstrate the ability of the reinforced elastomer bearings to reduce earthquake motions transmitted to the structure and its contents.

Although the base-isolation concept has been examined in depth for a few structures (Richli, et al., 1980; Skinner and McVerry, 1975), it is reasonable to expect that further work could address issues concerning the general applicability of base isolation as an aseismic design strategy. The present research is intended to contribute towards this goal. Its objectives are the following:

- Study the dynamic behavior of base isolated structures subjected to seismic ground motions.
- Develop an appreciation of the practical problems involved in the incorporation of base isolation in the foundation design.
- Study the cost and benefits derived from base isolation.
- Establish some guidelines for the selection of structures that are likely to benefit from base isolation.
- Establish simple design rules which could be used to evaluate the feasibility of base isolation in the preliminary design stage.

After a brief background in Chapter 2.0, the base-isolated design philosophy is described in Chapter 3.0. Prominent base-isolation schemes are described in Chapter 4.0, followed by a theoretical discussion on the dynamic behavior of base-isolated structures in Chapter 5.0.

Chapter 6.0 examines three case studies illustrating the base-isolated response of structures. The base-isolation design considerations and impact on structural costs are examined in Chapter 7.0. Chapter 8.0 describes the results of a probabilistic analysis of seismic response of base-isolated structures. Finally, Chapter 9.0 contains concluding remarks and suggestions for areas in which further work would be helpful.

2.0 BACKGROUND

The development of base isolation for use in seismic protection of building structures historically proceeded from considerations of such concepts as roller bearings, suspended supports, plain elastomer pads (Newmark and Rosenblueth, 1971), the soft first-story concept (Chopra, et al., 1973), mechanical springs (Agbabian, 1979), and reinforced elastomer pads (Plichon, 1975; Skinner and McVerry, 1975; Derham, et al., 1974). Some earlier concepts did not prove successful, perhaps due to shortcomings in the resulting seismic response and/or the strength characteristics of the bearings. Several concepts of base isolation and a few case histories where these have been used are described in the literature (Kelly, 1979; Dames and Moore, 1979).

Some recent structures which have been designed for seismic protection using base isolation include the Koeberg Power Station in South Africa, Kanun River Power Plant in Iran (D'Appolonia, 1979; Newmark, 1979), and Cruas Power plant in France (D'Appolonia 1980); the William Clayton Building in Wellington, New Zealand (Megget, 1978), a 230-kv circuit breaker in California (Kircher, et al., 1980), and the Law and Justice Center Building in California (ENR, 1983). As an example, the 230-kv circuit breaker is shown in Figure 2. This is a rather unique case in that the original structure, which was conventionally founded, was retrofitted with base-isolation bearings and served as a test structure.

The prominent base isolation systems currently in use employ reinforced elastomer bearings to support the weight of the structure. This type of bearing provides the necessary compressive strength and stiffness to carry the gravity building load and at the same time is relatively flexible in the shear (lateral) mode to allow horizontal motion.

Reinforced elastomer bearings have been used as a structural element to support bridge decks on the piers and simultaneously allow horizontal movement due to thermal expansion of the bridge deck. One of the earliest such applications is a reinforced concrete bridge in Victoria County, Texas. Reinforced neoprene bearings were installed here about 25 years or so ago and to date are in place and performing satisfactorily with no signs of degradation. Since that time, several bridges both in the United States and abroad have used reinforced elastomer bearings because of their durability and low maintenance requirement. A number of analytical and experimental studies have demonstrated the load carrying and deformation capability of such bearings (Lindley, 1962; Stanton and Roeder, 1982). The material, manufacturing, and testing requirements for reinforced elastomer bridge bearings are discussed in ASTM Specification D 4014-81 (ASTM, 1981). Reinforced elastomer bearings have also been successfully used to isolate building structures from ambient vibrations; for example, Albany Court in London which is supported on reinforced elastomer bearings to reduce the vibrations caused by the underground railway (Waller, 1969). The wide use of reinforced elastomer bearings in these and other structural applications has contributed to the confidence among some engineers that such bearings can also be used for seismic protection of building structures.

3.0 DESIGN PHILOSOPHY

The code minimum horizontal forces for which conventional structures are designed do not represent the level of motion attainable by the structure during the design seismic event. Rather, these forces reasonably assure a seismic performance of the structure which, in general, will preclude collapse in the event of a major earthquake but will allow nonstructural as well as some structural damage. Several reasons justify this approach. These include effective peak versus peak ground motion, averaging effects due to the physical size of the structure, effects of nonstructural elements which are normally not considered in the mathematical model, inherent damping, and reserve capacity of structural elements in the inelastic range.

Observations of the performance of various structures in earthquakes coupled with the need for a rational design process which will result in an economical structure have led to the acceptance of the above reasons. However, the reasons are accepted along with some degree of uncertainty and consequent risk. Therefore, a good understanding of the bases and the associated uncertainties is demanded of the structural engineer for proper application of the recommended minimum lateral loads.

Base isolation is primarily an attempt at reducing the effects of some of the uncertainties and thus mitigating the potential risk resulting from structural and nonstructural damage. It's basic premise is to minimize participation of the structural modes of vibration in the overall seismic response and force the base-isolation bearings to take up and dissipate the major portion of the seismic energy imparted to the structure. In effect, the earthquake forces transmitted to the structure are thus reduced with the result that the structure can now be designed, within economic constraints, to remain elastic and reduce potential repair costs and consequential down time, or the replacement cost of the facility.

The design strategy is to impart a low frequency to the predominant horizontal mode of the base-isolated structure. This mode is the horizontal motion of the structure on the isolation bearings. The frequency for this mode is chosen to be out of the range of frequencies normally encountered in seismic ground motions. The low horizontal frequency causes relatively little amplification of ground acceleration, thus effectively filtering out the high frequency ground motion which is predominantly responsible for earthquake-induced damage. In order to meet the requirements of this design approach, the bearings used to support the structure must be relatively flexible in the horizontal direction but must possess adequate strength and stiffness in the vertical direction to be able to support the gravity load of the structure.

Since the participation of structural modes is reduced, the response of the structure above the base-isolation bearings approaches that of a rigid body. This results in a decrease of overturning moments and interstory drift. Further, the motions imparted to the contents are therefore reduced quite significantly, especially at upper floors of the structure.

Qualitatively, base isolation can be seen to benefit low to intermediate height structures or, in general, structures whose fundamental mode is in the range of predominant earthquake frequencies when the structure is conventionally founded. As the structure gets more flexible and its fundamental mode decreases, the benefits of base isolation are seen to reduce. This is discussed in more detail in a subsequent section.

4.0 BASE-ISOLATION BEARINGS

Most modern base-isolation bearings use reinforced elastomers, shown for example in Figure 3. This type of bearing is made of alternating layers of elastomer and steel bonded together in a vulcanization process.

The horizontal stiffness of the bearing is inversely proportioned to the total elastomer thickness while the vertical stiffness is proportional to the shape factor of the bearing. The shape factor is defined as the ratio of the loaded area to the area free to bulge. In the case of the reinforced elastomer bearing shown in Figure 3, the area free to bulge comprises the edges of the elastomer layer between two steel plates. The vertical stiffness increases as the shape factor increases. Compressive stress strain curves of such bearings are shown, for example, in Figure 4 (DuPont, 1983).

The parameters that define the bearing design are thus the shape factor which determines the vertical stiffness and the allowable compressive stress on the bearing, the effective elastomer thickness which determines the horizontal stiffness, and the hardness of the elastomer which defines its shear modulus. The shape, size, and number of bearings used to isolate a structure can be established in accordance with its aseismic design requirements.

In general, elastomers are not linear materials. The shear modulus, for example, varies with the shear strain, compressive stress, and the frequency of load application. However, for the loading conditions encountered in a seismic event, the assumption of constant shear modulus is anticipated to give good estimates of quantities required in the seismic design of the structure and the bearing (Derham and Thomas, 1981).

The reinforced elastomer bearing has an inherent material damping associated with the elastomer which is generally in the range from about 5 to 10 percent. In order to prevent excessive displacements of the base-isolated structure, other devices which act in conjunction with the bearing have often been considered to increase the energy absorption in the system (Kelly and Tsztoo, 1978; Skinner, et al., 1980; Plichon, 1975). The resulting base-isolation designs can be generally divided into the following categories:

- Reinforced elastomer bearings,
- Reinforced elastomer bearings with an additional damper in parallel, and
- Reinforced elastomer bearing with a slip surface.

The first of the above was illustrated in Figure 3. In the second type, the additional damper consists of either a mechanical fuse or a lead plug, as illustrated for example in Figure 5. Both these devices provide a damping force resulting from a hysteresis loop as they undergo inelastic distortions. This damping force acts in parallel with the horizontal bearing stiffness.

The third type of bearing is shown in Figure 6. In this type, the bearing is not fixed to the superstructure, but forms a frictional slip interface with it. The coefficient of friction at the interface is chosen so that small to moderate earthquakes are accommodated by the elastic distortion of the bearing pad. Severe earthquakes are accommodated both by the elastic distortion and slip on the frictional interface. This results in additional damping from the hysteresis loop described by the structure as it undergoes slip during the seismic event. The damping force in this case acts in series with the shear stiffness of the bearing. In one system (Plichon, 1975), the chosen friction couple consists of a lead-bronze alloy plate bonded to the bearing and a stainless steel plate embedded in the superstructure. Tests have shown that this couple provides a fairly constant coefficient of friction under a wide range of vertical pressures and relative velocities of the surface.

5.0 SEISMIC BEHAVIOR OF BASE-ISOLATED STRUCTURES

The use of reinforced elastomer bearing pads results in a nonlinear response to the horizontal ground motions, especially as these seismic motions become severe. The nonlinearity arises due to the inherent nonlinear behavior of elastomers and the response associated with the additional damping devices. The computation of the seismic response therefore requires nonlinear time-domain analysis. This section describes the equations of motion for structures on base-isolation bearings along with procedures for the step-by-step numerical intergration of the nonlinear equations of motion.

5.1 MATHEMATICAL MODEL

In evaluating the seismic response of multistory structures on base-isolation bearings, it is assumed that all points at the base of the structure are subjected to identical earthquake ground motion. Although the bearings are distributed over the foundation area, it is assumed that all the bearings may be represented by a single stiffness with a nonlinear interface at the base of the structure. This assumption is evaluated later. The structure itself is represented as a shear beam with the building masses lumped at each floor. The spring constant, k_i, as shown in Figure 7, describes the lateral force required to produce a unit relative lateral displacement of story i.

The analysis of the structure as a shear beam is appropriate for many structures where the primary lateral resistance is from moment-resisting frames. For moment-resisting frames with infinitely stiff girders, the effective story stiffness is determined by the sum of the fixed end lateral stiffness of all the columns in a story. For frames with flexible girders, the story stiffness may be approximated by assuming equal joint rotations with inflection points at midheight of columns and midspan of girders. This gives a good approximation where the first mode of the structure dominates.

The shear beam approach is an approximation for structures with shear walls of dimensions such that shear deformations dominate bending deformations. The assumption of a shear beam structure is made to simplify the form of the structure stiffness matrix and the extension to structures including rotational degrees of freedom at each story is straightforward. However, for cases of base isolation where participation of higher modes is generally small, the shear beam model is a reasonable approximation.

5.2 EQUATIONS OF MOTION

The equations of motion for the relative displacements of the floors of the structure, shown in Figure 7, may be written in the following form

$$[M]\ \{\ddot{x}\}_t + [C]\ \{\dot{x}\}_t + [K]_t\ \{x\}_t = -[M]\{r\}\ddot{x}_g(t)$$

where all matrices and vectors in this equation are evaluated at time t and

- $[M]$ is the mass matrix,
- $\{r\}$ is a vector of showing the direction of ground movement. $r_i = 1$ for horizontal degrees of freedom and $r_i = 0$ otherwise,
- $[C]$ is the viscous damping matrix,
- $[K]_t$ is the stiffness matrix,
- $\{x\}_t$ is the vector of floor displacements relative to the ground, and
- $\ddot{x}_g(t)$ is the horizontal acceleration of the ground at time t.

Since the mass is lumped at each floor, the mass matrix is diagonal and is fixed with respect to time.

The aseismic bearing has been idealized with a general representation of a linear spring in series with a nonlinear slip surface. In reality, inelastic behavior of the neoprene as well as any external damping mechanism can be expected to result in hysteretic damping or energy absorption within the pads. This hysteretic damping has been replaced by the equivalent viscous pad damping, as shown in Figure 7. Under these assumptions, the stiffness matrix of the pads is bilinear with initial stiffness defined by the elastic stiffness of the neoprene pads. The stiffness matrix of the complete structure, as shown in Figure 7, is then bilinear and is assembled from the linear stiffness matrices of the shear beams plus the stiffness of the bearings. Since the structure has been assumed to behave as a shear beam, the stiffness matrix has tri-diagonal form. Damping is included within the structure through the form of Rayleigh damping, i.e.,

$$[C] = \alpha\ [M] + \beta\ [K]$$

A more complete discussion of the choice of damping parameters is included with the discussion of the examples to follow.

5.3 ANALYTICAL METHOD

The equations of motion are solved by step-by-step integration procedures. For the step-by-step procedure, with constant tangent stiffness $[K]_t$ over the time interval Δt, the equations of motion take the form:

$$[M]\ \{\ddot{x}\}_{t+\Delta t} + [C]\ \{\dot{x}\}_{t+\Delta t} + [K]_t\ \{\Delta x\} = -[M]\ddot{x}_{g(t+\Delta t)} - \{F\}_t$$

where

$\ddot{x}_g(t+\Delta t)$ is the horizontal ground acceleration at time, $t+\Delta t$,

Δt is the time step,

$\{F\}_t$ is the force vector equivalent to the shear forces and pad force at time, t, and

$\{\Delta x\}$ is the vector of floor displacement increments, i.e., $\{x\}_{t+\Delta t} = \{x\}_t + \{\Delta x\}$.

The solution of the incremental equation of motion is readily obtained with the Wilson-Theta or Newmark methods.

For the case of the bearing with a frictional slip surface, the stiffness of the bearings changes when the pads start and stop slipping. At these transition points, the calculated element forces may be out-of-balance with respect to the calculated inertial and damping force. It is assumed that the time step is small enough to limit error due to this out-of-balance force during the transition from slip to nonslip and vice versa; otherwise, it may be necessary to perform iteration using the unbalanced forces to obtain corrections to the displacement increments obtained from the solution of the step-by-step equations given previously. In general, the time step should be established by trial and error, until the use of a smaller time step does not significantly alter the results.

In the following chapter, some examples are analyzed using the above analytical procedure.

6.0 CASE STUDIES

To illustrate and evaluate the effect of base isolation on the overall seismic response, three case studies were examined. The structures selected for these case studies represented lateral force resisting systems, respectively, load bearing shear walls, a ductile moment-resisting space frame, and a combination of shear walls and a space frame. Further, the structures selected for the case studies were relatively rigid (fundamental frequency in the ranges from about 3 to 5 Hertz [Hz]) when conventionally founded.

6.1 GENERAL

In each case the conventional design was first analyzed to compute lateral loads in accordance with the SEAOC Code (SEAOC, 1974). These loads are representative of those that would normally be used in the design of the lateral load resisting system. The conventional design was then analyzed to compute the lateral loads that the system would experience if all the components of the structural system were to remain linearly elastic. An appropriate ground motion time history, as explained below, was used for this purpose. It should be noted here that for the conventional design the lateral loads computed on the linear assumption are unrealistic as the anticipated inelastic action of some of the structural components will provide additional damping in the system. Nevertheless, these forces are presented for illustrative purposes and to facilitate comparison with the time-history response of the base-isolated structures.

Each structure was then examined with reference to its mass and stiffness characteristics and an appropriate base-isolation scheme was designed. Basically, the base isolation was designed to result in a fundamental mode frequency of about 0.75 Hz. This frequency was chosen on the basis that it should be out of the range of frequencies generally encountered in earthquake ground motions so as to result in effective isolation, that it should not be too low such as to result in a high displacement response, and that with respect to the vertical load supported, it should be practically achievable with the use of reinforced elastomer bearings.

With the above basic base-isolation characteristics, each structure was analyzed for a different damping mechanism as follows:

- Reinforced elastomer bearings alone with a material damping of about 5 percent.
- Reinforced elastomer bearings with an additional damping device to yield a total of 15 percent damping.
- Reinforced elastomer bearings with a frictional interface with a coefficient of friction equal to 0.2.

The first case represents a fixed bearing pad with which no additional damping device is provided. A material damping of five percent in the elastomer is generally accepted, although the elastomers may be compounded to provide a somewhat higher material damping. The second case was analyzed to represent a fixed reinforced elastomer bearing with either a mechanical fuse in parallel or a lead insert in the bearing. The use of a 15 percent damping for the analysis is not meant to suggest any limitation on the damping capability of this type of bearing. Indeed, a higher damping could be justified by appropriate tests and analyses (Kelly and Hodder, 1971). The third case represents a reinforced elastomer bearing with a frictional slip surface. In addition to the attenuation of seismic motions due to the low frequency, the friction surface further limits the seismic motion in accordance with the coefficient of friction. In the present analysis, a coefficient of friction equal to 0.2 was assumed.

The following sections discuss the seismic input, details of the case study structures, and the resulting response.

6.2 SEISMIC INPUT

The manner in which a base-isolated structure is anticipated to respond during an earthquake is significantly different than the way a conventional structure responds. Consequently, the Code method of computing lateral loads and their distribution cannot be applied to the base-isolated structure, and a more rational procedure usually is necessary. A more direct approach using a ground motion time history and an appropriate mathematical model was used herein to compute the seismic response of the base-isolated case study structures.

For the purpose of the case studies, the structures analyzed were assumed to lie in Earthquake Zone 4, as defined by the SEAOC Code (SEAOC, 1974). Zone 4 is that area in California subjected potentially to the most severe earthquake shaking. Its boundaries are defined as being about 25 miles for a potential Magnitude 7 or greater earthquake and about 15 miles for a Magnitude 6 to 7 earthquake. Zone 4 was specifically chosen for the present case studies so as to examine the feasibility of base isolation for the more severe earthquakes, and to clearly bring out the differences in seismic responses and resulting seismic design quantities for conventional and base-isolated structures.

In accordance with the definition of Zone 4, the El Centro S00E record, which was obtained from a Magnitude 6.5 event at an epicentral distance of about nine kilometers, was chosen to be representative of the expected ground motions. It's acceleration time history and response spectra are shown in Figure 8. The El Centro record has peak ground motion parameters of 0.348g acceleration, 13 in/sec velocity, and 4.3 inches of ground displacement. Indeed, within Zone 4, these peak ground motion parameters are likely to be exceeded. To account for this possibility, the peak parameters were arbitrarily increased, and the ground motion time history for the case studies was defined by scaling the El Centro record to a peak acceleration of 0.6g.

6.3 CASE STUDY I - BUILDING 1

6.3.1 Building Configuration

The structure for this case study was taken from "Seismic Design for Buildings," NAV FAC P-355. It is a three-story administration building with reinforced concrete bearing walls and a series of interior, vertical load-carrying columns, and girder bents. As shown in Figure 9, the structure is 48 feet by 192 feet in plan.

The second and third floors are comprised of metal decking with concrete fill while the roof is metal decking with insulating board. The interior columns are founded on spread footings, and the load-bearing walls are founded on strip footings. The first floor comprises a concrete slab on grade. For the seismic analysis, 100 percent of the dead load plus 50 percent of the transient live load is assumed to exist at the time of the earthquake.

6.3.2 Base-Isolation Design

A possible method of incorporating base isolation for this building is illustrated in Figure 10. In this scheme, the structural columns and shear walls which form part of the superstructure bear on a grid of tie beams. The tie beams are supported by several base-isolation bearing pads which are, in turn, supported by a system of grade beams and strip footings. This scheme thus requires a structural slab at the first floor where, in the conventional case, a slab on grade existed. The total weight supported by the isolation bearings will therefore include the load from the first floor, also. For the present analysis, the first floor load is conveniently assumed to be equal to the second and third floors.

A typical base-isolation bearing for the above system is shown in Figure 11. The bearing is 15 inches square in plan and consists of 8 layers of a 50-durometer elastomer, each 3/8-inch thick, reinforced with 7 layers of 1/8-inch-thick steel shims. The bottom of the bearing is formed with a 1/2-inch-thick steel plate, and the top would consist of either a steel-bearing plate or a frictional surface in accordance with the design adopted. Each bearing has a horizontal stiffness of 11.25 kips/inch and is capable of sustaining a vertical load of about 1,100 pounds per square inch (psi). Twenty-one (21) such bearings are required to support the superstructure, and each is located along column lines and rows.

Proceeding on the above basis, the total horizontal bearing stiffness is 2,835 kips/foot, and the total building weight is 5,129 kips, resulting in a fundamental frequency of about 0.67 Hz.

6.3.3 Mathematical Model

The resulting mathematical model for the north-south direction used in subsequent response calculations in the north-south direction is shown in Figure 12. The model consists of concentrated masses at each floor location connected by the respective story stiffnesses. In addition to the horizontal bearing stiffness, the base-isolation representation includes a damper and a frictional interface. The various types of base-isolation schemes can be represented by this mathematical model. The conventional foundation can be represented by this model by assigning an arbitrarily large value to the horizontal bearing stiffness.

6.3.4 Seismic Response

In computing the Code minimum lateral loads, the K-factor was taken as 1.33 since the structure is without a complete load-carrying space frame; Z for Zone 4 is 1.0; and the importance factor is arbitrarily assigned as 1.5. The resulting total base shear in the north-south direction is about 990 kips which is distributed over the height of the structure in accordance with the Code procedure.

The story shears and overturning moments for the various cases analyzed are shown in Figure 13. It is seen that, in the base-isolated cases, the shears and moments for a 0.6g input are roughly the same order of magnitude as the Code minimum for Zone 4. It is also seen that for this structure the bearing pad with a frictional interface effects the largest reduction of seismic forces on the structure.

The displacements and accelerations over the height of the structure are shown in Figure 14. It is seen that the base-isolated response is more or less uniform over the entire height suggesting a rigid-body response of the structure above the base isolation with very little participation of the structural modes. Although the total displacements in the base-isolated cases are larger than the conventional case, the interstory drifts are smaller. The above response quantities are summarized in the following table:

TABLE 6-1

RESPONSE QUANTITIES
CASE STUDY I

DESIGN	BASE SHEAR (kips)	OVERTURNING MOMENT (kip-ft)	MAXIMUM ACCELERATION (g's)	INTERSTORY DRIFT (in.)
Conventional				
SEAOC (Zone 4)	987	23,190	-	-
Linear Elastic(1,2)	2,249	45,476	0.75	0.009
Base Isolated(2)				
5% Damping	1,230	24,221	0.337	0.005
5% Damp + Friction(3)	845	16,682	0.235	0.003
15% Damping	928	18,265	0.254	0.004

(1) Provided as an approximate measure of linear elastic displacements.
(2) Based on El Centro 1940 S00E record, 0.6g peak.
(3) Maximum slip = 2.2 inches.

In the case of the bearings with the frictional interface, the relative displacement and total slip time histories are shown in Figure 15.

These time histories show the buildup of permanent slip which attains a maximum value of 2.2 inches.

6.4 CASE STUDY II - BUILDING 2

6.4.1 Building Configuration

The structure for this case study is the same basic building as in Case Study I. It is a three-story administration building with a structural steel ductile moment-resisting space frame without shear walls. The exterior walls are nonbearing and nonshear metal panels. A series of interior vertical load-bearing columns and girder bents support the floors. The structural concept is shown in Figure 16.

The second and third floors comprise metal decking on structural beams with concrete fill, the roof is metal decking with insulating board, and the first floor consists of slab on grade.

6.4.2 Base-Isolation Design

A conceptual design of base isolation for this building follows the same principles illustrated in Figure 10. In this design, the superstructure columns are supported on concrete bearing blocks which are supported by the base-isolation bearings. The foundation consists of individual piers which carry the vertical and horizontal bearing loads. The piers are, in turn, supported on strip footings and interconnected by a grid of grade beams. In this scheme, the first floor is a structural slab possibly of the same design as the second and third floors.

A typical bearing which satisfies the isolation requirements for the above building is shown in Figure 17. The bearing is 15 inches square in plan and consists of six 3/4-inch layers of 50-durometer elastomer reinforced with five 1/8-inch steel shims. The horizontal stiffness of each such bearing is 7.5 kips/inch, and each bearing is capable of sustaining a safe vertical load of about 600 psi. Twenty-one (21) such bearings are required and result in a fundamental horizontal frequency of about 0.75 Hz.

6.4.3 Mathematical Model

A representative mathematical model for this case study base-isolated structure in the north-south direction, which was used in subsequent response calculations, is illustrated in Figure 18. In this representation, the interstory stiffnesses connecting the floor masses were evaluated on the basis of a finite element model of the moment-resisting space frame. For convenience, the structural steel beams supporting the first floor were assumed to be identical to those on the second floor. Column bases are assumed to be resting on the bearings so as to allow rotations of the column ends and potential uplift.

6.4.4 Seismic Response

The Code minimum lateral loads were computed on the basis of a K-factor of 0.67 and Z of 1.0. As in Case Study I, the importance factor was arbitrarily assigned as 1.5, resulting in a total base shear of about 270 kips for the north-south direction.

The story shears and overturning moments for the various cases analyzed are shown in Figure 19. The base-isolated shears and moments computed for 0.6g input are somewhat larger than the Code minimum for Zone 4, and they are about equal for the different base-isolation strategies, the 15 percent damped bearing effecting the largest reduction in the forces for this case study structure.

The displacements and accelerations over the height of the structure are shown in Figure 20. Again, the base-isolated response is more or less uniform over the height, perhaps a little less so than in Case Study I, but nevertheless significantly uniform when compared to the response in the conventional case. Because the moment-resisting frame is more flexible than a shear wall, the first structural mode of the frame is seen to participate more in the overall response. The various response quantities are summarized in Table 6-2 below:

TABLE 6-2

RESPONSE QUANTITIES
CASE STUDY II

DESIGN	BASE SHEAR (kips)	OVERTURNING MOMENT (kip-ft)	MAXIMUM ACCELERATION (g's)	INTERSTORY DRIFT (in)
Conventional				
SEAOC (Zone 4)	271	6,294	-	-
Linear Elastic[1,2]	2,640	60,876	2.845	1.73
Base Isolated[2]				
5% Damping	614	12,747	0.446	0.658
5% Damp + Friction[3]	559	12,480	0.446	0.423
15% Damping	548	11,793	0.436	0.584

(1) Provided as an approximate measure of linear elastic displacements.
(2) Based on El Centro 1940 S00E record, 0.6g peak.
(3) Maximum slip = 2.3 inches.

For the case of base isolation with the frictional interface, the relative displacement and total slip time histories are shown in Figure 21. The maximum value of slip in this case is about 2.3 inches.

6.5 CASE STUDY III

6.5.1 Building Configuration

For the third case study, the Veterans Administration Hospital in Loma Linda, California was chosen. In addition to illustrating its base-isolated seismic response, the intent in choosing this structure was also to assess the impact of base isolation on the design of the structural elements and their cost. This aspect is discussed in detail in Chapter 7.0.

The SEAOC forces have been obtained for this case study structure for comparison with the seismic response of the conventional and the base-isolated designs. It is noted, however, that neither the SEAOC or the linear elastic base shear was used in the as-built design of this structure; rather the as-built design was performed for a base shear of 0.5 x W in combination with yield level stresses. This is also discussed in more detail in Chapter 7.0.

The building is a four-story hospital structure about 430 feet square in plan. The lateral force resisting system consists of shear walls in combination with a ductile moment-resisting space frame. The floor plan and typical elevations are shown in Figures 22 and 23, respectively. The shear walls are nonload bearing and are designed to carry 100 percent of the lateral load. The space frame serves to carry the vertical load and is designed for a horizontal load equal to 5 percent of the weight of the structure. The floor slabs use lightweight concrete, and the building is supported on drilled and cast-in-place reinforced concrete piers. Only the four-story main structure is analyzed herein for the seismic response.

6.5.2 Base-Isolation Design

A conceptual method of incorporating the base-isolation bearings is illustrated in Figure 24. The bearings are mounted on pier caps. The bearings, in turn, support bearing blocks interconnected by a system of beams which support a structural slab at the first floor.

A typical bearing for the base-isolation design of the above building is shown in Figure 25. The bearing is 30 inches square in plan and consists of seven 3/4-inch layers of 50-durometer elastomer reinforced with six layers of 1/4-inch steel plates. The top and bottom bearing plates are 1/2-inch thick; the top plate could be either fixed or form a slip interface. The horizontal stiffness of each bearing is about 25 kips/inch. Two hundred and seventy-five (275) such bearings are used to support the vertical load which results in a fundamental frequency of about 0.7 Hz.

6.5.3 Mathematical Model

The mathematical model used in the response calculations for the north-south direction is shown in Figure 26. The building masses are concentrated at floor locations and interconnected by the respective story stiffnesses which consist predominantly of the shear stiffness of the shear walls. Again, the base isolation is represented by horizontal bearing stiffness coupled with a frictional interface and a damper so that all the analysis cases can be accommodated by the representation.

6.5.4 Seismic Response

The Code (SEAOC, 1974) minimum lateral loads for Zone 4 were computed on the basis of a K-factor of 0.8 and an importance factor of 1.5. The resulting total base shear is 18,645 kips and is distributed over the height in accordance with the Code procedures.

The story shears and overturning moments for the various cases analyzed are shown in Figure 27. It is seen that for the base-isolated cases, the resulting shears and overturning moments for a 0.6g input are in the range of the Code minimum forces for Zone 4. Among the base-isolation strategies, the bearings with the frictional interface effect the largest reduction of seismic forces.

Figure 28 shows the displacements and accelerations over the height of the structure. It is seen that for the base-isolated cases, both the accelerations and displacements are more or less uniform over the height of the structure, again suggesting an almost pure rigid-body motion of the superstructure. Correspondingly, a reduction in the interstory drift is effected by base isolation. The above response quantities are summarized in the following table:

TABLE 6-3

RESPONSE QUANTITIES
CASE STUDY III

DESIGN	BASE SHEAR (kips)	OVERTURNING MOMENT (kip-ft)	MAXIMUM ACCELERATION (g's)	INTERSTORY DRIFT (in)
Conventional				
SEAOC (Zone 4)	18,645	959,900	-	-
Linear Elastic[1,2]	119,322	5,780,000	1.65	0.0791
Base Isolated[2]				
5% Damping	33,019	1,381,000	0.31	0.0218
5% Damp + Friction[3]	25,059	1,061,000	0.25	0.0142
15% Damping	28,862	1,206,000	0.27	0.0186

[1] Provided as an approximate measure of linear elastic displacements.
[2] Based on El Centro 1940 S00E record, 0.6g peak.
[3] Maximum slip = 3.7 inches.

In the case of the bearings with a frictional interface, the relative displacement and total slip time histories are shown in Figure 29. The maximum total slip is seen to build up to 3.7 inches with a slip at the end of 10 seconds of about 1.5 inches.

6.6 DISCUSSIONS

The seismic response quantities for the three case study structures were computed using the El Centro 1940, S00E record, scaled to 0.6g peak ground accelerations. Although it is not possible to compare the base-isolated response with either the Code minimum or the linear elastic response on a one-on-one basis, some qualitative observations may be made with reference to results presented above.

Since inelastic behavior would occur in real structures, the linear elastic response of the conventional structure is unrealistic and provides only an approximate measure of the maximum deformations that can be reached within the structure for a 0.6g event. Most areas designated as Zone 4 are unlikely to experience 0.6g peak ground accelerations; a 0.35g to 0.4g is perhaps more realistic. Nevertheless, comparison with the Code minimum design loads for Zone 4 suggests the degree of motion that has to be accommodated by inelastic action and hence the damage potential.

In practice, structures which are likely to experience higher earthquake intensities will generally be designed using a more rational approach for forces that are well in excess of the Code minimum values. Case Study III is an example, and its as-built design will be reviewed in some detail in Chapter 7.0.

Comparison of responses of the conventional and base-isolated buildings illustrates a dramatic reduction in the level of forces under the consistent assumption of linear elastic behavior of the structure. For a 0.6g earthquake, these forces are reduced by factors in the range of 2 to 5 to about the same order of magnitude as the Code minimum forces for Zone 4. This suggests that a structure designed for this force level is more likely to experience inelastic distortion if conventionally founded than if base isolated. Permanent distortions in the latter case may be limited to the base-isolation bearings. Indeed, this is to be expected because base isolation forces a one-mode dominant response with litte participation of the structure modes.

The three case study structures analyzed vary from relatively rigid (box structure) to relatively flexible (DMRSF). It appears that for stiffer structures, the base-isolation bearing with a frictional interface effects somewhat larger reductions in seismic forces, while for a flexible structure a high damping bearing appears to be more successful. Also, the difference in the base-isolated forces for the three strategies decreases as the structure gets more flexible and the structural modes contribute more to the total response. It is noted that this conclusion is predicated on a fundamental base-isolated frequency of 0.75 Hz. Indeed, this frequency can be further reduced by an appropriate bearing design in an attempt to decrease the structural mode participation; however, lower frequencies result in higher seismic displacements and may present P-Δ type stability problems within the bearings.

The base-isolation bearings were represented in the mathematical models by a lumped horizontal and vertical stiffness. In the actual case, the bearings are dispersed over the foundation area, and the overturning moment, although small, is likely to result in variations in the vertical load on the bearings. In the case of bearings with frictional interface, the above phenomenon may add to the nonlinearity of the response and the potential for some bearings to slip more than others; in the case of fixed bearings, the phenomenon may lead to potential tension in the reinforced elastomer bearing.

To illustrate the effect of overturning moment on the base-isolated response, a more detailed analysis of the Case Study I structure was performed. In this analysis, the structure was mathematically represented as before (Figure 26). However, the base isolation was represented by three sets of nonlinear stiffness and damping elements, one at the center and one at each exterior wall location. The first floor mass is equally distributed at these three locations and connected by horizontal rigid elements of such stiffness characteristics that little flexure of the rigid links develop as the structure responds to seismic excitation. This mathematical representation is shown in Figure 30.

The objective of the above analysis was to compare a time-history response obtained from the two different mathematical representations of the base-isolated structure and to determine the effect of overturning, firstly, on the overall seismic response and, secondly, on the relative displacements of the bearings.

Figure 31 shows the variation of acceleration over the height of the building for the two mathematical representations. The difference in floor accelerations is indeed quite small, and further, the accelerations are fairly uniform over this height in both cases. This suggests that the rocking mode does not participate significantly and, further, that the overturning moment has little effect on the overall response of the structure. In other words, the inclusion of overturning in the analysis does not increase the participation of the first structural mode. The relative displacements are, for all practical purposes, the same in both models. Figure 32 shows a comparison of the displacement and slip time histories. The total relative displacements at all three bearings are necessarily equal because the three bearings are connected by a rigid diaphragm at the first floor. The total slip for the three-bearing cases is presented in this figure as the average of the individual slips at each of the three bearings. This comparison shows that the difference in response is rather small. The resulting maximum average slip is somewhat larger in the three-bearing model.

Of particular importance in the design of base isolation is the response of individual bearings. This is illustrated in Figure 33, which shows the slip time histories for the left, center, and in the right bearings in comparison with the slip time history of the one-bearing model. This figure shows that the onset of slipping occurs at different times, and the differences in the slip are small and are on the order of 1/2-inch.

The above example illustrates that for the particular structure analyzed the base-isolation response may be evaluated using a rather simple model of the structure and bearing system. Rocking modes and overturning moments do not appreciably affect the overall seismic response of the base-isolated structure. Simple models, such as used above, are therefore quite adequate for box-type structures, at least in the preliminary design stage, and provide reliable estimates of gross lateral forces and displacements in the base-isolated structure. Similar conclusions have been drawn for massive shear-wall structures such as those of a Nuclear

Power Plant (D'Appolonia, 1980). However, it is noted that depending upon the flexibility and weight of the structure, there may be cases where bearing uplift is experienced resulting in possible significant differences in seismic response.

Since the base isolation decreases the participation of the structural modes, it consequently reduces the potential for inelastic action in the structural elements. A large part of the earthquake energy imparted to the structure is accommodated by and dissipated in the distortion of the elastomer bearing. Consequently, the predominant failure mode of the base-isolated structure can be anticipated to be in the bearing. Therefore, further reduction of the structural forces, vis-a-vis the conventional design, may not be appropriate. This directly affects both the design of the structure as well as the base isolation system. This is evaluated in detail in Chapter 7.0.

6.7 DESIGN CONSIDERATIONS

Because the design concept of base isolation is fundamentally different from the conventional methods of seismic design, certain parameters are of less importance in the design of a base isolated structure when compared with a conventional structure. Alternately, other parameters may require more careful consideration. Some of the major factors which need to be considered in the design of structures founded on aseismic bearing pads are discussed below.

Soil Conditions

The seismic response of massive structures is usually influenced to some degree by the type of soil on which they are founded. In most conventional building structures, however, soil-structure interaction plays a less significant part in influencing the seismic response. In any event, generally there is some uncertainity in the computed response due to the inherent uncertainty in predicting the dynamic soil parameters. Inasmuch as the horizontal response is governed by the structure's predominant mode, base isolation reduces the above uncertainty. However, in addition to the structure, the use of base isolation must be established with due consideration to the site conditions. For example, if the site consists of soft soils such that the ground motion is dominated by low-frequency components, then base isolation could cause an unduly high displacement response by amplifying the low frequency motion and perhaps should not be considered.

Vertical Ground Motions

The stiffness of the aseismic bearing pads in the vertical direction is typically 500 to 1,000 times the stiffness in the horizontal direction. The vertical stiffness of the pads approaches that of the concrete bearing blocks which support them and hence the vertical response of the overlying structure is practically unchanged by the presence of the pads. The pads therefore offer no isolation against vertical ground motion.

The response of structures to vertical ground motion can lead to a nonuniform distribution of vertical load on the pads during an earthquake, producing a subsequent variation in the frictional resistance. This has the potential of affecting the horizontal displacements, especially the amount of slip. However, the potential 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. The vertical ground motions therefore have a minor effect on horizontal response.

Differential Settlements

Possible variations in settlement of the lower foundation can result in a redistribution of the vertical loads on the pads and a rotation of the bearing plane for each pad. Analyses have shown that the two phenomena do not appreciably affect the overall seismic response of the structure (Plichon, et al., 1980). Anticipated differential settlements and the consequential rotation for the bearing plane should be included in the design evaluation of the individual pads as they have a potential to cause additional stresses in the bearing pad when it undergoes shear distortion.

Relative Horizontal Displacements

Relative horizontal displacements of the base isolated structure has to be accommodated in the design by providing an appropriate seismic gap between it and the non-base isolated components. The design of utilities which traverse the boundaries of the base-isolated structure should include appropriate features to accommodate the differential movements.

Rotations about the Vertical Axis

Relative twisting of the foundations and the superstructure during a seismic event can arise due to dynamic eccentricity, differences in the centers of gravity of structures and the aseismic bearing pads, spatial variation in the physical properties of the pads, and the passage of Love waves. An analysis of these factors has shown (D'Appolonia, 1980) that the contribution to horizontal response from the relative twisting is about one to two orders of magnitude smaller than from the translational motion alone.

Bearing Pad Inspection and Replacement

The foundation system should permit regular inspection and, if required, instrumentation of the aseismic bearing pads. This provision is not included as a requirement but is believed to be prudent at least until such time as the professional community develops sufficient confidence in the performance of the bearings, such as to render documentation of in-service condition of the bearing unnecessary. The materials used in the pad construction have demonstrated their capability in other applications and the need for replacement of the bearings during the service life of the facility is therefore not anticipated. In any event, should such a need arise, the base isolation design could incorporate the provision for pad replacement.

Permanent Displacements

After a severe seismic event, permanent horizontal displacements are likely to occur between the foundation and the superstructure. These will be a result of the elastomer distortion in the fixed bearing and slip on the friction surface in the case of sliding bearings. The permanent displacements in the former will in general be smaller than in the latter. After a seismic event, therefore, inspection of the bearings should be performed as a minimum. Severely distorted bearings would have to be replaced and the structure may have to be repositioned to its original location.

In the case of the bearings with a frictional slip surface, the permanent distortion in the elastomer is limited if not eliminated at the expense of a permenant slip. The potential for bearing replacement is therefore eliminated. However, a significant permanent slip (on the order of 4 or 5 inches) has to be compensated in the repositioning operations. The procedure for repositioning consists of applying a horizontal force at the top of individual bearings sufficient to produce slip of that bearing with respect to the superstructure. When this horizontal force is removed, the superstructure is subjected to an unbalanced load which results in a horizontal displacement of the superstructure of such a magnitude as to satisfy equilibrium conditions. This procedure is repeated for all pads in a sequence such that at the end of the sequence the superstructure is in its original location with respect to the foundation.

7.0 STRUCTURAL DESIGN AND COST IMPACT

As part of the overall effort, the feasibility of base isolation as an aseismic design strategy was assessed by evaluating some of the design considerations in applying it to the Case Study III structure. This structure was chosen for the purpose, after a qualitative assessment of the structure types, seismicity, and importance. In general, base isolation appears to be attractive for facilities with the following characteristics:

- Building is located in areas of significant seismic potential.
- Building is relatively stiff (fundamental frequencies greater than about 3 Hz. A shear wall structure in the range of about 5 stories falls in this category.
- Building houses critical equipment.

The Veteran's Administration (VA) Hospital in Loma Linda, California has been designed for a 0.6g peak ground acceleration and its seismic criteria requires that the facility "should be operational during and following a major earthquake."

The specific objectives for the case study were the following:

- Evaluate the technical feasibility of base isolation.
- Assess some of the practical problems associated with incorporating base isolation.
- Establish the benefits of using base isolation.
- Evaluate the cost impact on the structural elements.

It is noted that base isolation, if considered in the early design stages, may affect the choice of the lateral force resisting system, configuration of the structural elements which resist the seismic forces, interface and layout of mechanical systems and, potentially, the architectural scheme of the building itself.

Although a few modifications in the building configuration are inevitable for incorporation of the base isolation in the building foundation, the general philosophy adopted for this case study was to retain as much as possible the as-built configuration and lateral force resisting element and investigate the effects on only the seismic design of the structural elements if the building were base isolated.

7.1 SCOPE OF CASE STUDY

The case study commenced with reviewing the structural data, namely the structural design drawings, the seismic criteria, and the structural design calculations. The structural drawings were obtained from the Veteran's Administration. The seismic criteria and the design calculations were provided for review by the structural engineers for the Loma Linda hospital. The following specific tasks were performed toward meeting the objectives outlined above:

- Determine the desired characteristic of the base isolation design for the structure.
- Develop mathematical models of the conventional and base isolated structure.
- Develop ground motion time histories to represent the expected motions defined by the ground design response spectra for the Loma Linda site.
- Perform linear and nonlinear dynamic analysis using the ground motion time histories.
- Determine displacements, accelerations, forces and moments, and floor response spectra for the above models.
- Develop the design of a base-isolation scheme.
- Redesign representative structural elements for the seismic forces for the base-isolated case.

- Develop a cost comparison for the structural elements associated with the conventional and the base-isolated strategies of aseismic design.

The above tasks and the important results obtained are discussed in subsequent sections following a brief background on as-built conditions.

7.2 BACKGROUND

7.2.1 Structural Configuration

The Loma Linda VA Hospital is a four-story structure consisting of a complete vertical load carrying moment-resisting space frame and nonload-bearing shear walls. The floor plan and typical building sections are shown in Figures 22 and 23, respectively. The main structure is approximately 430 feet square in plan and is essentially symmetrical. The shear walls are arranged to minimize diaphragm dependence for shear distribution.

The shear walls are designed to carry 100 percent of the lateral seismic load and the moment-resisting space frame is designed to carry about 5 percent of the weight of the structure. Thus, the space frame serves as a backup lateral load-resisting system which will be mobilized in the tail end regions of a seismic event after the lateral load carrying capacity of the shear walls has been exhausted.

The as-built foundations consist of drilled piers 36 to 42 inches in diameter. Typically, a single drilled pier is located under each column of the space frame. The shear wall edge columns are founded on groups of three piers to provide added vertical capacity to resist effects of overturning moment.

7.2.2 Site Conditions

The Loma Linda VA Hospital site consists of deep unconsolidated alluvium. Measured shear wave velocity in the upper 135 feet is reported to be about 1,800 feet per second (fps). The shear wave velocity increases to about 8,000 fps in the rock-like material at depths of about 2,500 feet. The ground water table is located at about 152 feet and, hence, liquefaction potential was precluded. The profile of maximum shear modulus used in the site-specific analysis is shown in Figure 34. Published data on shear modulus in sands was also used for parametric variation in determining the surface ground motion.

7.2.3 Seismic Design Criteria

The Loma Linda VA Hospital is located in San Bernardino County in southern California. Its location, with respect to major faults is shown in Figure 35. The San Andreas and the San Jacinto faults affect the site seismic potential the most significantly, although there are approximately 10 fault zones within 65 miles of the site.

On the basis of the VA criterion (Veteran's Administration, 1973) that the hospital should be "operational after a major earthquake," the as-built design is based on earthquakes only slightly smaller than the maximum credible events.

The design seismic motions originating from the San Andreas Fault (SA) and the San Jacinto (SJ) are described in Table 7-1. The estimated peak surface accelerations for the San Andreas and the San Jacinto events are 0.59 and 0.56, respectively. The smoothed ground response spectra are shown in Figure 36 and represent the upper average of the computed ground surface spectra.

TABLE 7-1

DESIGN SEISMIC MOTION
LOMA LINDA VETERANS ADMINISTRATION HOSPITAL

SOURCE	DESIGNATION	MAGNITUDE	DISTANCE FROM SITE	MAXIMUM ROCK ACCELERATION (g's)	PEAK SURFACE ACCELERATION (g's)
San Jacinto	SJ-1	7 to 7.5	1.25 mi	0.65	0.56
San Andreas	SA-1	8+	7 mi	0.53	0.59

From considerations of such variable parameters as effective peak, duration, and out-of-phase input motions due to large building size and ductility, the as-built seismic design has been based on a base shear of 0.5 times the building weight.

Base Isolation Characteristics

In selecting the criteria for the base-isolation design, it is noted that the general philosophy of this design is to impart a low frequency to the predominant horizontal mode of the base-isolated structure. This frequency should be outside the range of frequencies commonly encountered in earthquake ground motions and yet should not be so low as to result in an unacceptable displacement response. Further, the base-isolation bearing should be sufficiently rigid in the vertical direction to be able to support the weight of the structure.

Since the predominant frequencies of the seismic ground motions are generally in the range of about 3 to 7 Hz, a predominant mode frequency of about 0.7 Hz was chosen at the outset for the design of the base-isolation scheme. The total weight of the structure, including the first floor, being about 132,000 kips, the required total horizontal stiffness of the base-isolation scheme is thus about 84,000 kips/ft. From practical considerations, the total horizontal stiffness would result from several reinforced elastomer bearings distributed under and providing support to the structure, with each contributing to the horizontal stiffness in accordance with its physical dimensions, namely, the bearing area and total elastomer thickness and the shear modulus of the elastomer used.

The low frequency of the predominant mode is indeed expected to result in a significant displacement response. Some means of energy absorption may therefore be necessary to limit the structure displacements, especially when severe earthquake motion is expected.

Starting with the basic frequency criterion for the base isolation, the following three design strategies were examined for their effect on the resulting seismic response of the base-isolated structure:

- A 5 percent damped bearing,
- A 15 percent damped bearing, and
- A 5 percent damped bearing with a friction interface.

In general, the choice of the appropriate base-isolation strategy depends on the type of structure, its importance, the site seismic criteria, and the structural response. For the present case study, the base-isolation design and details of its incorporation in the foundation are discussed following the evaluation of the seismic response for the above three base isolation strategies.

7.3 SEISMIC ANALYSIS

The seismic response of the structure under consideration was computed using time-history methods of dynamic analysis and representative ground motion time histories. In addition to the three base-isolation design strategies, the linear dynamic response of the conventionally founded structure was also evaluated. The latter response is provided to facilitate comparison and to gain an insight into the uncertainties associated with the evaluation of the seismic response for and the effective mitigation of seismic risks afforded by the various aseismic design strategies.

Subsequent sections describe the mathematical models, time histories of input motion, and the resulting dynamic response.

7.3.1 Mathematical Model

The mathematical model was described in the preceeding chapter as the representation of the Case Study III structure and is illustrated in Figure 26. It assumes that the floors behave as rigid diaphragms and distribute the horizontal seismic shears to the shear walls in proportion to the shear wall stiffnesses. The building mass was lumped at floor locations, and these masses were interconnected by beam elements representing the interstory stiffness.

In computing the interstory stiffness, only the shear deflection of the shear walls was considered. The flexural deflection of the shear walls was ignored because usually this is small in comparison with the shear deflection. Also, the stiffness of the moment-resisting space frames was ignored as this is small in comparison with the shear stiffness of the walls. This model is justified on the basis that the dynamic response of the base-isolated structure is not very sensitive to the degree of complexity of the structural model. This is so because the predominant mode of the base-isolated structure is a rigid body translation of the superstructure on the bearing pads with a small participation of the first mode of the superstructure. Higher modes of the superstructure participate only minimally in the overall response.

The mathematical representation of the base-isolation system includes the vertical and horizontal stiffnesses of the bearings. In the horizontal direction, the stiffness element of the reinforced elastomer is coupled with an element representing a slip surface which limits the horizontal force on the bearing to the frictional resistance of the slip surface. In both the horizontal and vertical directions, the material damping associated with the elastomer is represented by appropriate viscous dampers. The distribution of the bearing pads under the building results in effective rotational stiffness about the horizontal and vertical axes. These are represented as rocking and torsional springs in the mathematical model of the base-isolation system.

The parameters of the general model described above, such as the spring stiffness, damping, and friction coefficient, can be appropriately chosen to represent the three base-isolation strategies examined.

7.3.2 Seismic Input

Time histories of ground motion compatible with the ground response spectra, shown in Figure 36, were used in the dynamic analysis. The time histories were obtained by adjusting the frequency content of real earthquake records until the response spectra of the resulting time histories matched the smoothed ground response spectra. The synthetic time histories matching the SJ-I spectrum are shown in Figure 37, and it's response spectrum in comparison with the smoothed ground response spectra for the SJ-I event is shown in Figure 38. In accordance with the seismic criteria used in the as-built design, a peak ground acceleration of 0.6g was used in the seismic analysis.

7.3.3 Results

The seismic response for the various aseismic design strategies was evaluated first for the San Jacinto input on the basis of which an appropriate base-isolation design was established. This design was subsequently verified for the San Andreas event.

The seismic response was obtained in terms of story shears and overturning moments, floor accelerations and relative displacements, and in-structure floor response spectra. The computations for the base-isolated as well as the conventional building are based on a linear elastic behavior of the structure but include nonlinear effects in the base-isolation due to potential slip on the friction surface. Since some inelastic structural behavior in the conventional building is expected, the corresponding accelerations, story shears and moments, and floor response spectra are unrealistic and are provided only as an approximate measure of the possible displacements and a trend of behavior which may be realized in smaller magnitude events.

Figures 39 and 40 show the distribution of story shears and moments, respectively. In addition to the cases analyzed, these figures also present the quantities used in the as-built design. Although not shown in the above figures, it is noted that the shears and moments in the conventional case for linear elastic behavior are about three to four times those used in the as-built design. Figure 41 presents the floor accelerations, and Figure 42 shows the in-structure floor response spectra.

As expected, the above results indicate that all three base-isolation strategies impose a predominantly rigid body response of the structure. This is evident from the variation in the response quantities over the height of the structure. All the base-isolation strategies effect a significant reduction in the forces. The friction bearing and the 15 percent damped bearing result in comparable shears and moments on the structure, the friction bearing showing a slightly larger reduction. This is indicative of the effective damping introduced into the system due to energy loss in a hysteresis loop as the structure slips on the friction surface. With a friction coefficient of 0.2, the equivalent viscous damping is estimated to be about 20 percent.

The floor response spectra represent forces on the equipment and other nonstructural components within the building. A comparison of the spectra shows that base isolation dramatically reduces the peak seismic motions and forces on the building contents. Also, the peaks in the base-isolated floor response spectra occur in the lower frequency range away from the predominant natural frequencies of the equipment and other nonstructural components. These two effects are expected to result in a substantial reduction in the potential for nonstructural damage.

The time history of relative displacement between the ground and first floor is shown in Figure 43. The total relative displacement consists of the elastic distortion of the bearing and the slip on the friction surface. For a large value of friction coefficient, such as would prevent slip, all of the total relative displacement consists of the elastic distortion of the bearing. As seen from the above figure, the maximum total relative displacement for the San Jacinto input is about 9 inches. With the slip surface having a friction coefficient of 0.2, the elastic distortion is about 4 inches, and the remaining 5 inches is the maximum slip.

Additional results for the San Andreas input are described following the discussion on the base-isolation design.

7.4 BASE-ISOLATION DESIGN

As outlined above, the seismic response of the base-isolated structure was computed for the following cases:

- 5 percent damped bearing,
- 15 percent damped bearing, and
- 5 percent damped bearing with a slip surface.

As seen from Figures 39 through 42, all three strategies result in comparable forces on the structural elements. For the structure considered and the intensity of seismic motion used, the base-isolation design using reinforced elastomer bearings, coupled with a frictional interface ($\mu = 0.2$), results in the least structural forces.

An important response quantity from the point of view of bearing design is the elastic distortion of the bearing pad. Large shear distortions of the bearing are likely to affect their vertical load carrying capacity and, therefore, it is important to limit the pad distortion. In bridge bearings, the pad distortions are generally limited to about half the elastomer thickness, although the European practice allows this distortion to be about equal to the total elastomer thickness (Stanton and Roeder, 1982). Tests on bridge bearing pads have shown that repeated shear distortions, some of which exceed 1 to 1.25 times the pad thickness, have not resulted in pad deterioration (Imbsen and Schamber, 1981). Although seismic loading conditions are different than the movements normally experienced by bridge bearings, a good design practice for seismic conditions appears to be to limit the shear distortion in the range of 1 to 1.25 times the elastomer thickness.

In allowing slip, the elastic distortion is limited to about four inches, which can be easily accommodated in the above distortion criterion. The maximum slip, which is on the order of five inches, can be accommodated by providing a slip interface which is designed to maintain contact between the slip plates with an adequate safety margin to provide for larger values of slip than those predicted. This provision lends an added reliability to the base-isolation system in being able to accommodate larger events.

Although an appropriate base-isolation design can be developed for each of the strategies considered, the bearing with the frictional slip surface is chosen here as an example of practical considerations. This type of base isolation was adapted for further investigations related to this case study. Similar impact on structural design is seen to result from other bearing designs.

7.4.1 Bearings Design

As discussed above, the design criteria for the bearings include the horizontal and vertical stiffness, allowable compressive and shear stresses, allowable vertical and horizontal bearing distortions, degree of slip to be accommodated, and practical considerations of manufacture and installation.

After investigating various bearing sizes, a bearing 24 inches square in plan was chosen. Its thickness comprises five 3/4-inch layers of 50-durometer elastomer reinforced, with four 3/8-inch steel shims. The bottom bearing plate is made of steel and is fixed to the foundation. The top bearing plate is about 1/2-inch thick and is made of a lead-bronze alloy which forms a friction couple with a stainless steel plate embedded in the first floor beam. This friction couple provides a fairly constant coefficient of friction over a wide range of vertical pressure and relative velocities of the two surfaces (Jolivet, 1979).

The stainless steel plate is 40 inches square in plan and provides an overhang of 8 inches on all sides of the bearing. The bearing is thus capable of accommodating a slip on the order of about 12 inches which will uncover the bearing surface by 4 inches or about 15 percent.

The horizontal stiffness of each bearing is about 23 kips/inch; thus requiring 275 bearings under the entire main structure to provide the required horizontal stiffness. The bearings are distributed one under each typical building column and two under the edges of each shear wall. This distribution allows the required horizontal stiffness to be realized and simultaneously limits the maximum vertical stress to about 1,500 psi, well below typical failure loads for such bearings.

7.4.2 Bearing Installation

The conceptual design of the bearing installation was developed within the constraints of the as-built structure and foundation design. This concept is illustrated in Figures 44 to 47. As shown in Figure 47, the bearing is grouted in-place on the pier cap after leveling. The stainless steel plate embedded in a concrete block to facilitate handling is centered on the bearing and held in place until the first floor beams, bearing blocks, and the floor are cast. The rest of the superstructure is constructed as in the conventional case.

The conceptual design shown in the above figures provides about a nine-foot-high basement over the entire area of the structure. The basement allows periodic inspection of the bearings and their replacement, if replacement should become necessary. Installation of a mud mat or a slab on grade at the basement floor level will permit other uses for this space. Elevation differences in areas of the basement floor can be accommodated in the scheme as illustrated in Figure 46. A perimeter retaining wall forms the outside boundary of the basement. A space of about 18 inches is provided between the retaining wall and the components of the base-isolated structure in order to accommodate the displacement of the structure during a seismic event.

The construction sequence is anticipated to be as follows:

- Excavation
- Install foundation piers
- Cast retaining wall
- Cast pier caps
- Install grade beams and mud mat
- Install bearing pads and slip plate
- Cast bearing blocks and first floor beams
- Place structural slab on first floor
- Continue with the superstructure.

7.5 REDESIGN OF STRUCTURAL ELEMENTS

The as-built lateral force-resisting system of shear walls was adopted also for the base-isolated cases, the present aim being only to reduce the strength requirements, such as thickness, reinforcing, shear transfer to beams and columns, collector elements, etc., for the reduced seismic shears due to base isolation. It is noted that relocating shear walls, perhaps even considering other load-resisting systems more suited to the reduced level of seismic forces, may be worth considering in a real situation.

Some of the reasons given to reduce the seismic forces to design values (SEAOC, 1974) such as peak versus effective peak and wave propogation effects due to foundation size are equally applicable to base-isolated structures. However, in the present case study, no reduction was allowed for these reasons, and the design response quantities for the base-isolated structure were taken to be those resulting from a time-history dynamic analysis with peak ground acceleration of 0.6g. This is seen as reducing some of the uncertainties that are inherently present in adopting the code response quantities for the design of conventionally founded structures.

The structural elements of base-isolated structures were designed for the SJ-I ground motion and verified for the SA-I ground motion. Allowable material stresses were the same as those used in the as-built design. The shear walls were designed to carry 100 percent of the lateral forces. Because of the following reasons, it is felt that the secondary or back-up lateral load-resisting system, namely the lateral resistance of moment-resisting space frame, may not be required:

- There is less uncertainty in predicting the response of the base-isolated structure.
- No reduction in the free-field response spectrum is taken.
- The structural elements of the primary load-resisting system are designed to remain below their ultimate load capacities.

As a result, the moment capacity of the beam column connections may be significantly reduced; perhaps standard connections may be adequate.

Of the primary lateral load-resisting system the structural components directly affected include the roof and floor diaphragms and their shear reinforcements, collector elements and chord reinforcement, shear walls and transfer of shear to beams and columns, shear wall edge columns, column base tension blocks, and foundation piers under shear walls.

The as-built roof and floor diaphragms are cast of lightweight concrete. Due to the reduced forces in the base-isolated structures, it was possible to eliminate the additional shear and chord reinforcement in the diaphragms, which was necessary in the as-built case at some locations. Similarly, the load transfer from the diaphragms to collector beams required less additional load transfer reinforcement.

No attempt was made to reduce typical floor beam sizes since the design of these beams is in most cases controlled by vertical loads. However, special collector beams which transmit the diaphragm load to the dispersed shear walls and therefore use heavier sections than required by vertical loads alone, were reevaluated. As expected, the most significant difference in the collector beam requirement was noted at the roof level. This difference decreased towards the lower floors. At the second floor, the difference is eliminated, as the design of the collector beams begins to be governed by vertical load and there is adequate margin past the allowable beam stresses to accommodate the seismic axial loads.

Shear walls were redesigned for the base-isolated forces using the same allowable gross shear stress and the pier shear stress as in the as-built design. Typically, the fourth floor shear walls could be reduced in this case from 14 inches to 10 inches and from 12 inches to 8 inches. Although force requirement alone could permit further reduction in thickness of some shear walls, 8 inches was taken as the minimum for constructibility and to avoid potential shear instability problems. At the first floor elevation, typically, the 24-inch-thick shear walls could be reduced to 15 inches and 18 inches to 12 inches. Shear reinforcement was provided in the as-built case at some locations in addition to the nominal wall reinforcement. Although this could be eliminated in the base-isolated design, no account was taken in the quantity calculation since it is believed to be small. Moderately significant reduction is observed in the requirement for shear friction reinforcement in transmitting horizontal shears from the shear walls to the floor beams and the vertical shear to the shear wall edge columns.

As the overturning moments are reduced, the vertical forces on the shear wall edge columns are correspondingly reduced. This is reflected most significantly in the first floor where a savings of about 30 tons in the structural steel columns was realized. Similarly, most of the tension block requirements could be eliminated because no net uplift results in the base-isolated case.

The as-built foundation pier design at the shear wall edges was controlled by vertical loads due to seismic overturning moments. In the base-isolated case, both the dead load and the vertical load due to overturning moment are reduced. As a result, significant reduction in the required pier capacity is realized. For example, at column location E-25, the governing load combination is dead load plus earthquake load and the vertical design load from this combination is about 3,800 kips. In the as-built cases, three 42-inch-diameter piers, 84 feet long, provide the required capacity. For the base-isolated case, the same load combination results in a vertical design load of about 1,300 kips. Required pier capacity at the shear wall edges could thus be reduced by two thirds. In the base-isolated design, the required capacity could be provided by one 42-inch-diameter pier of approximately the same length. Correspondingly, the pier cap size at these locations could also be reduced.

Figures 48 through 51 present a comparison of seismic response of the base-isolated structure for the SA-I and the SJ-I input. These responses have been obtained for the base-isolation design strategy adopted for design, namely, that using bearings with a friction slip surface. As seen from this figure, the forces and displacements for the SA-I input are only slightly greater than for the SJ-I input. It is therefore concluded that the design performed on the basis of SJ-I input would be adequate for the SA-I input, also.

7.6 COST EVALUATION

In addition to decreasing the probable repair or replacement cost of the facility in the event of the design earthquake, the above considerations suggest that some reduction in capital cost may also be expected if base isolation is used as an aseismic design strategy. However, the installation of base isolation requires additional construction items which would not be required in the conventional case. In order to assess the cost differential, the probable deduct and added costs associated with the base isolation were evaluated for the case study structure.

The intent of the cost evaluation was to provide an estimate of the relative cost of incorporating base isolation on an aseismic strategy in place of the conventional design. The cost evaluation has been performed under the constraints of the as-built conditions and it is restricted to only those major structural items directly affected by the base-isolation system. Several details of the as-built construction which may be affected by base isolation and therefore have some cost impact are not included. Certain change in geometry, especially at the foundation and basement level, was inevitable. In the following discussions, the items ascribed to the latter are noted. They appear, in appropriate quantities, in both the probable deduct and the added costs; e.g., excavation. Unit costs used in the computation are based on 1982 construction costs.

Figure 52 is a plan of the as-built structure showing the layout of the mechanical/electrical (M/E) rooms and the utility tunnel located below the first floor. This figure is included here to facilitate subsequent discussions of probable deduct and added costs.

In evaluating the cost differentials, it was assumed that both the main building and the adjacent one-story central plant are base isolated and are tied together horizontally to limit the potential differential horizontal motion between these two structures. Indeed, utility, piping, and mechanical connections can be designed to accommodate the maximum differential horizontal movements expected during the design seismic event and, hence, it is conceivable that only the main structure need be base isolated. The effect on costs using base isolation only for the main structure is subsequently examined.

7.6.1 Probable Deduct Costs

The probable deduct costs represent the reduction in strength requirements and other structural modifications that can be made when base isolation is incorporated. Prominent items contributing to the deduct costs are included in Table 7-2.

TABLE 7-2

PROBABLE DEDUCT COSTS

ITEM	COST ($)
Diaphragms	131,000
Shear Walls	1,312,660
Basement Walls	207,000
Foundation	1,505,000
Structural Steel	243,600
Tunnels and M/E Walls	437,500
Engineered Backfill	9,500
Slab-on-Grade	907,500
Excavation for M/E Rooms and Tunnel	151,500
Equipment and Utility Hold Downs	-
TOTAL	4,905,260

The savings associated with the diaphragm include predominantly the shear reinforcement at connections between the diaphragms and certain shear walls, and collector beams. Approximately 38 tons of shear reinforcement are eliminated and the collector beams are reduced by about 62 tons.

The deduct in shear walls above the first floor results from decrease in the required wall thickness and the horizontal and vertical shear friction reinforcement. The shear walls below the first floor are eliminated altogether. This item is attributed to the inevitable change in the geometry of the basement. Similarly, the basement walls and the walls associated with a utility tunnel and the mechanical/electrical rooms were eliminated. These items would be replaced as required, consistent with the geometry of the base-isolated basement. For example, the M/E basement walls would in part be replaced by a continuous perimeter retaining wall in the base-isolated geometry.

The savings in the foundations include elimination of certain drilled piers installed under the edges of the shear walls and the associated decrease in pier cap volume.

The deduct in structural steel includes reduction in shear wall edge columns and elimination of structural beams at the first floor and the shear wall edge columns below the first floor. Totally, about 162 tons of structural steel are thus eliminated. Although further reduction could be taken due to the elimination of the lateral resisting capacity of the space frame, this time is not included here.

Again, due to change in the geometry, the grade and tie beams in the as-built design are taken as deduct items to be included in their proper quantities in the added item.

The engineered backfill includes the compacted backfill in and around a utility tunnel and the basement M/E rooms. The excavation for these items is also eliminated as this item is included in the proper quantity as an added item.

The as-built design provides a six-inch-thick slab-on-grade over the entire area. After deduction for the grade and tie beams, the floor area occupied by the slab-on-grade is about 190,000 ft^2. The first floor slab-on-grade will be replaced by a structural slab to accommodate the base-isolation system in the basement. The deduct cost for the slab-on-grade includes concrete slab and the subbase preparation.

The deduct item, excavation, includes the utility tunnel and the M/E rooms in the basement.

Additional items such as anchorages for nonstructural elements, equipment and utility hold down, replication of essential equipment, and other system redundancies could be significantly reduced since the seismic forces on these elements are dramatically less in the base-isolated case with the attendant reduction in the potential for their damage. A realistic cost estimate for such items would entail the combined expertise in various engineering disciplines as well as participation of the architect and the owner. Since this was beyond the scope of the present study, such items are only listed as possible deduct items, but no cost is associated with them.

The total deduct cost amounts to about $4,905,260.

7.6.2 Probable Added Cost

The major items contributing the added cost for base isolation include the additional excavation, the perimeter retaining wall, structural floor and tie beam system at the first floor, and the base-isolation bearing pads. The added costs for the structure under consideration are listed in Table 7-3.

TABLE 7-3

PROBABLE ADDED COSTS

ITEM	COST ($)
Excavation	242,500
Retaining Wall	350,000
M/E Room Walls	81,250
Mud Mat	172,000
Bearing Pads	1,575,000
Bearing Blocks	240,000
First Floor Beams	750,000
First Floor Slab	1,044,000
Perimeter Joint	103,000
Flexible Utility Connections	-
TOTAL	4,557,750

To accommodate the M/E rooms in the basement, in addition to the base-isolation system, a 15-foot excavation is provided for in the areas of the M/E rooms. A 10-foot excavataion is provided over the balance of the foundation area.

A perimeter retaining wall is provided to enclose the entire basement area. On the sides along the M/E rooms, a 15-foot-high retaining wall is provided while a 10-foot-high retaining wall encloses the remaining area.

On the interior, the utility tunnel and the M/E rooms would be enclosed by four-hour, fire-rated steel stud walls. This item is included in the above table as M/E room walls.

The balance of the basement floor after deducting the areas occupied by the M/E rooms would be paved using a mud mat. This provision is made with the assumption that the basement is not subjected to use other than periodic inspection of the base-isolation system. It is noted, however, that the balance of the basement provides approximately 150,000 ft^2 of usable area.

The base-isolation system includes a total of 315 reinforced elastomer bearing pads with slip surfaces, 275 under the main structure and 40 under the central plant, each bearing measuring 24 inches square in plan. The cost of each bearing pad is estimated to be about $5,000 for a total of $1,575,000. A typical bearing block measures approximately 5 feet by 5 feet by 4 feet thick. A 5-foot by 7-foot by 4-foot bearing block is provided at locations at the edges of shear walls to accommodate the bearing pads at these locations.

A grid of beams is provided at the first floor. They serve to support loads on the first floor as well as to tie the bearing blocks in the horizontal plane. A typical beam size is 2 feet by 3 feet.

After deducting the area above the M/E rooms and the utility tunnel where as-built design provides a structural slab, the balance of the area, approximately 174,000 ft^2 will now require a structural slab. A one-way ribbed slab spanning 22 feet, 6 inches with an ultimate capacity of about 250 psf is used. This item is included as the first floor slab. In the central plant area, where heavier floor loads may be encountered, the disposition of the bearing pads under the first floor structural slab and beams may be established. In accordance with the load distribution, more bearing pads may be provided under heavier loads and fewer under lighter loads. On this basis, for the purpose of the cost estimation, the above structural slab and beam system is assumed to be adequate for the central plant area.

A clear gap of about 18 inches is provided between the retaining wall and the base-isolated structure. This gap would be enclosed by a flexible perimeter joint which is estimated at $50/foot and is included as an added cost item.

Flexible utility connections are anticipated to be a small cost item especially because the central plant is also base isolated. Although this item is listed, no cost is associated with it for the same reason that no deduct cost was associated with equipment and utility hold down.

The total added cost amounts to about $4,557,750.

7.6.3 Discussion

The above analysis shows that the possible savings in structural costs resulting from the lower lateral loads associated with base isolation outweighs the cost of additional construction items required for its installation. The difference in the deduct and added costs and hence the savings are on the order of about $400,000. Similar cost savings are reported elsewhere in the literature (Tarics, 1982).

As indicated above, several details of the as-built construction which may be affected by base isolation have not been included in the study. These could potentially revise the cost differential upward or downward somewhat. However, it is believed that the major structural items having been considered, the cost comparison does provide a good indication of relative costs of incorporating base isolation.

It appears likely that greater cost savings could be realized if the analysis included the impact of base isolation on nonstructural items or if other design strategies were considered. For example, the central plant may be conventionally founded or could be accommodated in the basement of the base-isolated main building, thus eliminating the need for a separate structure. Both these alternatiaves are anticipated to affect the cost differential significantly. Admitting that the latter alternative is a major change in the as-built configuration and hence contrary to the basic rules under which the cost study was performed, the following paragraphs discuss the effect on costs if only the main building were base isolated.

At the outset it was rather obvious that base isolating the one-story central plant would not result in substantial reduction of structural requirements in this structure. Eliminating base isolation for this structure from the overall scheme decreased the deduct costs by about $200,000, predominantly in the grade and tie beams and the slab on grade in the central plant area which would not now be deductible items. However, the total added costs are reduced by about $550,000. Construction items contributing to this figure are the retaining wall, basement mud mat, base-isolation bearings, bearing blocks, first floor structural slab and tie beams, and the perimeter joint. Consequently, an additional savings in structural costs of about $350,000 could be realized using this design strategy.

The above cost analysis shows that for the structure considered, a savings of about $400,000 to about $800,000 could be realized in structural costs if base isolation is used as a design strategy. Indeed, the impact on other nonstructural items needs to be studied further to obtain an estimate of cost differentials here. These and other details affected by base isolation which could not be considered in the study could revise the cost estimates upward or downward somewhat. As a minimum, however, it can be concluded that the cost of providing an aseismic design using base isolation is equal if not lower than providing an aseismic design in a conventional manner.

The incorporation of base isolation also affects construction schedule and consequently the cost. In general, major items contributing to the cost and schedule when compared to conventional construction are:

- Deeper excavations,
- First floor structural slab, and
- Aseismic bearings and bearing blocks.

Project experience from the construction of base isolated nuclear power plants measuring typically about 320 feet by 450 feet in plan suggests that by coordinating the construction of the first floor slab with the installation of the aseismic bearings bads, the schedule impact can be reduced to the time lag between the first concrete pour of the foundation and the first pour of the first floor slab. The above nuclear power plant experience suggests that this time lag is about two and one-half months. The overall construction schedule here is generally unaffected, as this time lag is compensated for in succeeding construction activities, especially the installation of equipment tie downs, utility supports, etc. Indeed, the time lag will depend to some extent upon the facility, type of construction and the construction environment.

8.0 PROBABILISTIC RESPONSE OF BASE-ISOLATED STRUCTURES

Among the sources of uncertainty entering into the computation of seismic response of structures, the predominant ones are the seismic ground motions and the mathematical representation of the structure-foundation system. These uncertainties may be addressed in a probabilistic sense where the variability in ground motion and the structural properties are considered.

In addition to the above sources, the uncertainty in the response of base-isolated structures results also from potential variability of bearing properties. Parameters affecting the variability include material properties of the elastomer, weathering effects, and manufacturing tolerances. Because of the anticipated response of base-isolated structures, the variability in the mechanial properties of bearings contribute to the response uncertainty more than the variability in the structural properties.

It is assumed herein that the variability in the bearing properties can be treated separately. This would require as a first step examination of the variances in measured elastic properties, friction parameters, and inelastic material type dampers that may comprise the base isolation scheme. It is conceivable that additional experimental work would be required to define variability in these parameters. Since this was beyond the scope of the present study, the probabilistic analysis reported herein does not address mechanical bearing properties.

The experience with the use of base-isolation bearings in nuclear power plants (D'Appolonia, 1980) suggests that the variation in the parameters such as shear stiffness and friction coefficient can be limited to within ±5 percent. Generally, with good quality control during manufacturing, handling, and installation, these limits can be expected in non-nuclear construction as well.

In the following paragraphs, the uncertainty in response due to the variability in ground motion is examined. The Case Study III structure of Chapter 6.0 is chosen as the basis and the response of this four-story structure is determined for a large number of simulated ground motions. The use of simulated ground motion is an appropriate approach where only a limited number of real ground motion records exist for a particular magnitude, focal distance, local geologic conditions, etc.

Similar studies for other base-isolation parameters have been reported in the literature (e.g., Constantinou and Tadjbaksh).

Section 8.1 describes a methodology for generation of synthetic ground motion time histories for use in a probabilistic study of nonlinear response. Section 8.2 presents the results for the response of the four-story base-isolated structure to these simulated ground motions.

8.1 GENERATION OF SIMULATED GROUND MOTIONS FOR USE IN STATISTICAL STUDIES OF NONLINEAR RESPONSE

The simplest method for the simulation of earthquakes is through the use of white noise. White noise adequately represents ground motions for the study of structures whose natural frequency falls near the approximately flat portion of the expected velocity spectrum. While this is close to the case for the base isolated structure, more realistic simulated ground motions were generated using the procedure described by Murakami and Penzien (1975). The procedure consists of the following five steps:

(1) Stationary white noise having a specified constant power spectral density is generated. The procedure uses a series of pseudo random numbers with Gaussian distribution assigned to an equally spaced time interval. The time interval was taken as 0.01 second, such that the resulting power spectral density function is nearly constant over the lower range of frequencies (0 to 10 Hz).

(2) Nonstationary white noise is obtained by multiplying each stationary segment of white noise

by a prescribed envelope function. The time intensity envelope used for this study is shown in Figure 53 and is representative of motions close to the fault during a Magnitude 7 earthquake.

(3) The resulting nonstationary noise is passed through a filter which amplifies frequency content in the neighborhood of a characteristic ground frequency while attenuating higher frequencies. Filter characteristics were taken to be those suggested by Kanai (1957) for firm soil conditions.

(4) The wave is then passed through a filter to eliminate very low frequency content, i.e., periods on the order of seven seconds and greater.

(5) A baseline correction procedure is applied to the synthetic accelerograms in order to produce realistic integrated displacements and velocities.

The simulation procedure was used to generate 50 synthetic accelerograms. Figure 54 shows a plot of a typical accelerogram generated using the procedure. All accelerograms were normalized to a peak ground acceleration of 0.6g.

8.2 RESPONSE OF A FOUR-STORY STRUCTURE WITH BASE ISOLATION TO SIMULATED EARTHQUAKES

The four-story structure described in Chapter 6.0 was analyzed to determine response due to each of the 50 simulated horizontal ground accelerations obtained as described above. Time histories of response were determined using the numerical integration procedures described in Chapter 4.0. The analysis was repeated for each simulated ground motion for the following two cases:

- A base isolation scheme using reinforced elastomer bearings alone.
- A base-isolation scheme using reinforced elastomer bearings with a slip surface.

For each of the models and each of the 50 simulated ground motions, the response time histories were determined. A typical relative displacement time history is shown in Figure 55. The responses which were examined included maximum relative displacements in the base isolation system, maximum base shear, and maximum overturning moments. These responses are presented in the form of probability distribution functions as described below.

It has been demonstrated previously (Murakami and Penzien, 1975) that the maximum response to a single earthquake follows closely the extreme value or Gumbel Type I distribution which is defined by the relations:

$$F(x) = \exp(-e^{-y})$$
$$y = \alpha(x - \mu)$$

where

α, μ = parameters of distribution,

x = extreme value of response,

y = reduced extreme value, and

$F(x)$ = cumulative probability distribution function.

The parameter α and μ were estimated from the relations:

$$1/\alpha = \frac{\sigma_x}{\sigma_y}$$

$$u = \bar{x} - \bar{y}/\alpha$$

where σ_x and $\bar{x}$ are the standard deviation and mean, respectively, of the extreme values of response while σ_y and $\bar{y}$ are the standard deviation and mean, respectively, of the reduced extreme values, y.

Extreme values of response quantities were ranked in order of increasing value and plotted on Gumbel extreme value probability paper. On this scale, the Type I extreme value distribution appears as a straight line. Figures 56, 57, and 58 present the resulting probability plots, respectively, for maximum relative displacement of the bearings, maximum shear force at the base pads, maximum overturning moment at the base of the building, respectively.

Table 8-1 summarizes the statistical parameters for the probability distributions which are shown in Figures 56, 57, and 58. It is observed from Figure 56 that introducing the friction pad on the model does not significantly affect the distribution of maximum relative displacement of the bearings. However, the mean value and standard deviation of the base shear and overturning moment are significantly reduced by incorporating the friction surface.

TABLE 8-1

STATISTICAL PARAMETERS FOR FOUR-STORY STRUCTURE
SUBJECTED TO 50 SIMULATED EARTHQUAKES

	RANDOM VARIABLE, χ					
	BASE DISPLACEMENT (ft)		MAXIMUM SHEAR (kips)		MAXIMUM OVERTURNING MOMENT (kip-ft)	
	MODEL 1	MODEL 2	MODEL 1	MODEL 2	MODEL 1	MODEL 2
Mean, $\bar{\chi}$	0.862	0.8181	26,460	55,850	1.123×10^6	2.336×10^6
Standard Deviation, σ_χ	0.2959	0.2067	1,000	14,070	0.045×10^6	0.588×10^6
Coefficient of Variation, CV	34.2%	25.3%	3.8%	25.2%	4.0%	25.2%
Gumbel Dispersion Parameter, $1.0/\alpha$	0.2524	0.1763	900	12,000	0.038×10^6	0.502×10^6
Gumbel Mode Parameter, u	0.7268	0.7214	26,000	49,270	1.101×10^6	2.061×10^6

Model 1 - Four-story structure with base isolation including friction pad with $\mu = 0.2$.
Model 2 - Four-story structure identical to Model 1 except without friction pad.

It is concluded that the base-isolated response follows closely the Gumbel Type I distribution. Further, it can also be concluded that because structural mode participation is rather small, the above results within some limits would be applicable to other structures as well. The distributions of relative displacements and forces provide some guidance in establishing the design of base-isolation schemes.

9.0 SUMMARY AND CONCLUSIONS

The feasibility of the base-isolation concept as an aseismic design strategy has been investigated through analysis of case studies and evaluation of its impact on the seismic design of structures. The base-isolation concept reviewed uses reinforced elastomer bearings with or without additional damping devices.

It is concluded that the concept is sound. It is technically feasible to design and incorporate a base-isolation scheme using reinforced elastomer bearings in the foundation of structures for their earthquake protection. Base isolation is most effective in reducing seismic forces on the structure when the structure is rigid (fundamental frequency in the range from about 3 to 10 Hz), if conventionally designed.

Some of the main conclusions of the study are as follows:

- For rigid type structures, base isolation can significantly reduce the seismic forces on the structure and components.

- Among the base-isolation schemes, the bearing with friction surface effects the most significant reduction for structures at the rigid end, while a high damping bearing is more successful for more flexible structures.

- The seismic behavior of the base-isolated structures is rather uncomplicated compared to that of the conventional structure. A more simple mathematical model is usually sufficient to predict the base-isolated response. Also, the seismic response of base-isolated structures is more predictable.

- The seismic forces within the base-isolated structures are reduced to levels for which, within the constraints of economics, the structure can be designed to remain below the ultimate capacity of structural elements.

- Properly designed, base isolation can reduce the seismic risk to the building structure, its contents, and occupants.

- For the structure analyzed, the additional cost of incorporating a base-isolation scheme is offset by the cost savings in structural items. It is concluded that, in general, the additional earthquake protection with base isolation can be provided at equal or lower costs.

- Cost optimization studies should be performed so that, in synthesis with the base-isolation design optimization, a strategy and a method for value analysis may be developed.

- At present, no general rules for seismic design of base-isolated structures can be established; however, this is not seen as a severe handicap because a relatively simple mathematical model satisfies the requirement of a rational dynamic analysis.

- More case studies need to be analyzed to derive general rules for design of base-isolated structures.

- Existing rules for the design of reinforced elastomer bearings can be used to establish the size, thickness, and number of such bearings which will satisfy the aseismic design requirements. However, the physical properties of elastomers, weathering effects, and effects of long-term changes on the mechanical properties of the bearing need to be addressed.

- The bearing is the all important structural element in the aseismic design scheme. Its behavior under dynamic conditions needs to be analyzed in more detail to investigate its failure modes.

- From a probabilistic analysis, it is concluded that the cumulative probabilities of displacements, base shear, and overturning moments, given a peak ground acceleration, follow the Gumbel Type I distribution.

- Variability in bearing material properties and the dynamic characteristics of the structures need to be included in a probabilistic analysis to develop risk assessment for base-isolated design.

REFERENCES

LIST OF REFERENCES

Agbabian Associates, 1979, "The Feasibility of Seismic Isolation of Nuclear Power Plant Systems," topical report prepared for The Division of Reactor Research and Technology, Department of Energy, Washington, D.C., under Contract No. DE-AC-0376SF01011.

American Society for Testing and Materials, 1981, "Standard Specifications for Plain and Steel-Laminated Elastomer Bearings for Bridges," ASTM D4014-81, 1916 Race Street, Philadelphia, PA.

Chopra, A. K., D. P. Clough, and R. W. Clough, 1973, "Earthquake Resistance of Buildings with a 'Soft' First Story," Earthquake Engineering and Structural Dynamics, Vol. 1, No. 4.

Dames and Moore, 1979, "A Review of Seismic Isolation for Nuclear Structures," EPRI Report NP-1220-SR, special report prepared for the Electric Power Research Institute by Dames and Moore Advanced Technology Group, London, England.

D'Appolonia Consulting Engineers, 1980, "Base Isolation for Power Plant Structures, An Aseismic Design Concept," Technical Document, 78-255, December 1980.

D'Appolonia Consulting Engineers, 1979, "Synthesis of Responses to Licensing Operation," Atomic Energy Organization of Iran, 78-255, June 1979.

Delfosse, G. C., 1977, "The GAPEC System: A New Highly Effective Aseismic Design," Proceedings, Sixth World Conference on Earthquake Engineering, New Delhi, India.

Derham, C. J. and A. G. Thomas, 1981, "The Design of Seismic Isolation Bearings," Control of Seismic Response of Piping Systems and Other Structures by Base Isolation, J. M. Kelly (ed.), Report No. UCB/EERC-81/01, Earthquake Engineering Research Center, University of California, Berkeley.

DuPont de Nemours and Company, 1983, "Engineering Properties of Neoprene Bridge Bearings," Wilmington, Delaware.

Engineering News Record, 1983, "Law Building Can 'Roll with Punch,'" McGraw-Hill Publication, August 4.

Imbsen, R. A. and R. A. Schamber, "Increased Seismic Resistance of Highway Bridges Using Improved Bearing Design Concepts," Vol. I, Final Report to U.S. Department of Transportation, Federal Highway Administration, Engineering Computer Corporation, Sacramento, California.

LIST OF REFERENCES (Continued)

Jolivet, F. and M. Richli, 1977, "Aseismic Foundation System for Nuclear Power Stations," Transactions of the Fourth International Conference on Structural Mechanics in Reactor Technology, Vol. K, No. 9/2, San Francisco, California.

Kanai, K., "Semi-Empirical Formula for Seismic Characteristics of the Ground," Bulletin of Earthquake Research Institute, University of Tokyo, Japan, Vol. 35, June 1957.

Kelly, J. M., R. I. Skinner, and A. J. Heine, 1972, "Mechanisms of Energy Absorption in Special Devices for Use in Earthquake Resistant Structures," Bulletin of New Zealand Society for Earthquake Engineering, Vol. 5, No. 3, pp. 63-88.

Kelly, J. M. and D. F. Tsztoo, 1978, "The Development of Energy-Absorbing Devices for Aseismic Base Isolation Systems," Report No. UCB/EERC-78/01, Earthquake Engineering Research Center, University of California, Berkeley.

Kelly, J. M., 1979, "Aseismic Base Isolation: A Review," Proceedings of the Second U.S. National Conference on Earthquake Engineering, Earthquake Engineering Research Institute, Stanford, California.

Kelly, J. M. and S. B. Hodder, 1981, "Experimental Study of Lead and Elastometric Dampers for Base Isolation Systems," Report No. UCB/EERC-81/16, Earthquake Engineering Research Center, University of California, Berkeley.

Kircher, C. E., G. C. Delfosse, C. C. Schoof, O. Khemici, and H. C. Shah, 1979, "Performance of a 230 kV ATB 7 Power Circuit Breaker Mounted on GAPEC Seismic Isolators," Report No. 40, the John A. Blume Earthquake Engineering Center, Stanford University, Stanford, California.

Lee, D. M., P. C. Jennings, and A. W. Honsner, "A Selection of Imortant Stron Motion Earthquake Records," Earthquake Engineering Research Laboratory, Report No. EERL 80-01, California Institute of Technology, January 1980.

Megget, L. M., "Analysis and Design of a Base-Isolated Reinforced Concrete Farm Building," Bulletin of the New Zealand Society of Earthquake Engineers, Vol. II, No. 4.

Murakami M. and J. Penzien, "Nonlinear Response Spectra for Probabilistic Seismic Design and Damage Assessment of Reinforced Concrete Structures," Report N. EERC 75-38, Earthquake Engineering Research Center, Berkeley, California, November 1975.

LIST OF REFERENCES
(Continued)

Newmark, N. M., 1979, "Aseismic Bearing Pads for Nuclear Power Plants and Other Applications," a report to D'Apppolonia, November 1979.

Pavot, B. and E. Polus, 1979, "Aseismic Bearing Pads," Tribology International.

Plichon, C. E., 1975, "Hooped Rubber Bearing and Frictional Plates: A Modern Antiseismic Engineering Technique," Proceedings, Specialist Meeting on the Antiseismic Design of Nuclear Installations, Paris, France.

Richli, M., I. V. Constantopoulos, and N. R. Vaidya, 1980, "Seismic Design Considerations for Structures Supported on Aseismic Bearing Pads," accepted for publication at the Seventh World Conference on Earthquake Engineering, Istanbul, Turkey.

"Seismic Design for Buildings," 1974, Department of the Army, the Navy and the Air Force, TM 5-809-10, Nav Fac P-355, and AFM 88-3, U.S. Government Printing Office, 1974-559-231/1044.

Sharpe, R. L., Kost, G., and Lord, J., "Behavior of Structural Systems Under Dynamic Loads," Building Practices for Disaster Mitigation, Building Science Series 46, Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402, pp. 352-395.

Skinner, R. I. and G. W. McVerry, 1975, "Base Isolation for Incrased Earthquake Resistance of Buildings," Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 8, No. 2, pp. 93-101.

Skinner, R. I., R. G. Tyler, and A. J. Heine, 1980, "Steel-Beam Dampers for Increasing the Earthquake Resistance of Structures," presented at the Seventh World Conference on Earthquake Engineering, Istanbul, Turkey.

Skinner, R. I., R. G. Tyler, A. J. Heine, and W. H. Robinson, 1980, "Hysteretic Dampers for the Protection of Structures from Earthquakes," Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 13, No. 1, March.

Stanton, J. F. and C. W. Roeder, "Elastomer Bearings Design, Construction, and Materials," National Cooperative Highway Research Program Report 248, Transportation Research Board, National Research Council, Washington, D.C., August 1982.

Structural Engineers Association of California, 1974, Seismology Committee, "Recommended Lateral Force Requirements and Commentary."

LIST OF REFERENCES
(Continued)

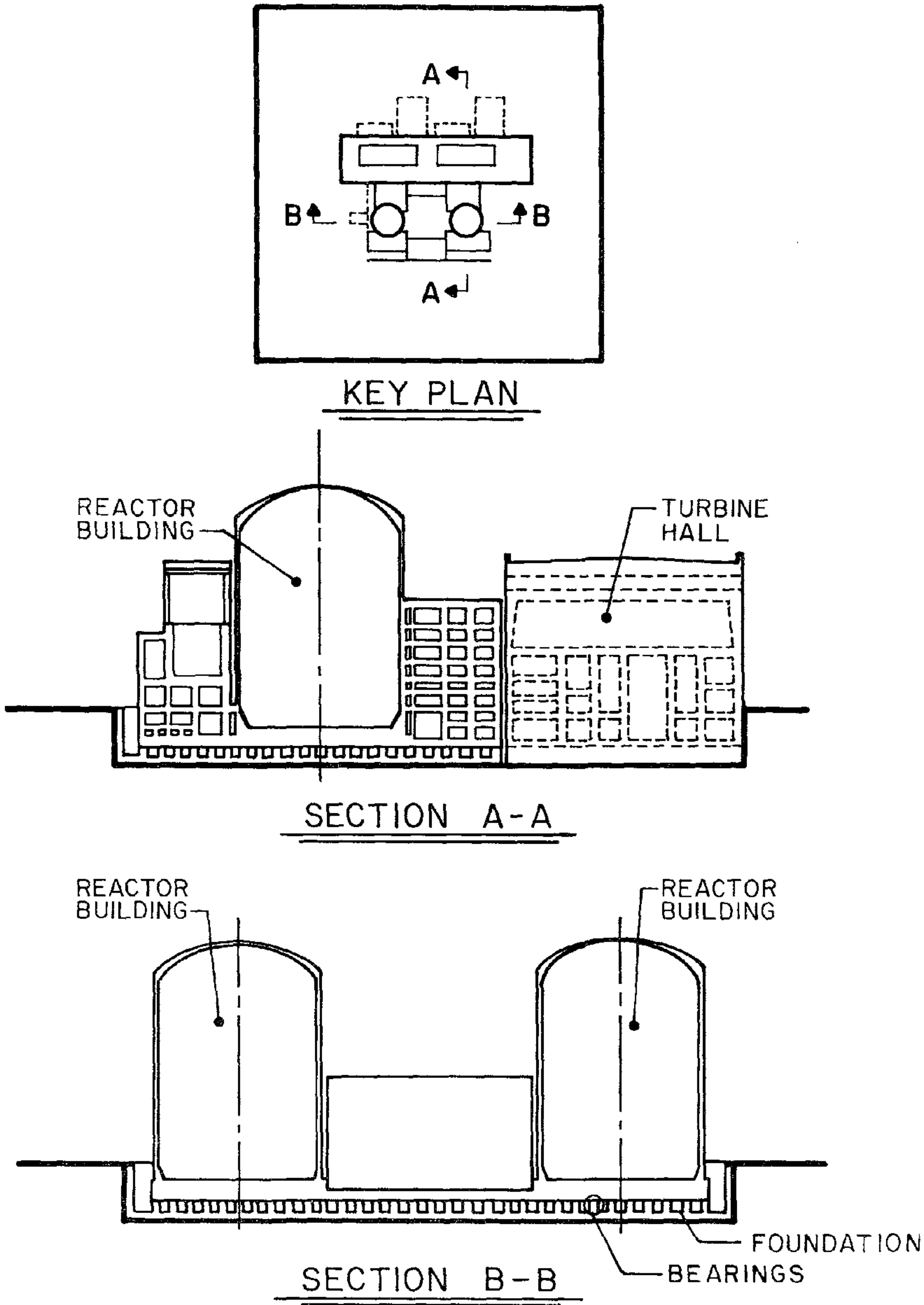
Tarics, A. G., 1982, "Cost Consideration of Base Isolation," International Conference on National Rubber for Earthquake Protection of Buildings and Vibration Isolation, Kuala Lampur, Malasia.

"Uniform Building Code," 1979 Edition, International Conference of Building Officials, Whittier, California.

Veterans Administration, 1973, "Earthquake Resistant Design Requirements for VA Hospital Facilities," Appendix B, Handbook H-08-8, Office of Construction, Veterans Administration, Washington, D.C., June.

Waller, R. A., 1969, Building on Springs," Pergamon Press, Oxford.

FIGURES



REF. JOLIVET & RICHLI, 1977

FIGURE I BASE ISOLATION CONCEPT

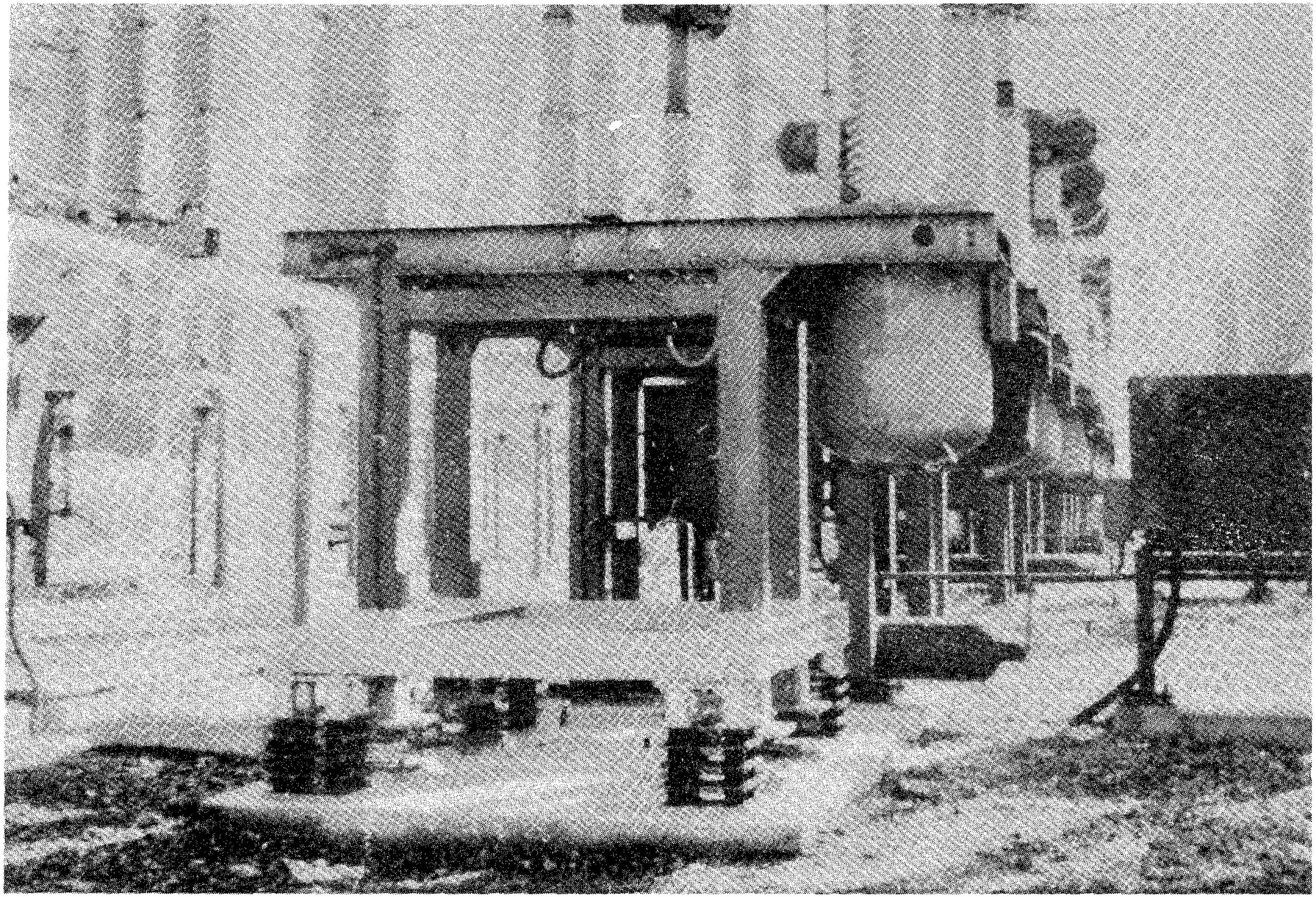
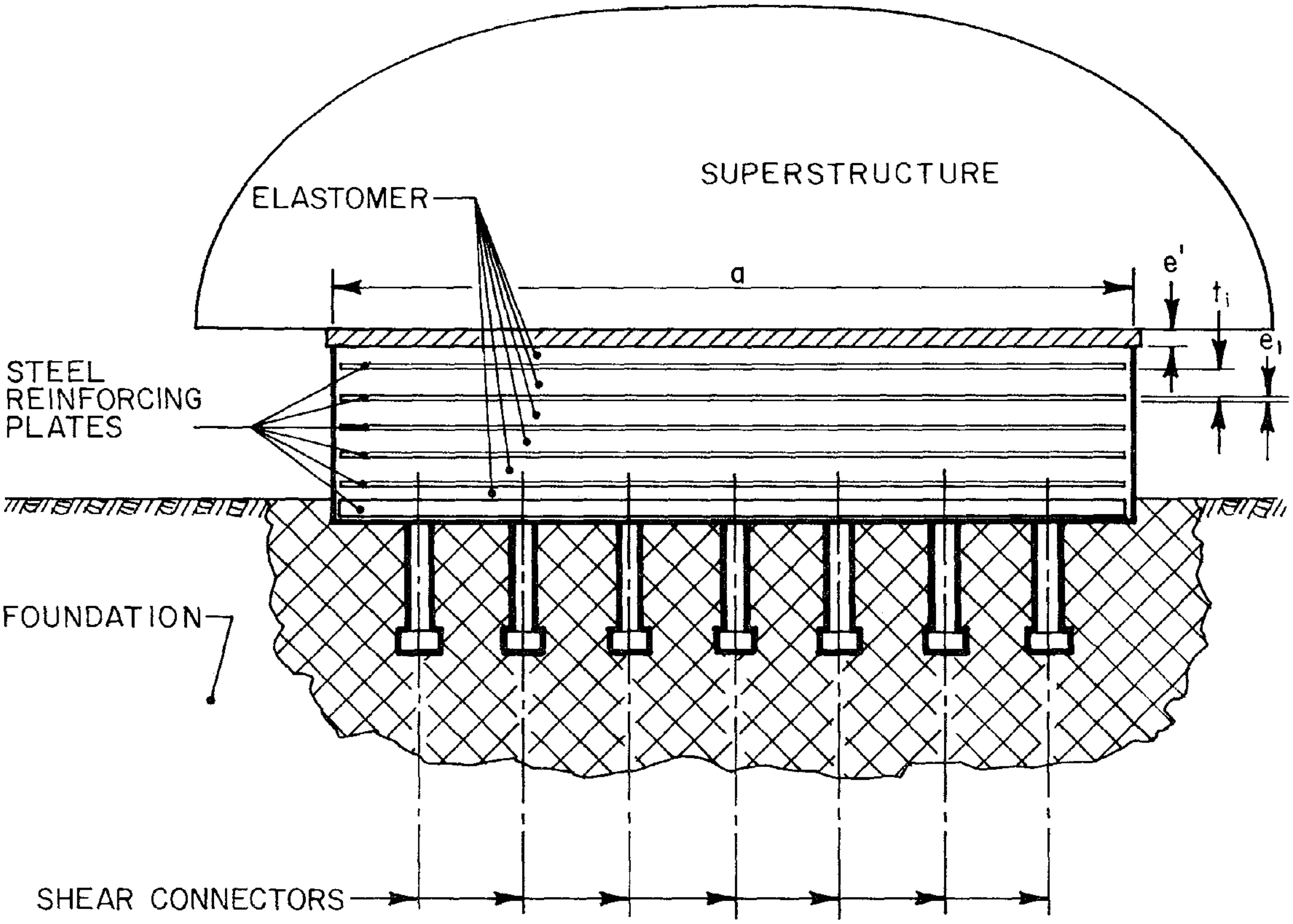


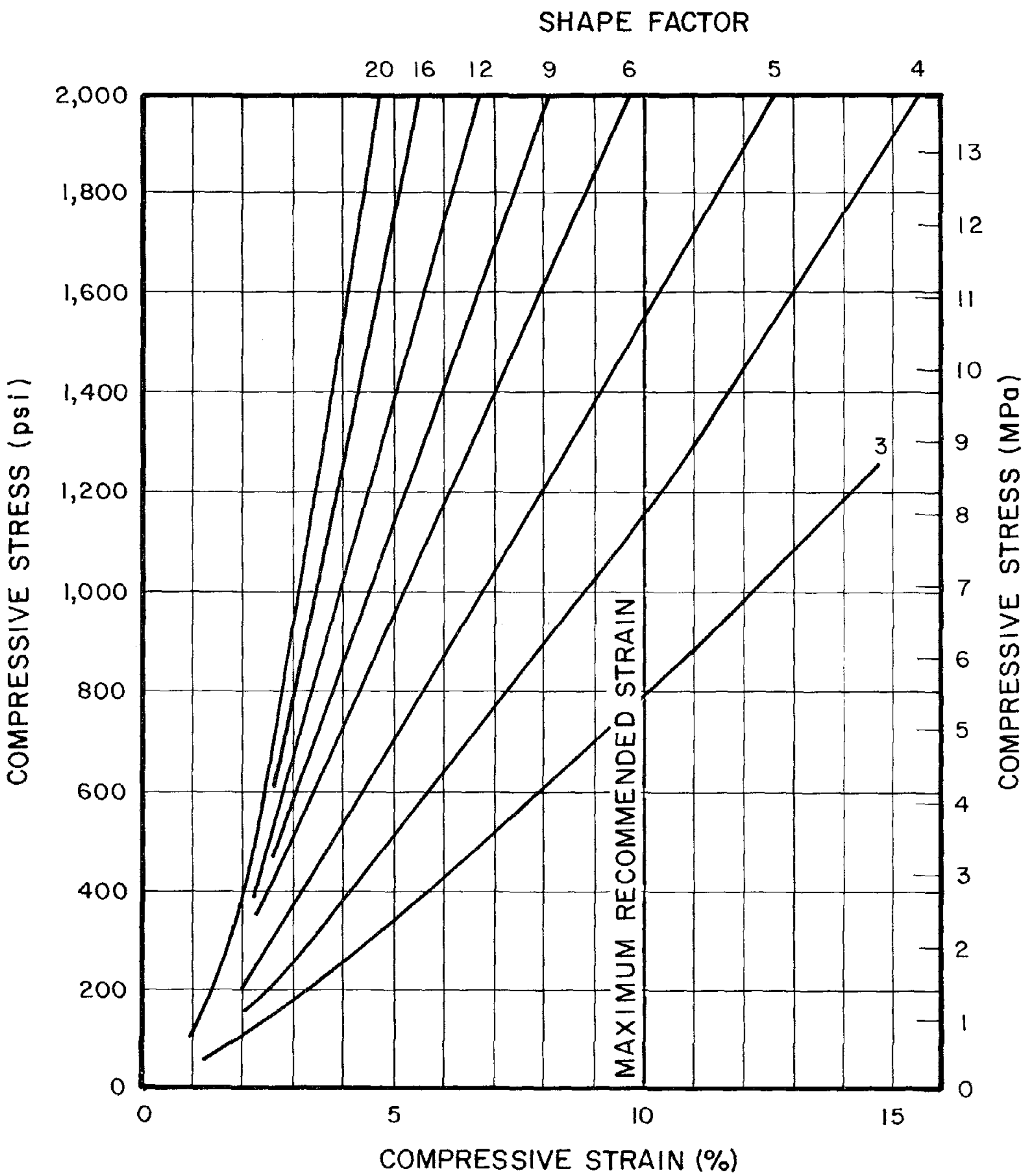
FIGURE 2 SUPPORT STRUCTURE OF A 230 KV CIRCUIT BREAKER ON BASE ISOLATION BEARINGS



LEGEND

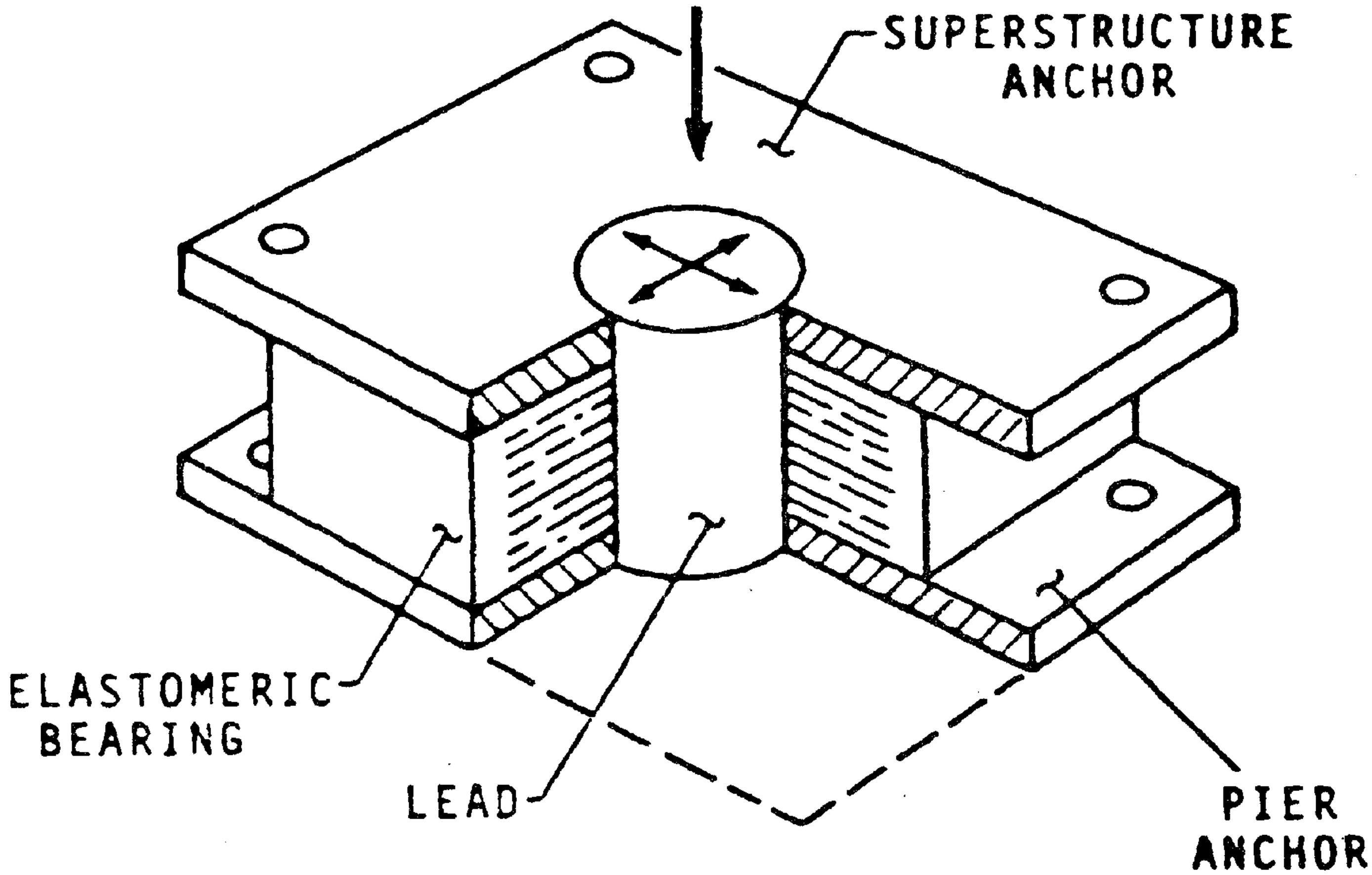
a PLAN DIMENSION OF BEARING
t_i THICKNESS OF ELASTOMER LAYER i FROM THE BOTTOM
e_i THICKNESS OF THE STEEL REINFORCING PLATE IN LAYER i FROM THE BOTTOM
e' THICKNESS OF BEARING PLATE

FIGURE 3 DETAIL OF BASE ISOLATION BEARING



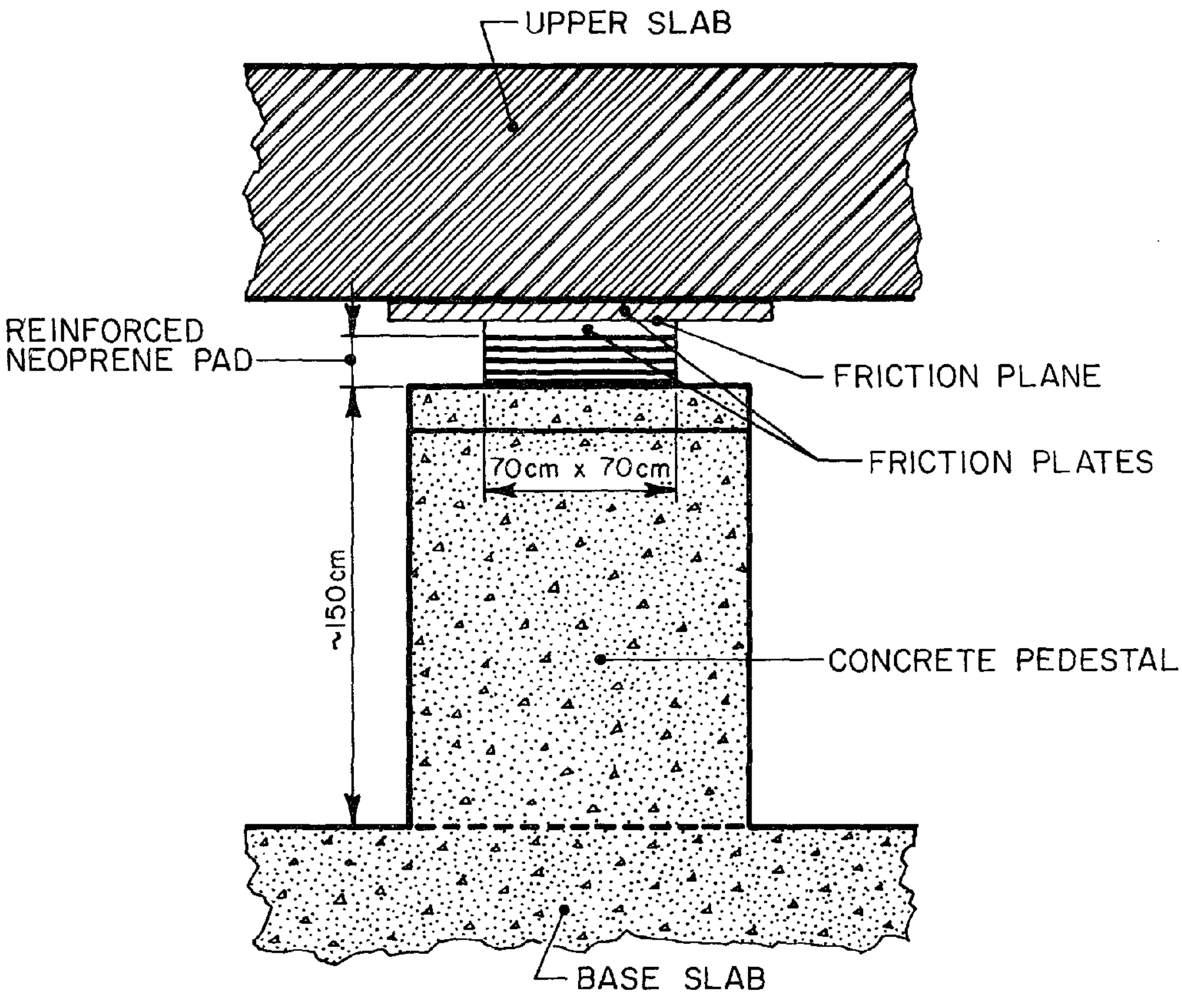
REF. DUPONT, 1983

FIGURE 4 LOAD/DEFLECTION IN COMPRESSION, PLAIN AND STEEL REINFORCED BEARINGS-50 HARDNESS



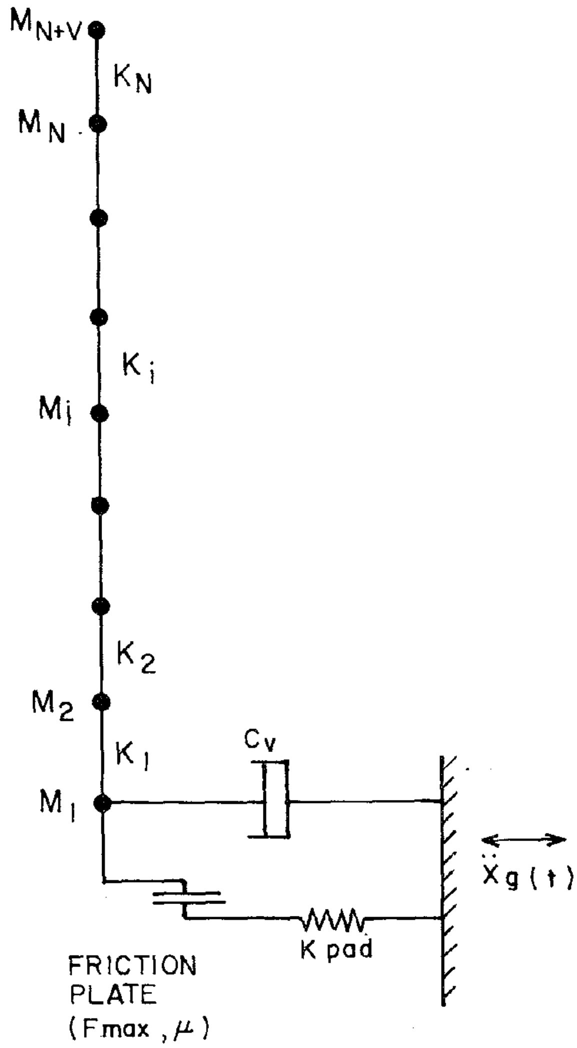
REF. SKINNER AND McVERRY, 1975

FIGURE 5 REINFORCED ELASTOMER BEARING WITH LEAD PLUG



REF. PLICKON, 1975

FIGURE 6 REINFORCED ELASTOMER BEARING WITH FRICTION SURFACE



LEGEND

$\ddot{X}g(t)$	HORIZONTAL GROUND ACCELERATION
N	NUMBER OF STORIES
M_i	LUMPED MASS AT FLOOR i
K_i	STIFFNESS OF STORY i
C_v	VISCOUS DAMPING AT BASE
K_{pad}	LINEAR STIFFNESS OF PAD
μ	COEFFICIENT OF FRICTION
F max	μ TOTAL WEIGHT

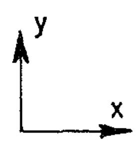
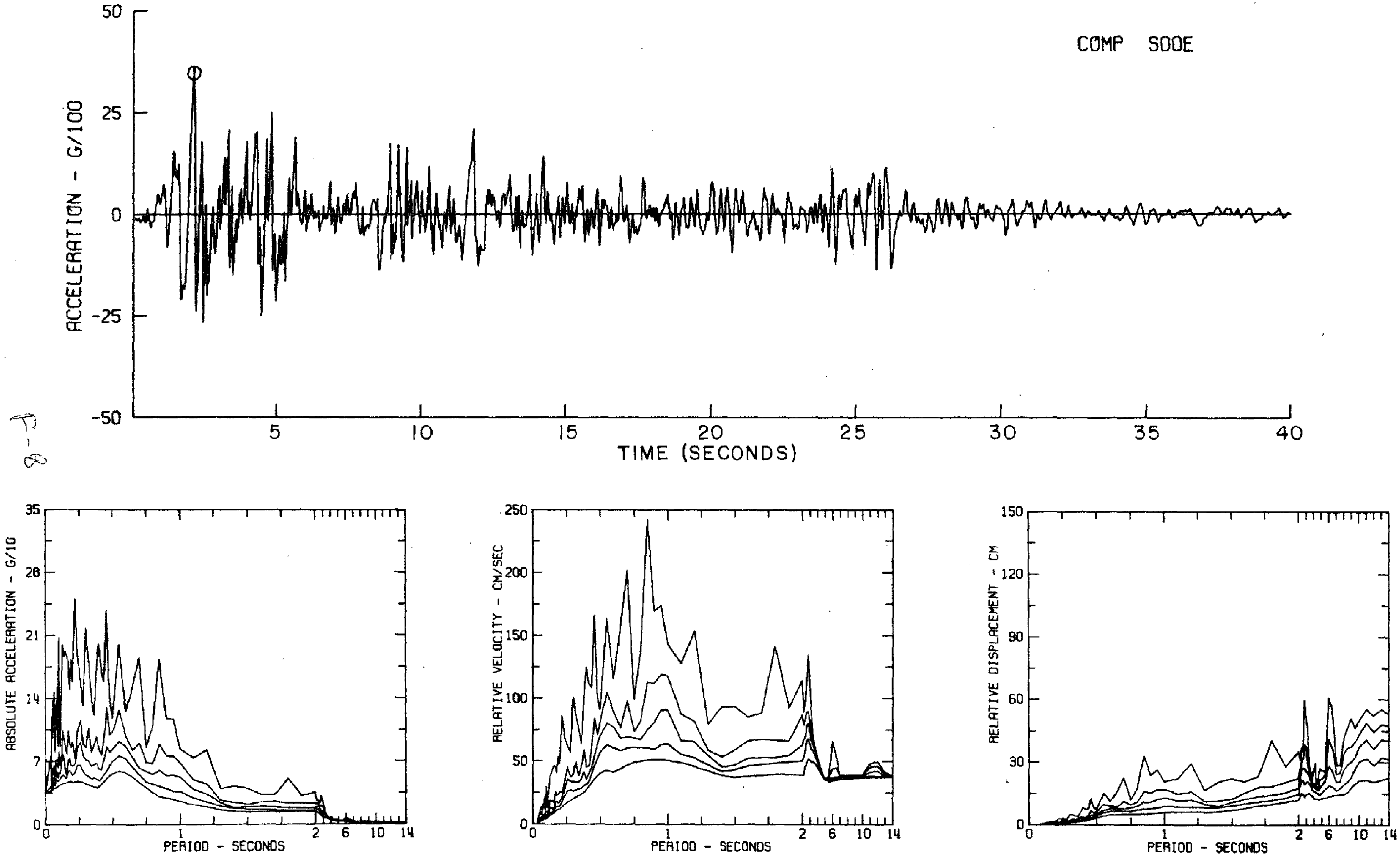
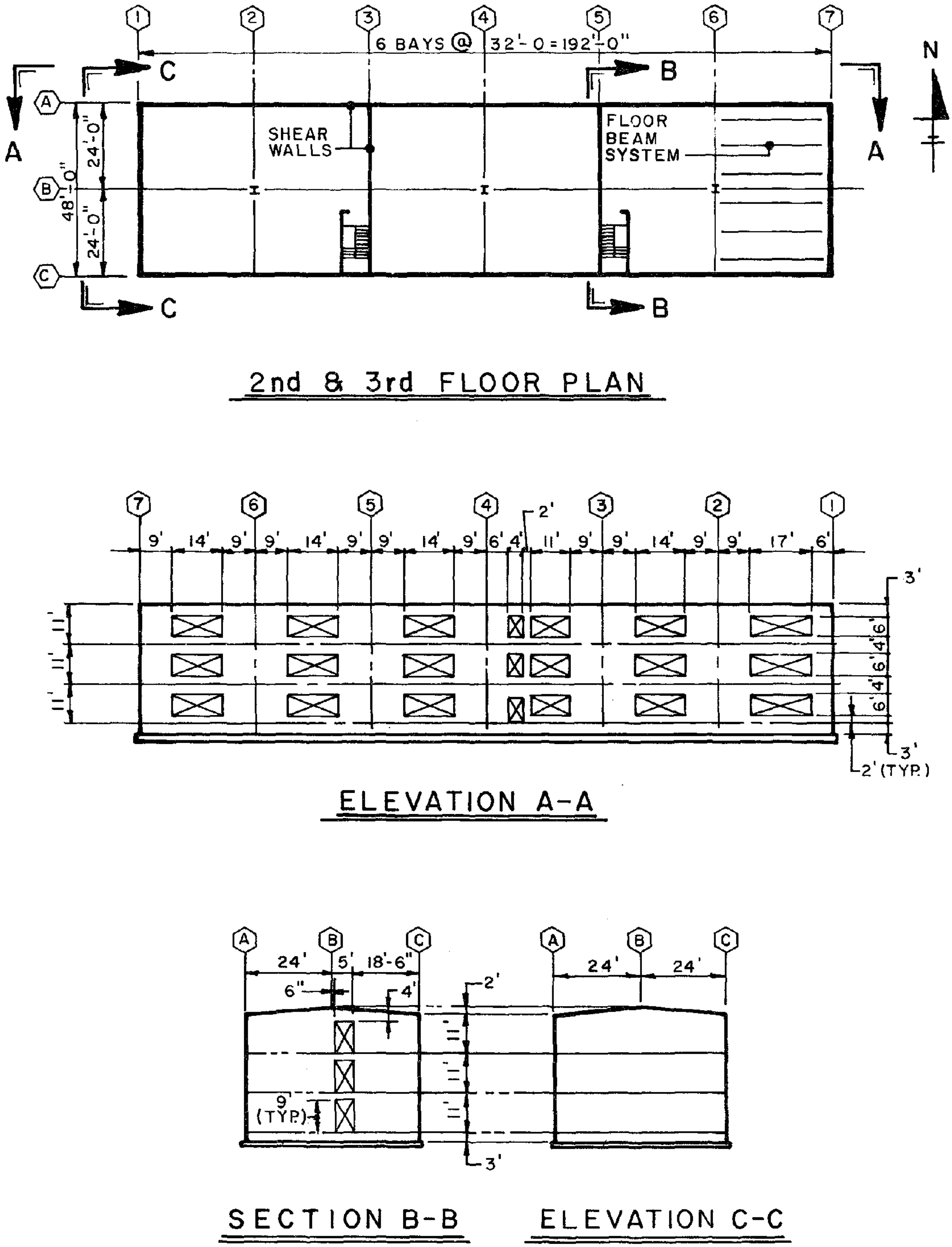


FIGURE 7 MODEL OF SHEAR BEAM STRUCTURE ON BASE ISOLATION



REF. LEE, et. al., 1980

FIGURE 8 EL CENTRO, 1940 S00E RECORD AND ITS RESPONSE SPECTRUM



REFERENCE—NAVFAC P-355(1973)

FIGURE 9 STRUCTURAL CONFIGURATION
CASE STUDY I

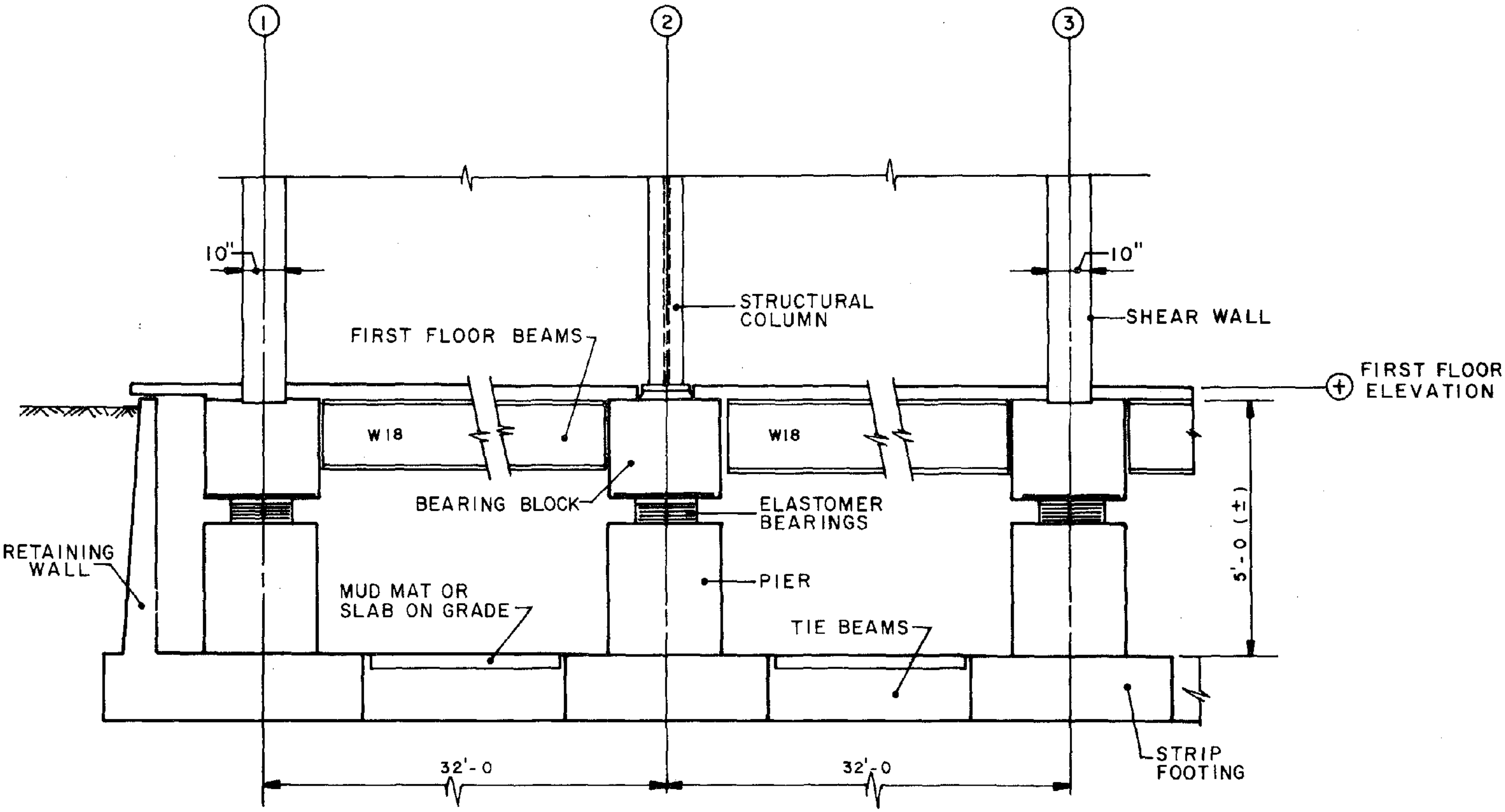
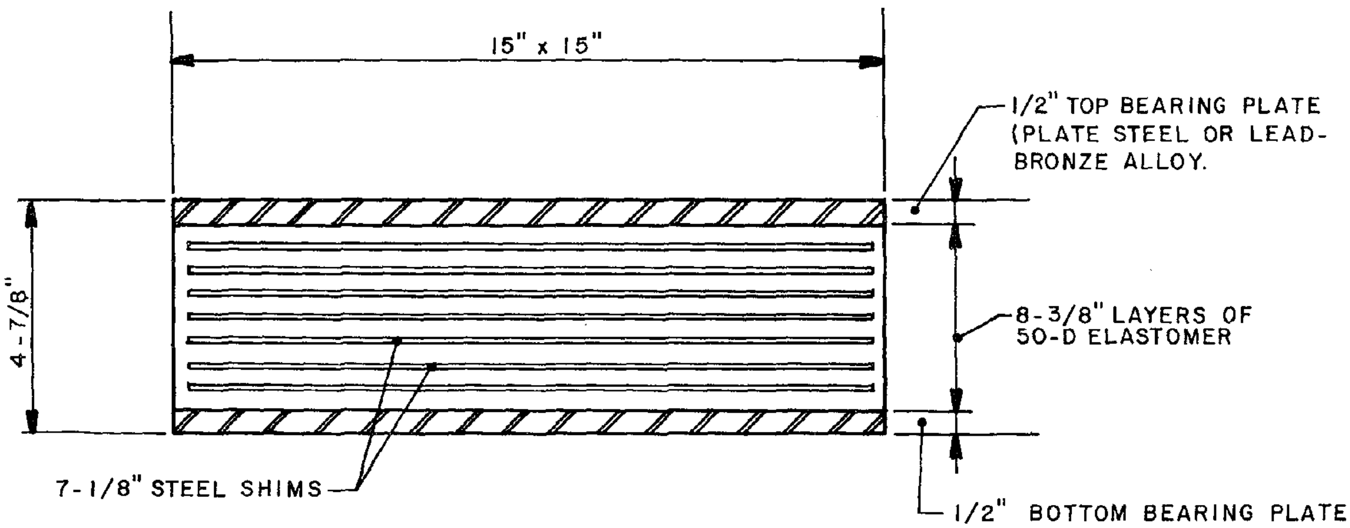


FIGURE 10 CONCEPTUAL DESIGN OF BASE ISOLATION, CASE STUDY I



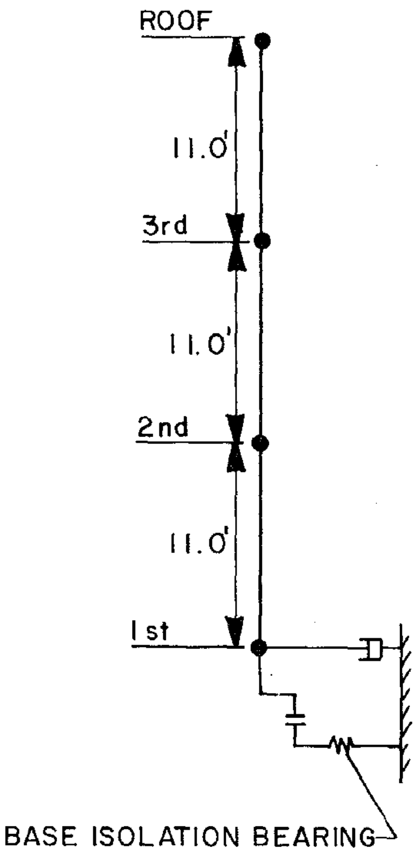
SHAPE FACTOR, $S = \frac{A}{A_s} = 10$

HORIZONTAL STIFFNESS, $K = \frac{AG}{T} = 11.25$ KIPS/IN.

WHERE,

A = LOADED AREA
A_s = AREA FREE TO BULGE
G = SHEAR MODULUS OR ELASTOMER
T = TOTAL ELASTOMER THICKNESS

FIGURE 11 TYPICAL BEARING DETAIL, CASE STUDY I



LEVEL	WEIGHT (KIPS)	MASS K-Sec²/Ft
ROOF	704	21.863
3rd	1475	45.808
2nd	1475	45.808
1st	1475	45.808
TOTAL	5129	159.286

STORY	HORIZONTAL STIFFNESS (K/Ft)
3	2.74×10^6
2	2.90×10^6
1	2.90×10^6
BEARINGS	2835

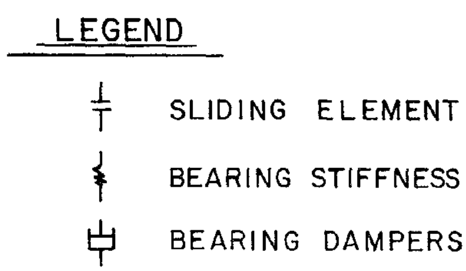
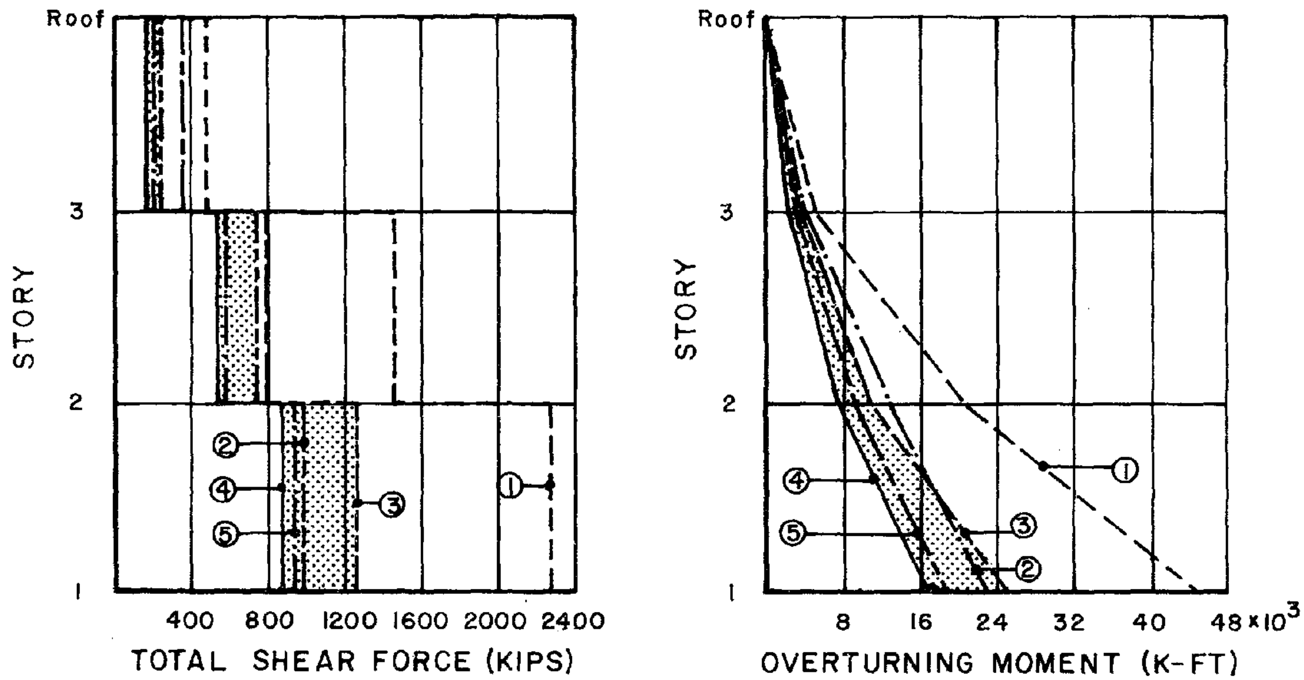


FIGURE 12 MATHEMATICAL MODEL
CASE STUDY I



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

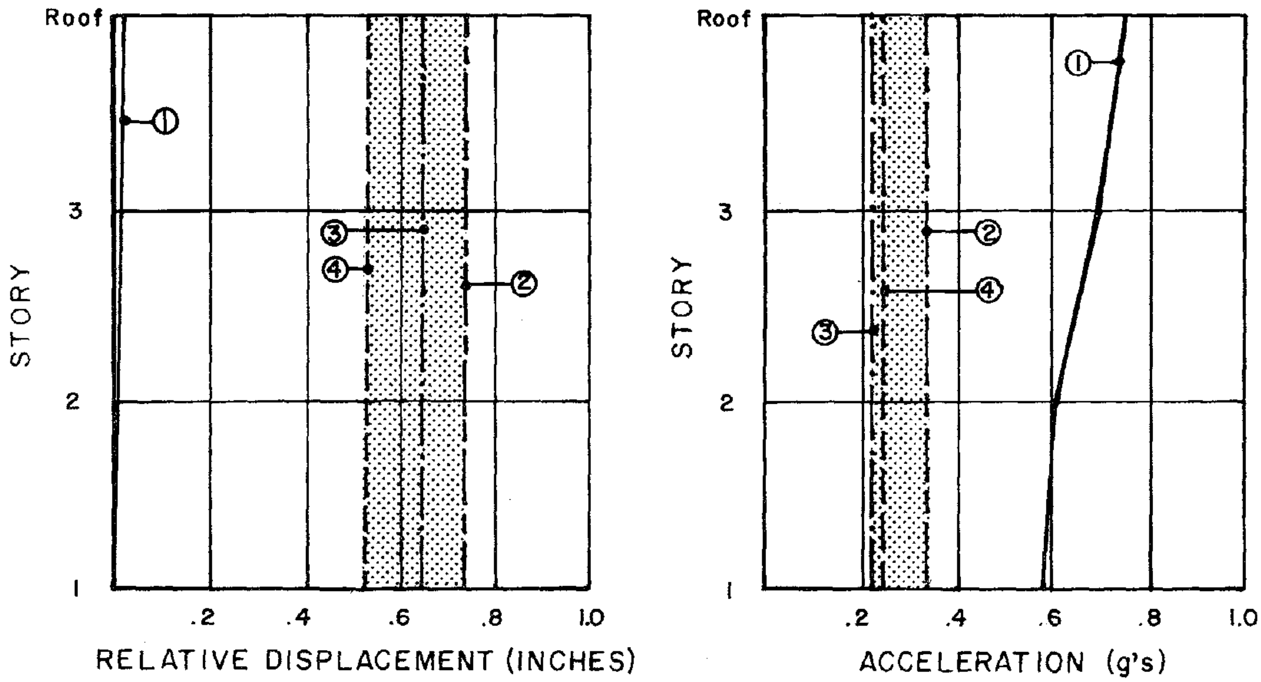
⑤ 15% DAMPED BEARING

---

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

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

FIGURE 13 STORY SHEARS AND OVERTURNING MOMENTS
CASE STUDY I



LEGEND

CONVENTIONAL BUILDING

① LINEAR ELASTIC ANALYSIS(1)

BASE-ISOLATED BUILDING(1)

② 5% DAMPED BEARING

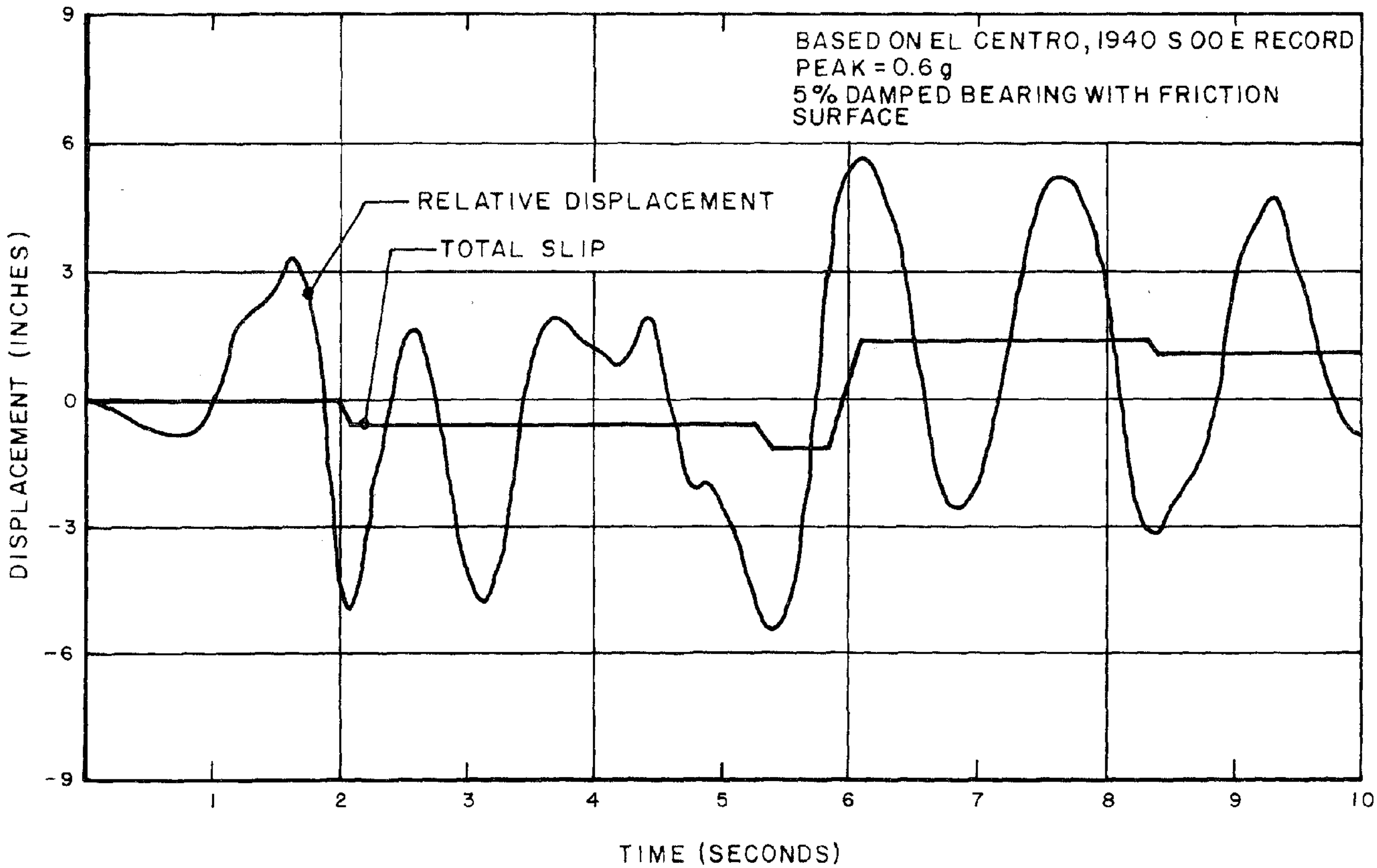
③ 5% DAMPED BEARING WITH FRICTIONAL INTERFACE

④ 15% DAMPED BEARING

---

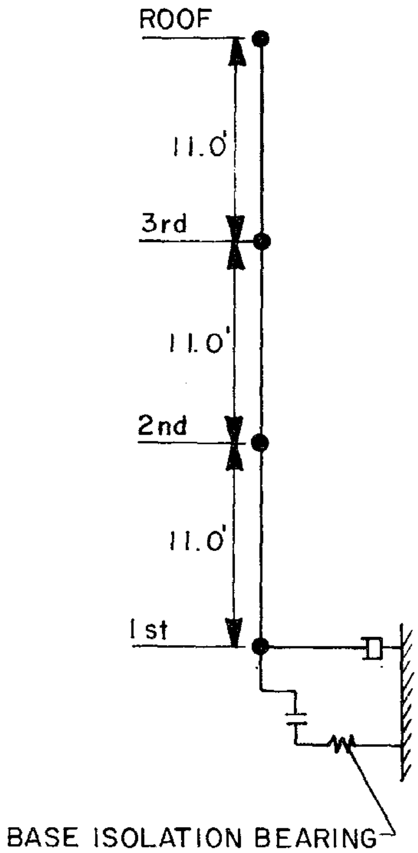
(1) BASED ON EL CENTRO 1940 S 00 E RECORD SCALED TO 0.6 g PEAK.

FIGURE 14 DISPLACEMENT AND ACCELERATIONS CASE STUDY I



F-15

FIGURE 15 DISPLACEMENT TIME HISTORY
CASE STUDY I



LEVEL	WEIGHT (KIPS)	MASS (K-SEC/Ft^2)
ROOF	375	11.634
3rd	799	24.814
2nd	799	24.814
1st	799	24.814
TOTAL	2772	86.076

STORY	HORIZONTAL STIFFNESS (K/Ft)
3	3,042
2	8,145
1	11,211
BEARINGS	1890

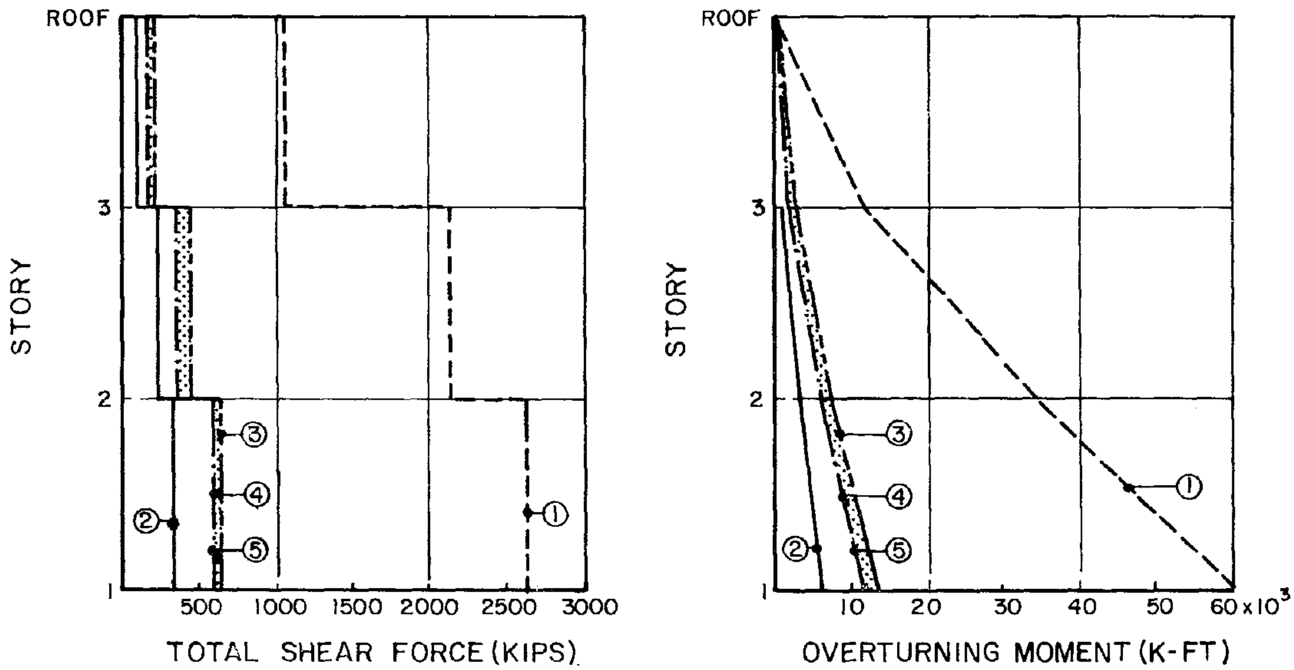
LEGEND


SLIDING ELEMENT


BEARING STIFFNESS


BEARING DAMPERS

FIGURE 18 MATHEMATICAL MODEL
CASE STUDY II



LEGEND

CONVENTIONAL BUILDING

① LINEAR ELASTIC ANALYSIS (1)(2)

② ZONE 4 (SEAOC, 1974)

BASE-ISOLATED BUILDING (2)

③ 5% DAMPED BEARING

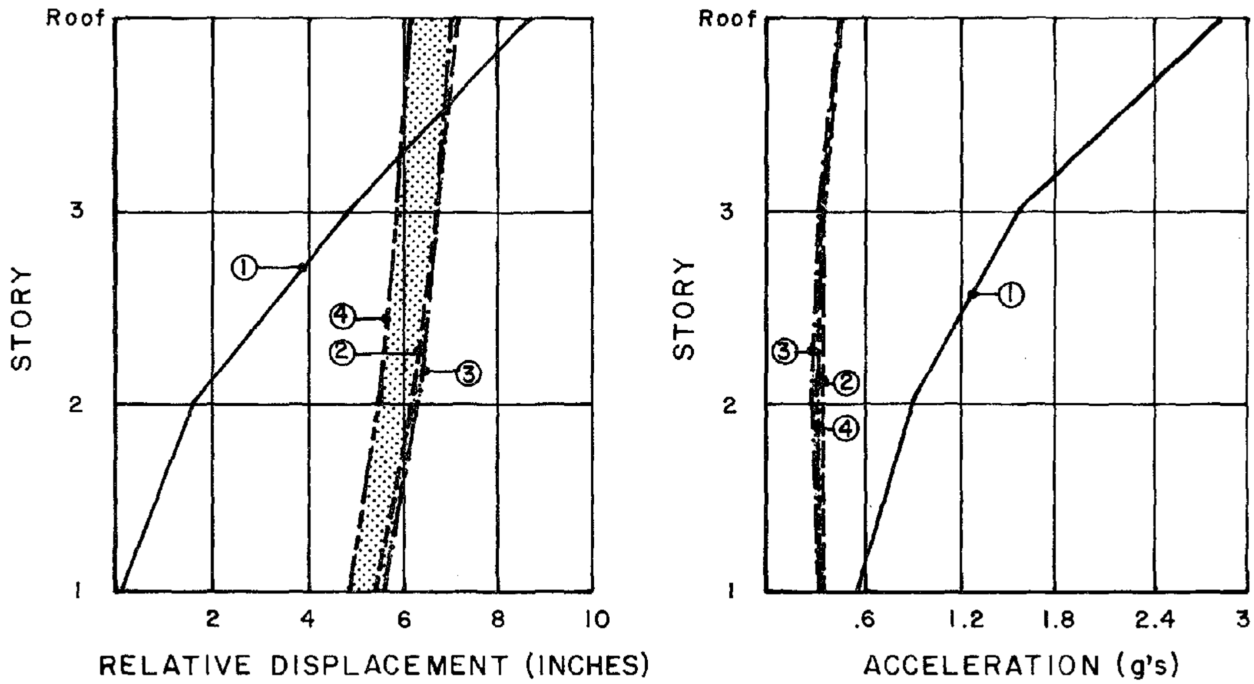
④ 5% DAMPED BEARING WITH FRICTIONAL INTERFACE

⑤ 15% DAMPED BEARING

(1) PROVIDED AS AN APPROXIMATE MEASURE OF LINEAR ELASTIC DISPLACEMENTS

(2) BASED ON EL CENTRO 1940, SOOE RECORD SCALED TO 0.6g PEAK

FIGURE 19 STORY SHEARS AND OVERTURNING MOMENTS, CASE STUDY II



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 S 00 E RECORD SCALED TO 0.6 g PEAK.

FIGURE 20 DISPLACEMENTS AND ACCELERATIONS CASE STUDY II

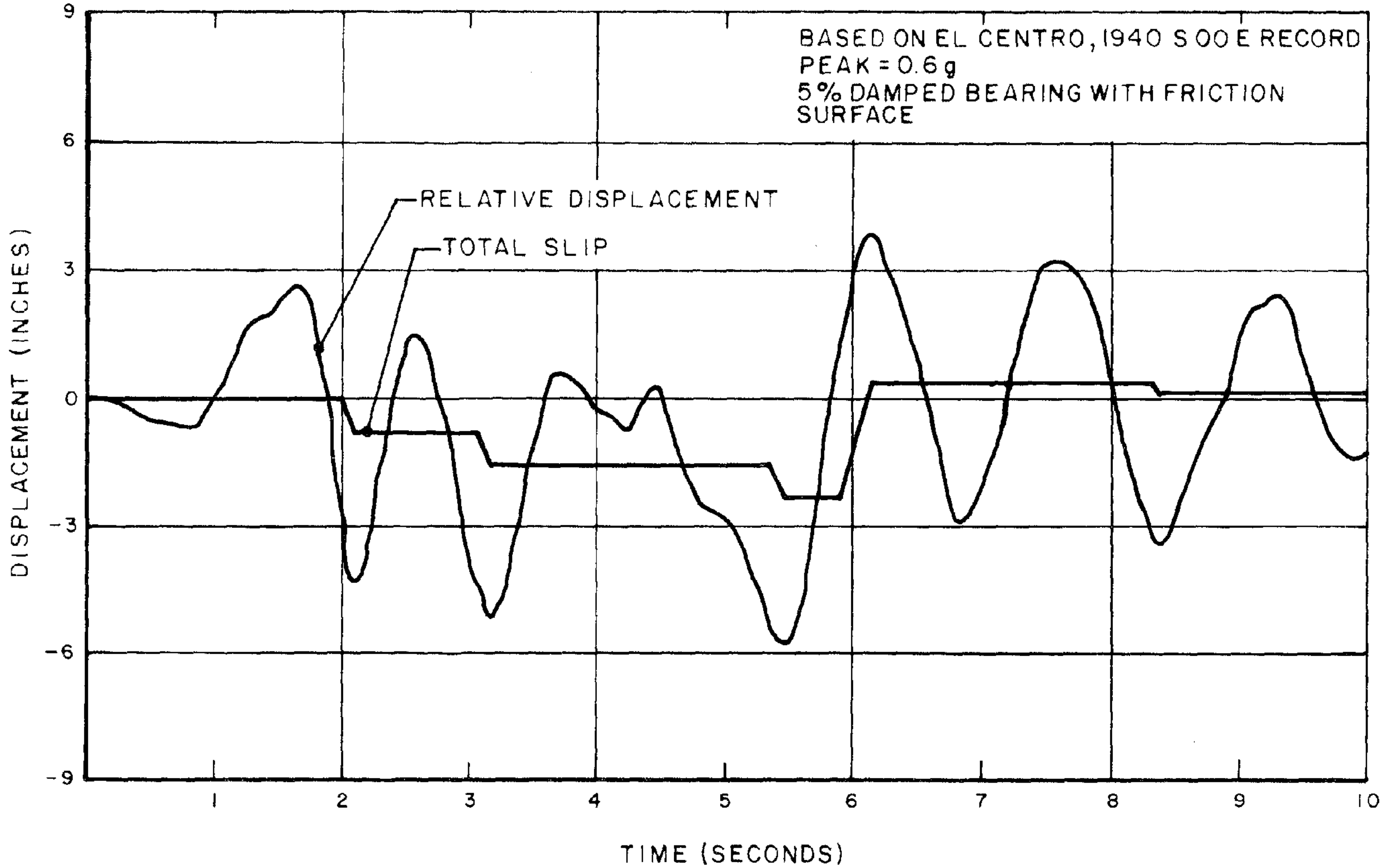
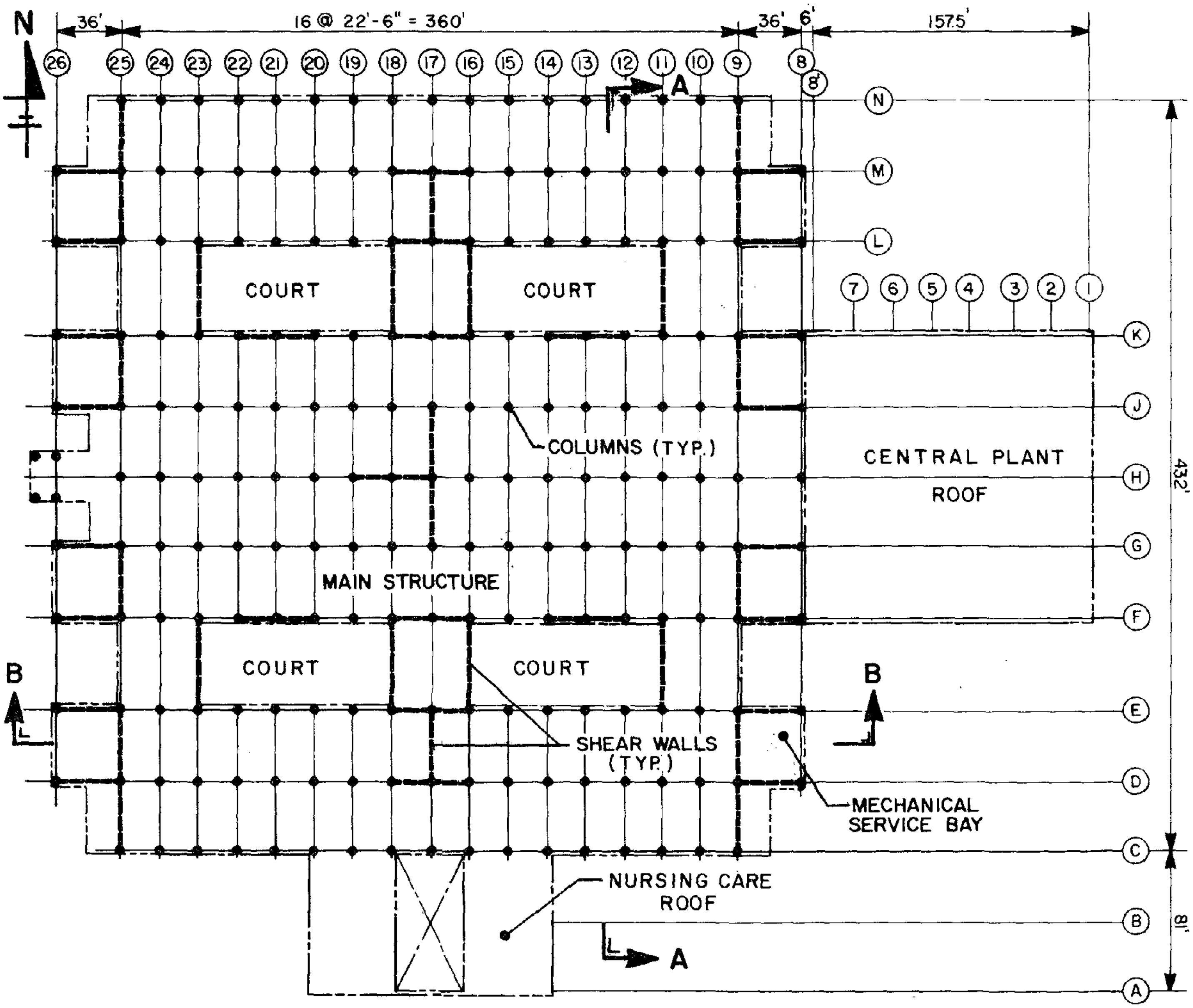


FIGURE 21 DISPLACEMENT TIME HISTORY
CASE STUDY II



NOTE:

SEE FIGURE 23 FOR SECTIONS A-A & B-B

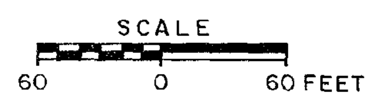
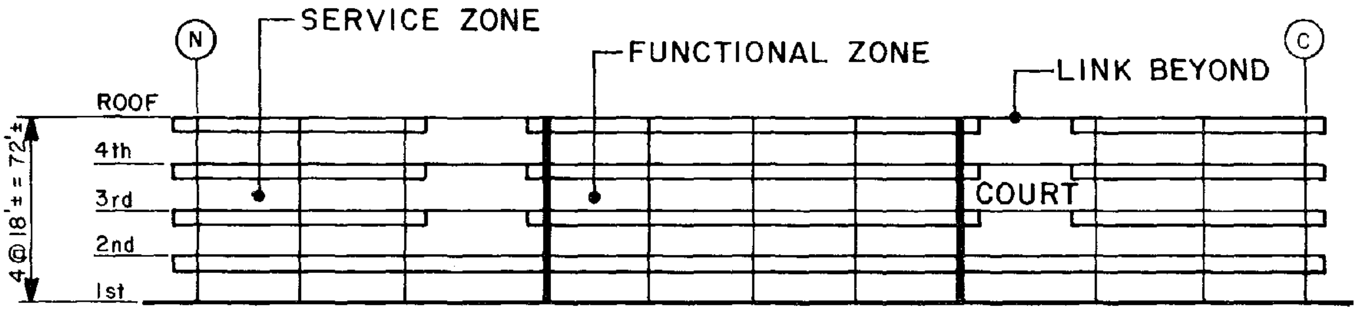
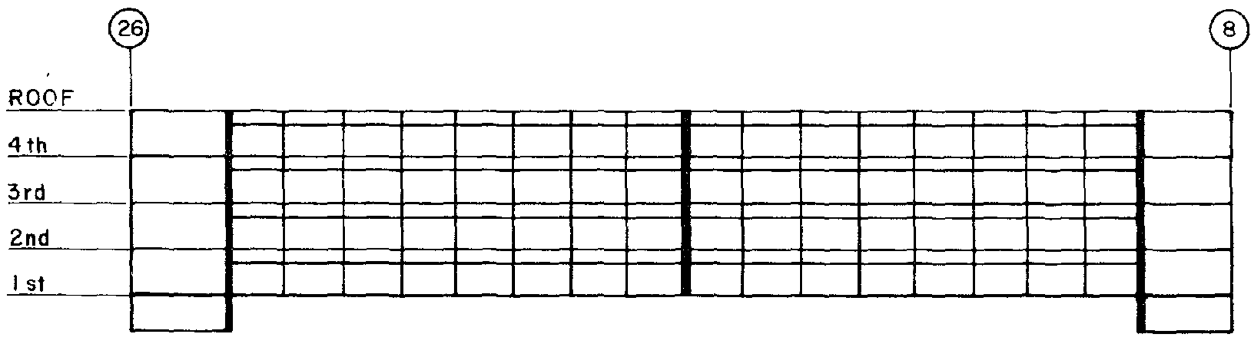


FIGURE 22 FLOOR PLAN, CASE STUDY III
LOMA LINDA VAH

F-22



SECTION A-A



SECTION B-B

FIGURE 23 TYPICAL SECTIONS, CASE STUDY III
LOMA LINDA VAH

F-23

F-24

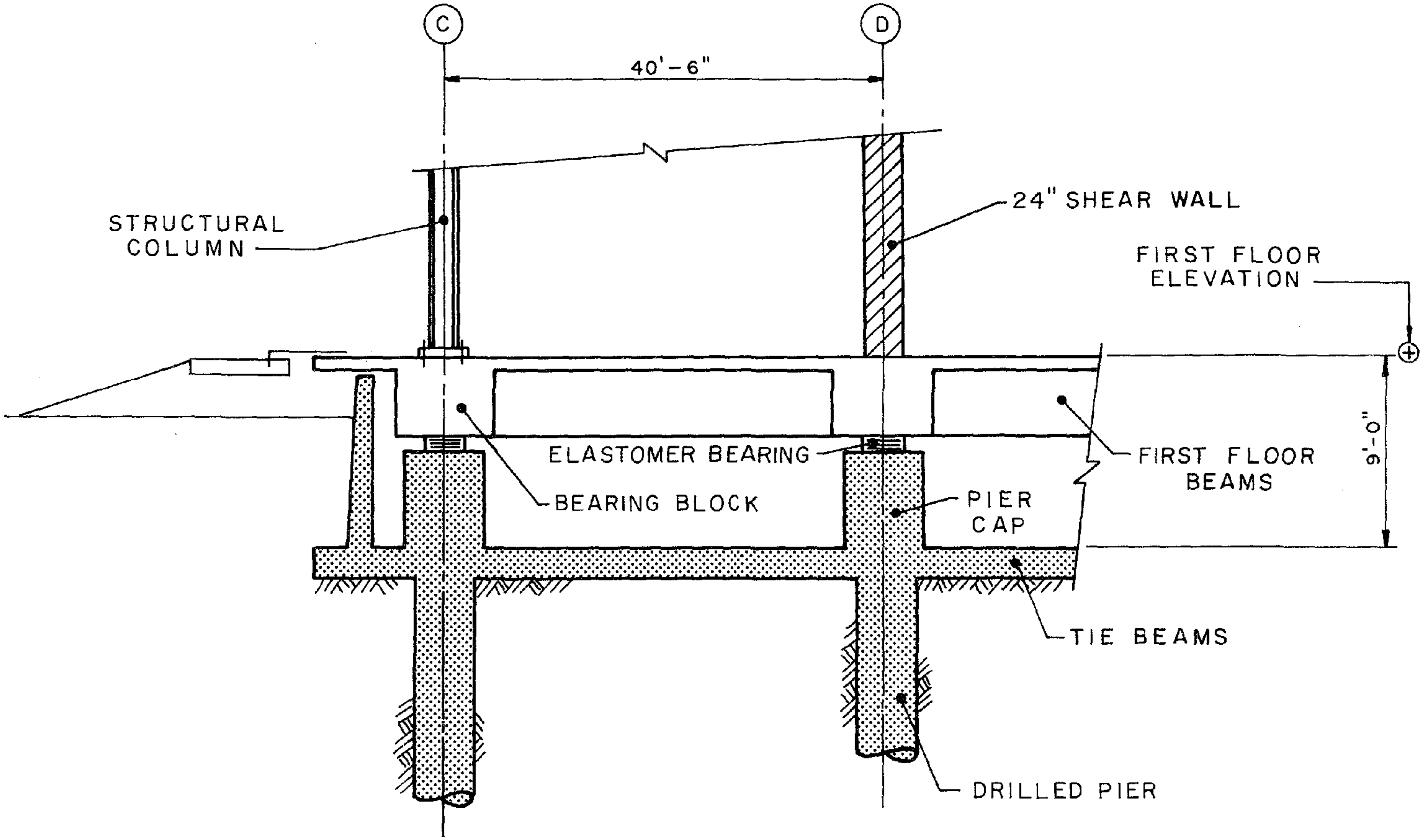
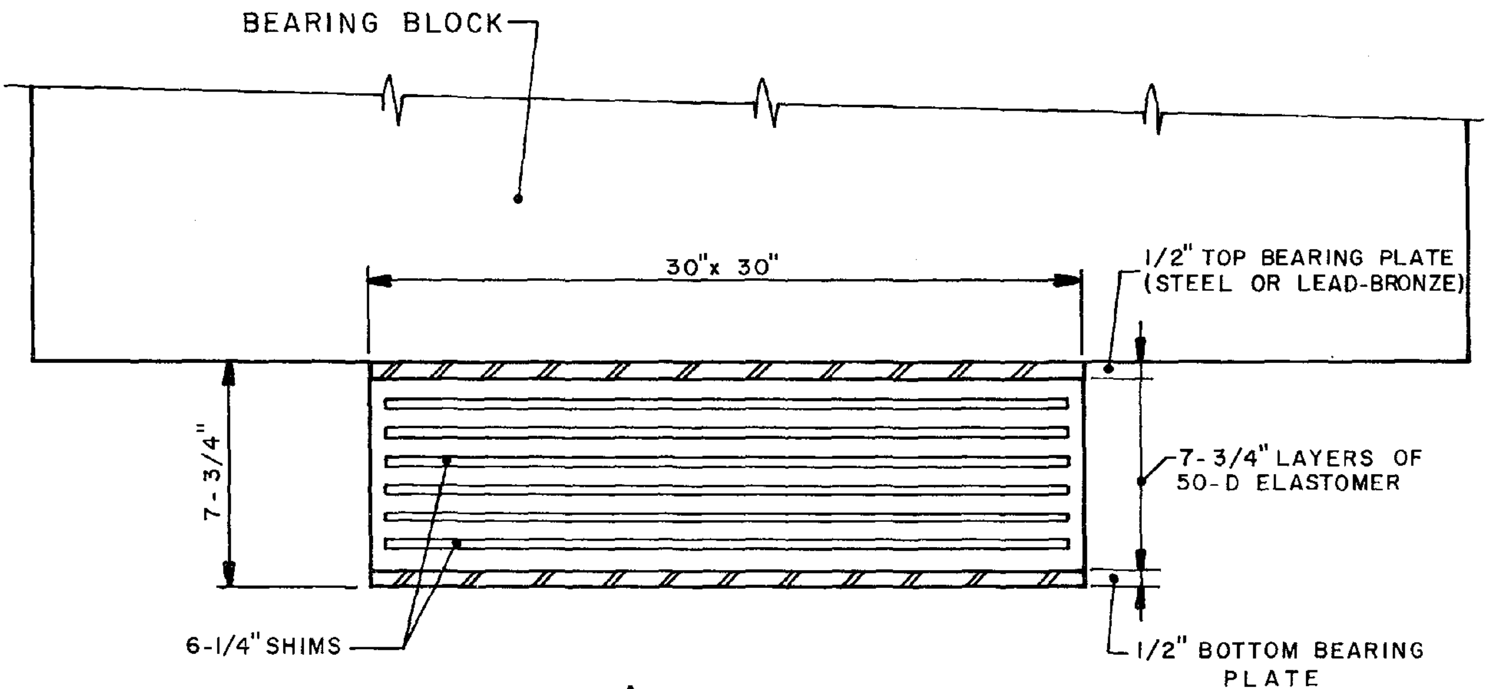


FIGURE 24 CONCEPTUAL DESIGN OF BASE ISOLATION, CASE STUDY III



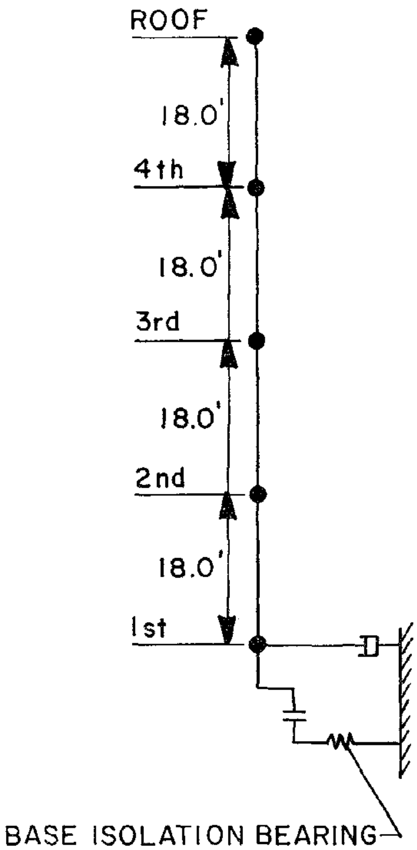
SHAPE FACTOR, $S = \frac{A}{A_s} = 10$

HORIZONTAL STIFFNESS, $K = \frac{AG}{T} = 25.7$ KIPS/IN.

WHERE,

A = LOADED AREA
A_s = AREA FREE TO BULGE
G = SHEAR MODULUS OR ELASTOMER
T = TOTAL ELASTOMER THICKNESS

FIGURE 25 TYPICAL BEARING DETAIL, CASE STUDY III



LEVEL	WEIGHT (KIPS)	MASS ($K\text{-}SEC^2/Ft.$)
ROOF	21,332	662.5
4th	25,296	785.6
3rd	26,488	822.6
2nd	33,788	1049.3
1st	25,300	705.7
TOTAL	132,204	4105.7

STORY	HORIZONTAL STIFFNESS (K/Ft)
4	1.19×10^7
3	1.36×10^8
2	1.56×10^8
1	1.81×10^8
BEARINGS	83,688

LEGEND

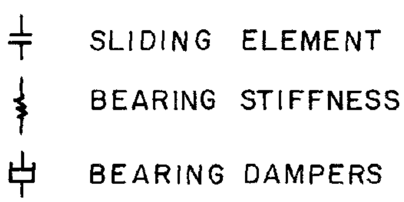
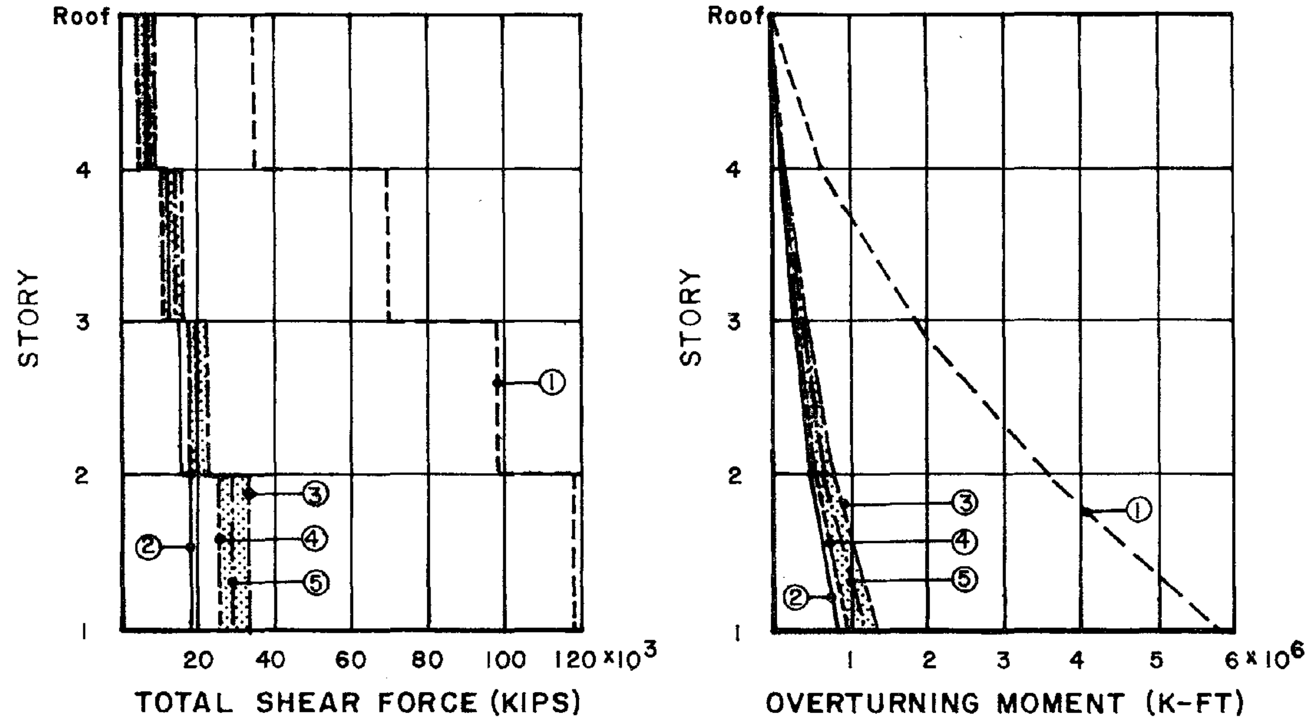


FIGURE 26 MATHEMATICAL MODEL
CASE STUDY III

F-26



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

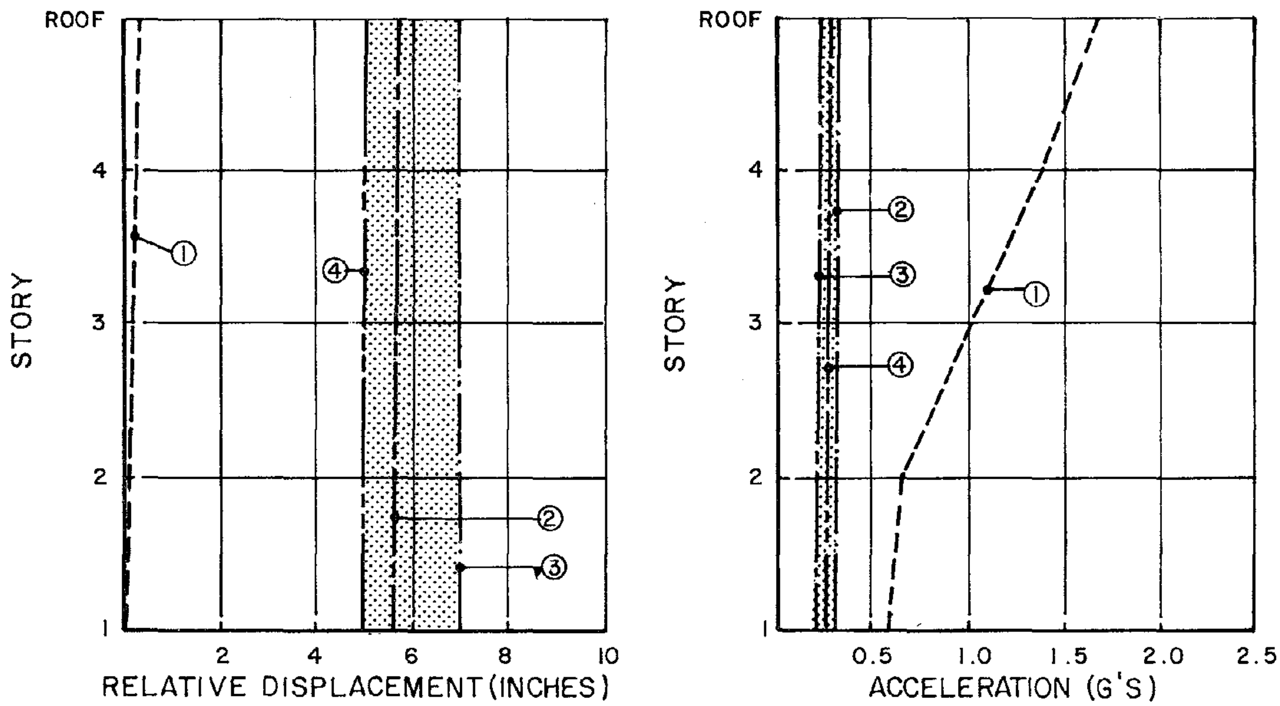
⑤ 15% DAMPED BEARING

---

(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.

FIGURE 27 STORY SHEARS AND OVERTURNING MOMENTS CASE STUDY III



LEGEND

CONVENTIONAL BUILDING

① LINEAR ELASTIC ANALYSIS (1)

BASE ISOLATED BUILDING

② 5% DAMPED BEARING

③ 5% DAMPED WITH FRICTIONAL INTERFACE

④ 15% DAMPED BEARING

---

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

FIGURE 28 DISPLACEMENTS AND ACCELERATIONS
CASE STUDY III

F-28

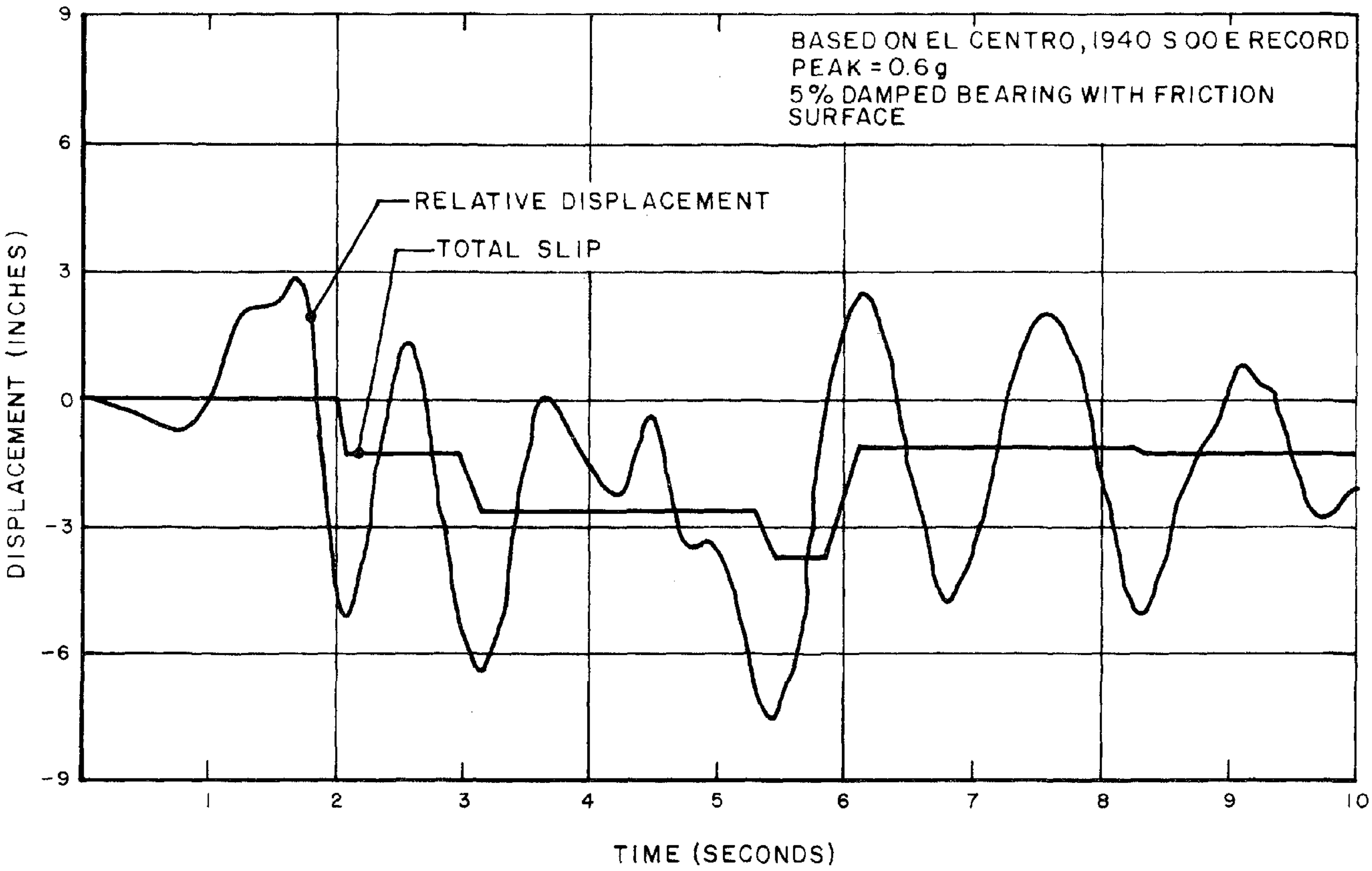


FIGURE 29 DISPLACEMENT TIME HISTORY
CASE STUDY III

F-29

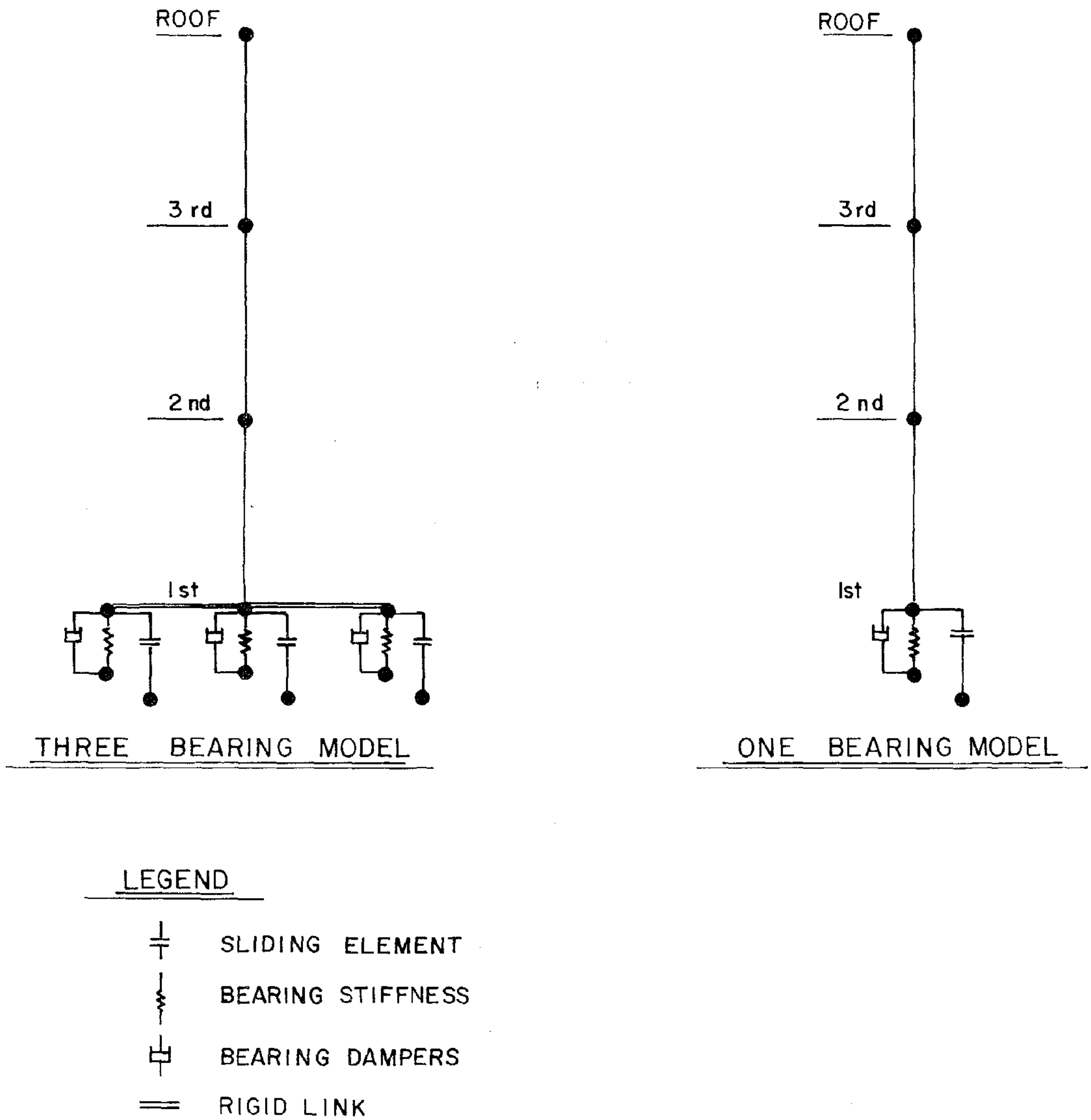
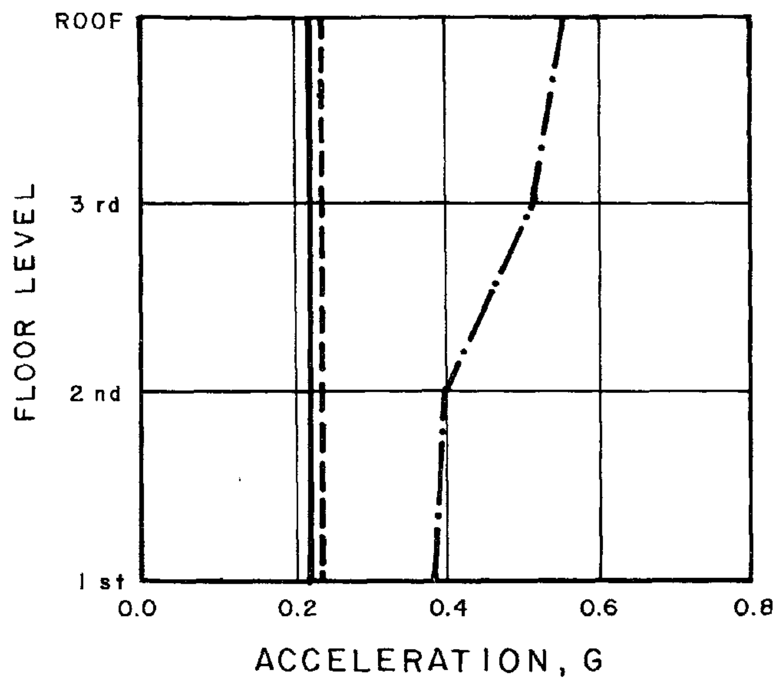


FIGURE 30 MATHEMATICAL MODEL INCLUDING OVERTURNING EFFECTS
CASE STUDY I

F-30



LEGEND

——— 3 BEARING MODEL (NONLINEAR)

– – – – 1 BEARING MODEL (NONLINEAR)

—·— CONVENTIONAL BUILDING — LINEAR ELASTIC ANALYSIS

NOTE:

BASED ON EL CENTRO 1940 S 00 E RECORD SCALED TO 0.6g.

FIGURE 31 DISTRIBUTION OF ACCELERATION INCLUDING OVERTURNING EFFECTS CASE STUDY I

F 31

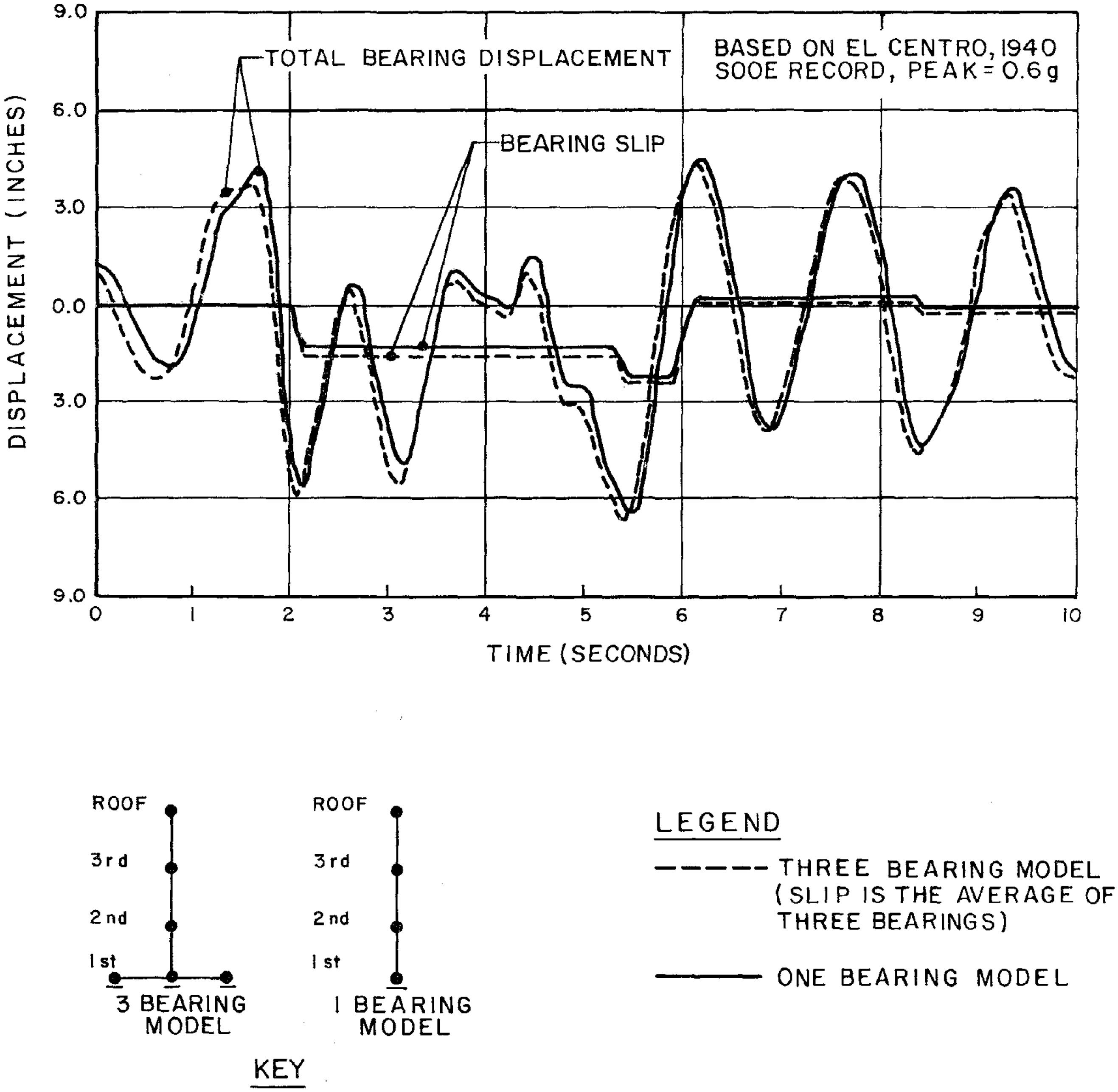


FIGURE 32 BEARING DISPLACEMENTS INCLUDING OVERTURNING EFFECTS CASE STUDY I

F-32

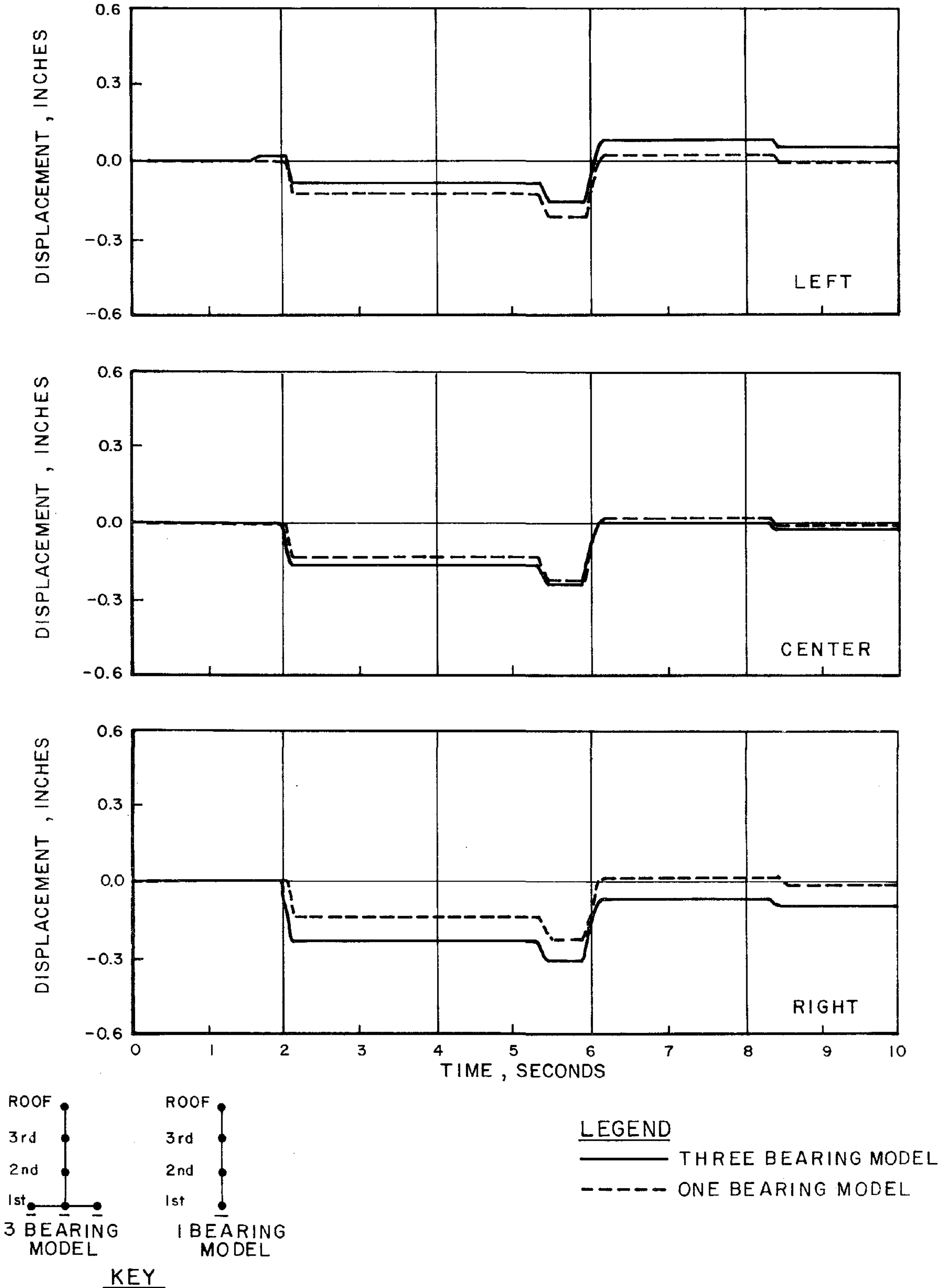
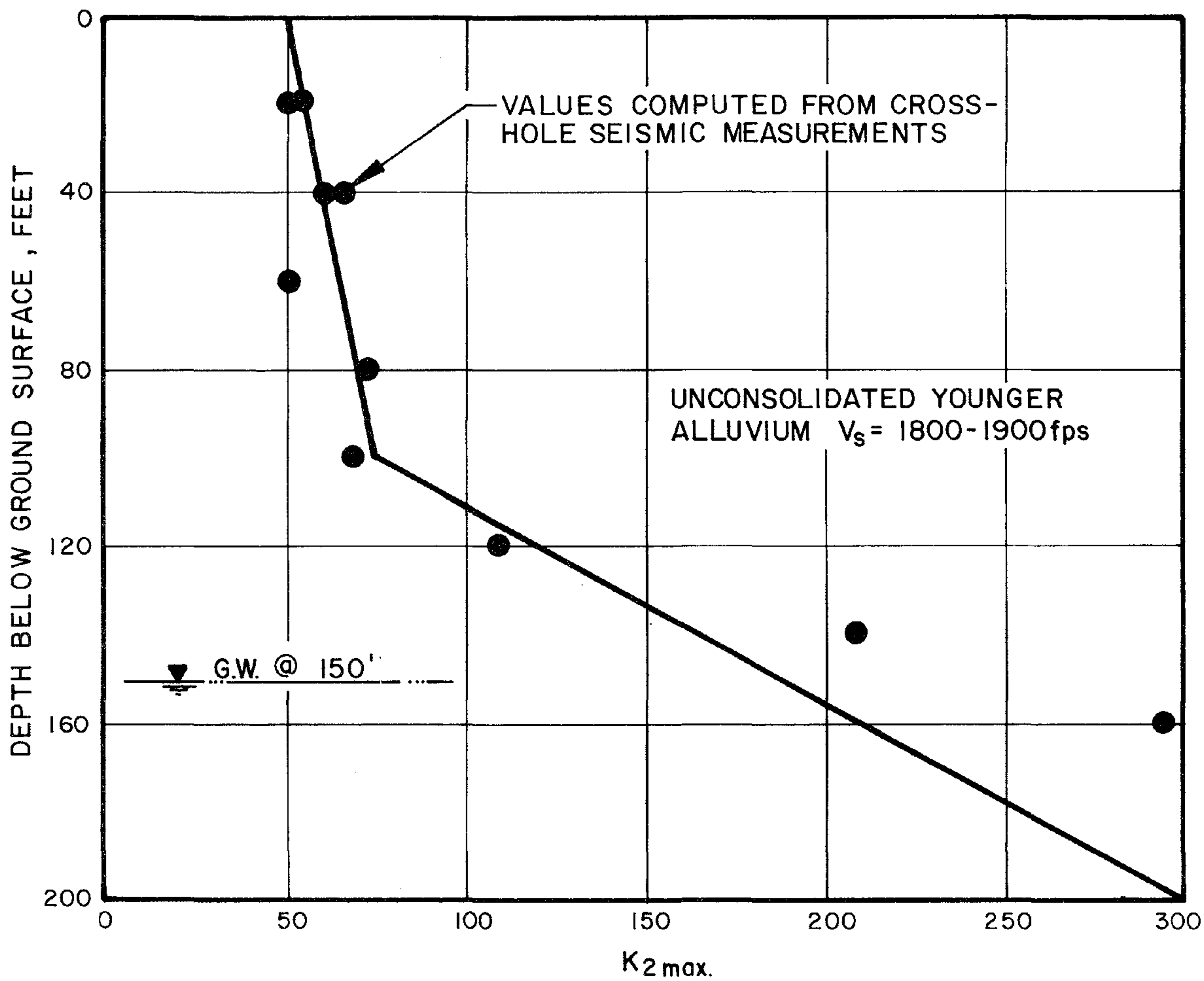


FIGURE 33 BEARING SLIP INCLUDING OVERTURNING EFFECT CASE STUDY I

F-33



NOTE:

$G_{max} = 1000\, K_{2max} \times \sigma_m^{1/2}$ psf

FIGURE 34 SHEAR MODULUS PROFILE
LOMA LINDA VAH

F-34

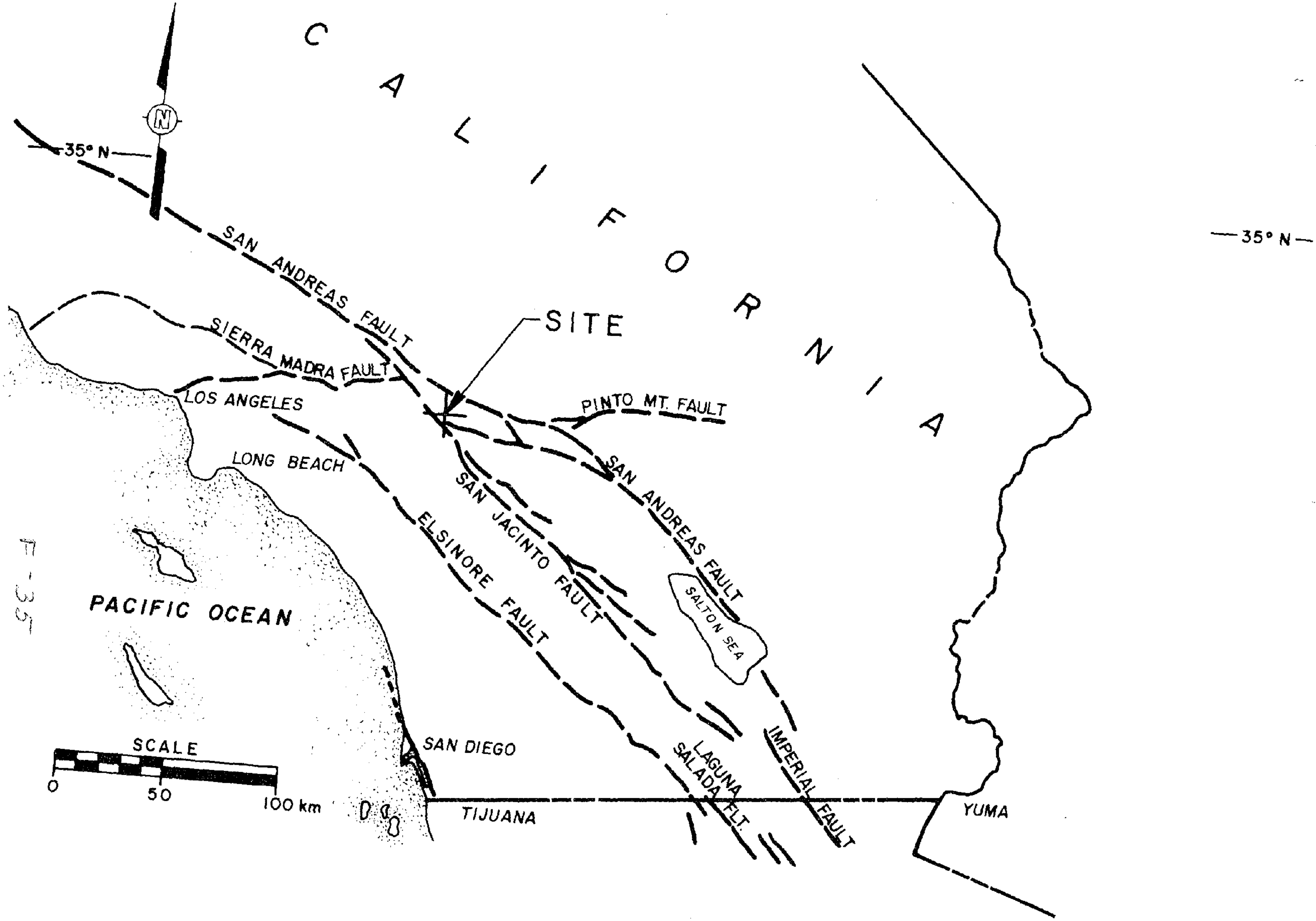
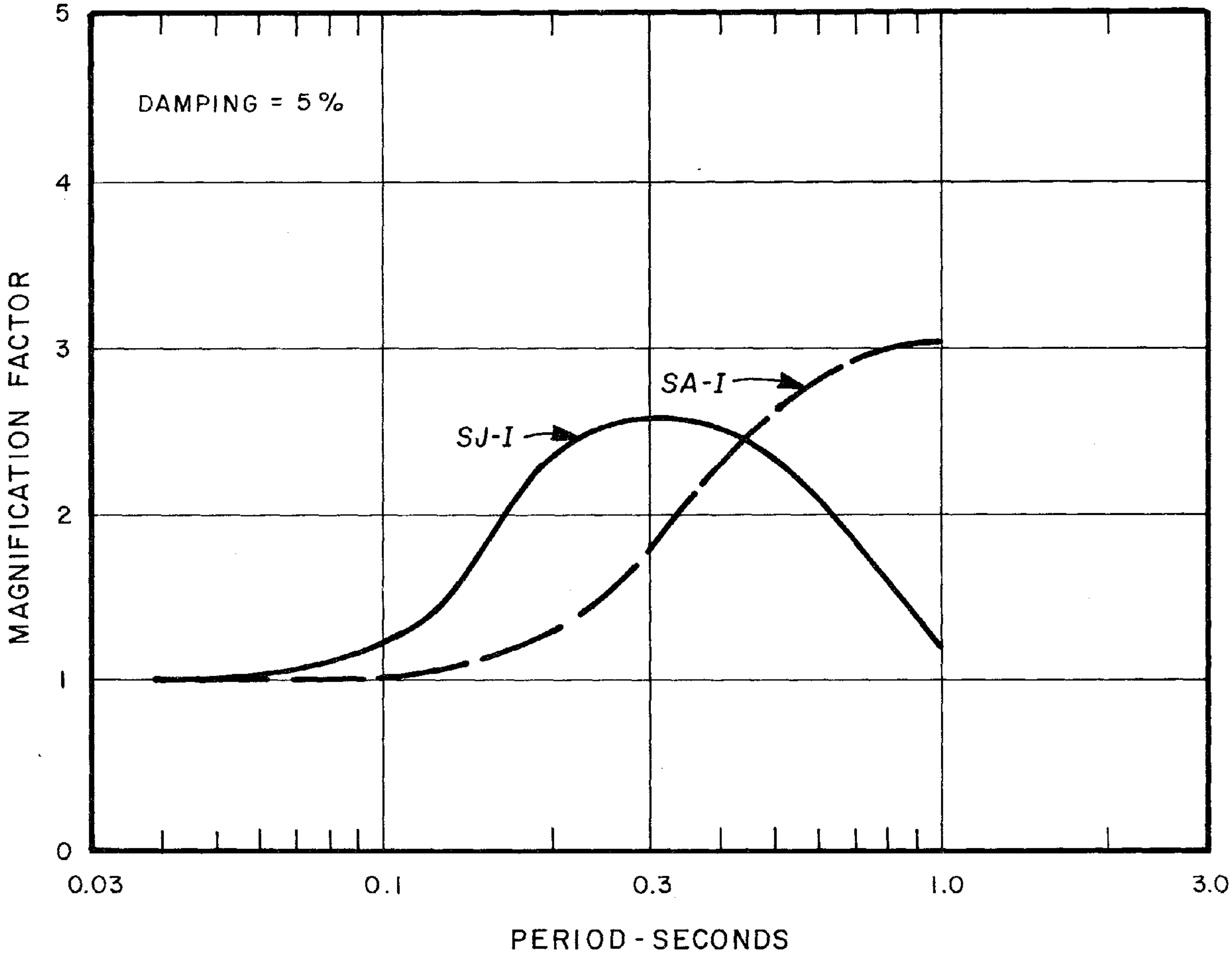


FIGURE 35 SITE LOCATION
LOMA LINDA VAH

F-35



LEGEND:

SA-1 SAN ANDREAS EVENT
SA-2 SAN JACINTO EVENT

FIGURE 36 SMOOTHED GROUND RESPONSE SPECTRA LOMA LINDA VAH

F-36

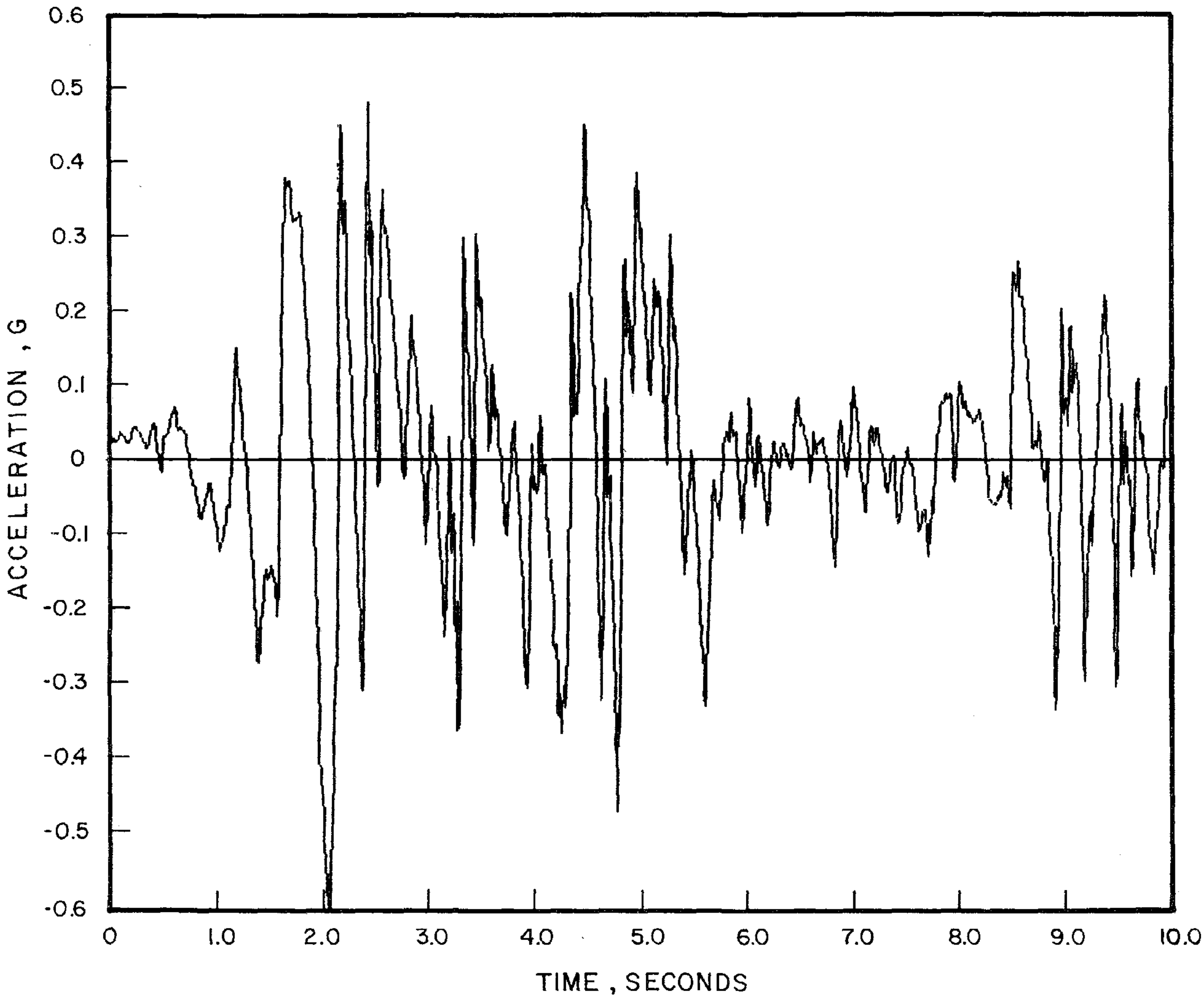
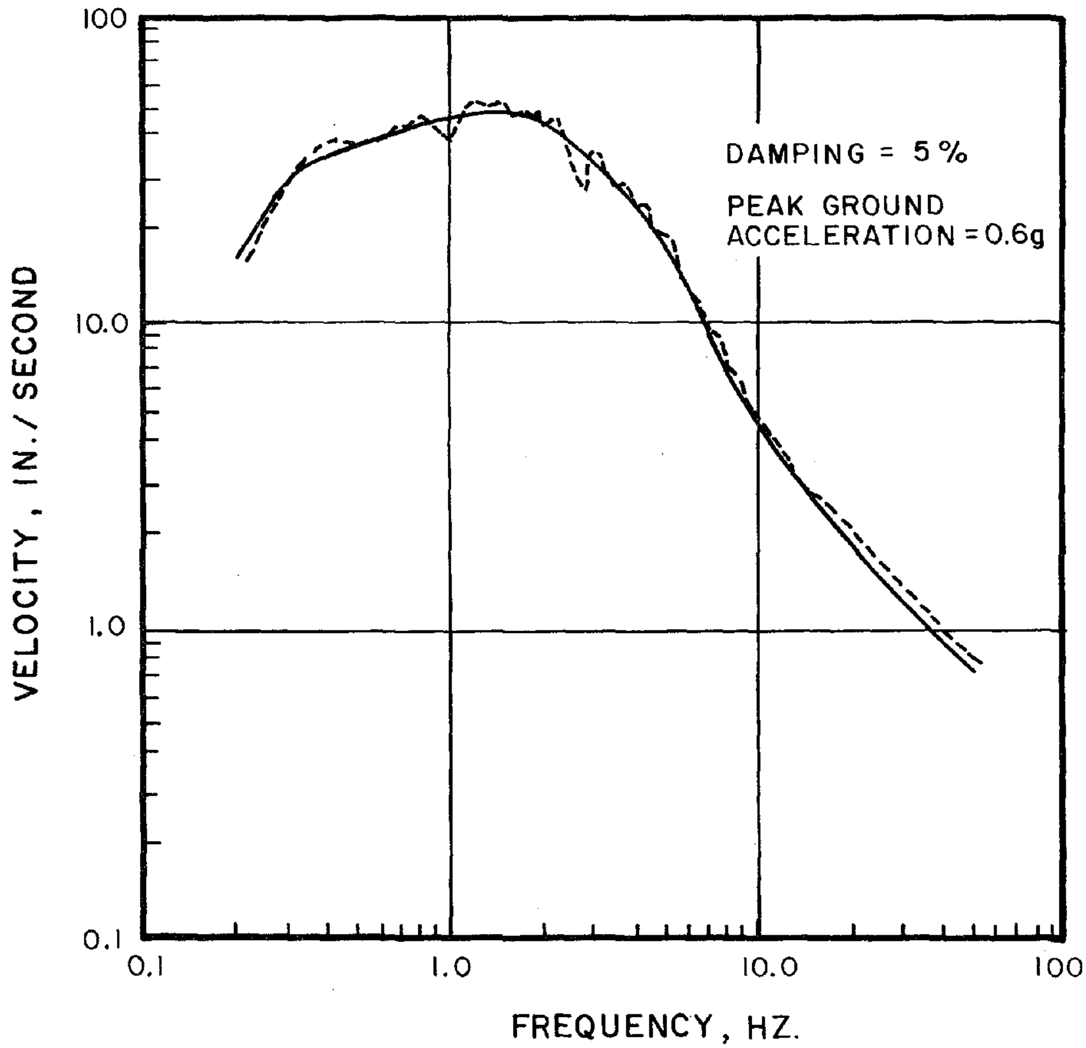


FIGURE 37 SYNTHETIC GROUND MOTION TIME HISTORY
MATCHING SJ-I SPECTRUM
LOMA LINDA VAH

F-37



LEGEND

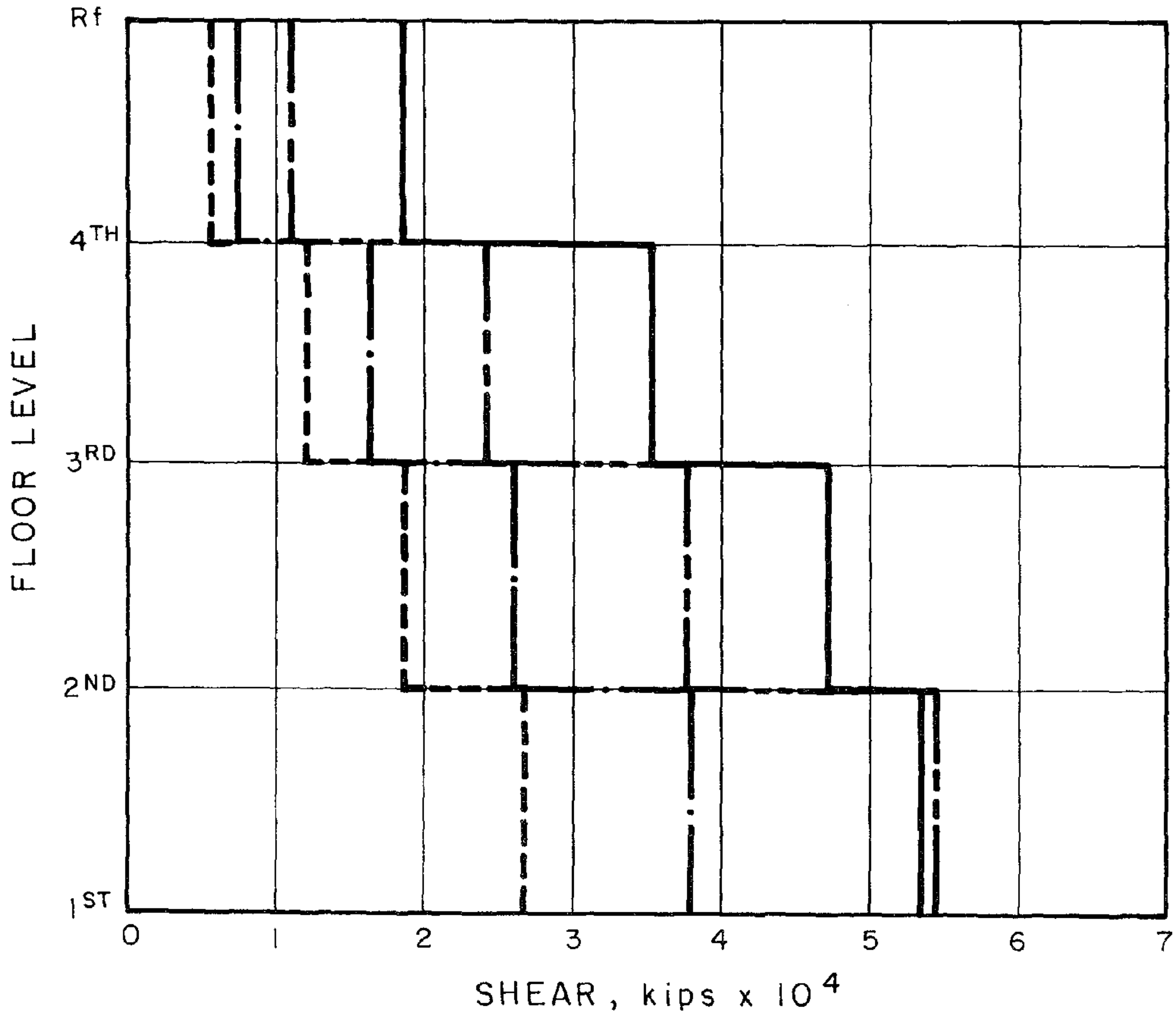
———— DESIGN RESPONSE SPECTRUM

– – – – COMPUTED RESPONSE SPECTRUM

NOTE:

SJ–1 GROUND MOTION
PEAK GROUND ACCELERATION = 0.6 g

FIGURE 38 COMPUTED VERSUS DESIGN RESPONSE SPECTRA
LOMA LINDA VAH



LEGEND

- ———— CONVENTIONAL FOUNDATION, BASE SHEAR = 0.5W
- —·–·– ASEISMIC BEARING (DAMPING = 5%)
- ——·—— ASEISMIC BEARING (DAMPING = 15%)
- – – – – ASEISMIC BEARING WITH SLIP SURFACE (DAMPING = 5%)

NOTE:

SJ-1 GROUND MOTION
PEAK GROUND ACCELERATION = 0.6g

FIGURE 39 SHEAR DISTRIBUTION OVER HEIGHT OF BUILDING, LOMA LINDA VAH

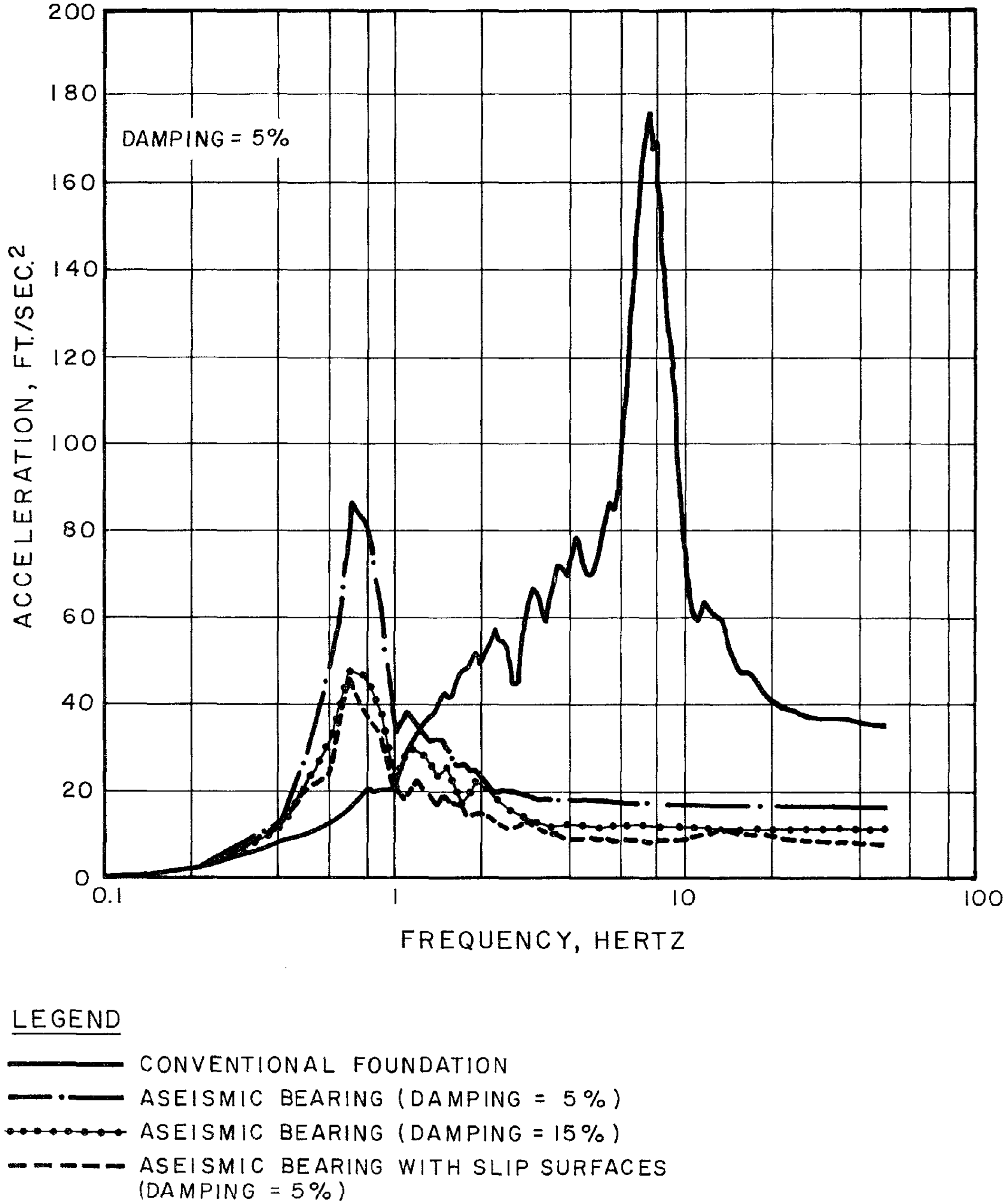
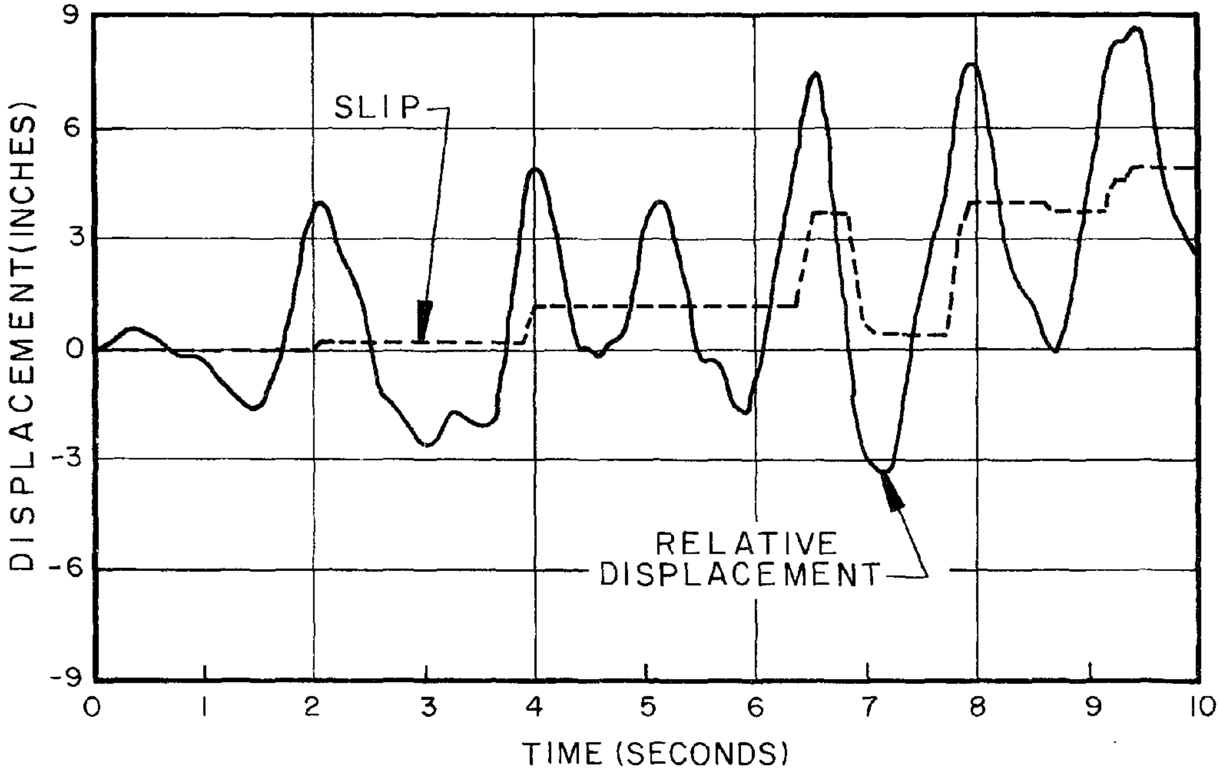


FIGURE 42 FLOOR RESPONSE SPECTRUM AT ROOF ELEVATION, LOMA LINDA VAH



TIME (SECONDS)

NOTE:

SJ—1 GROUND MOTION
PEAK GROUND ACCELERATION = 0.6 g

FIGURE 43 RELATIVE DISPLACEMENT AND SLIP TIME HISTORIES, LOMA LINDA VAH

P-43

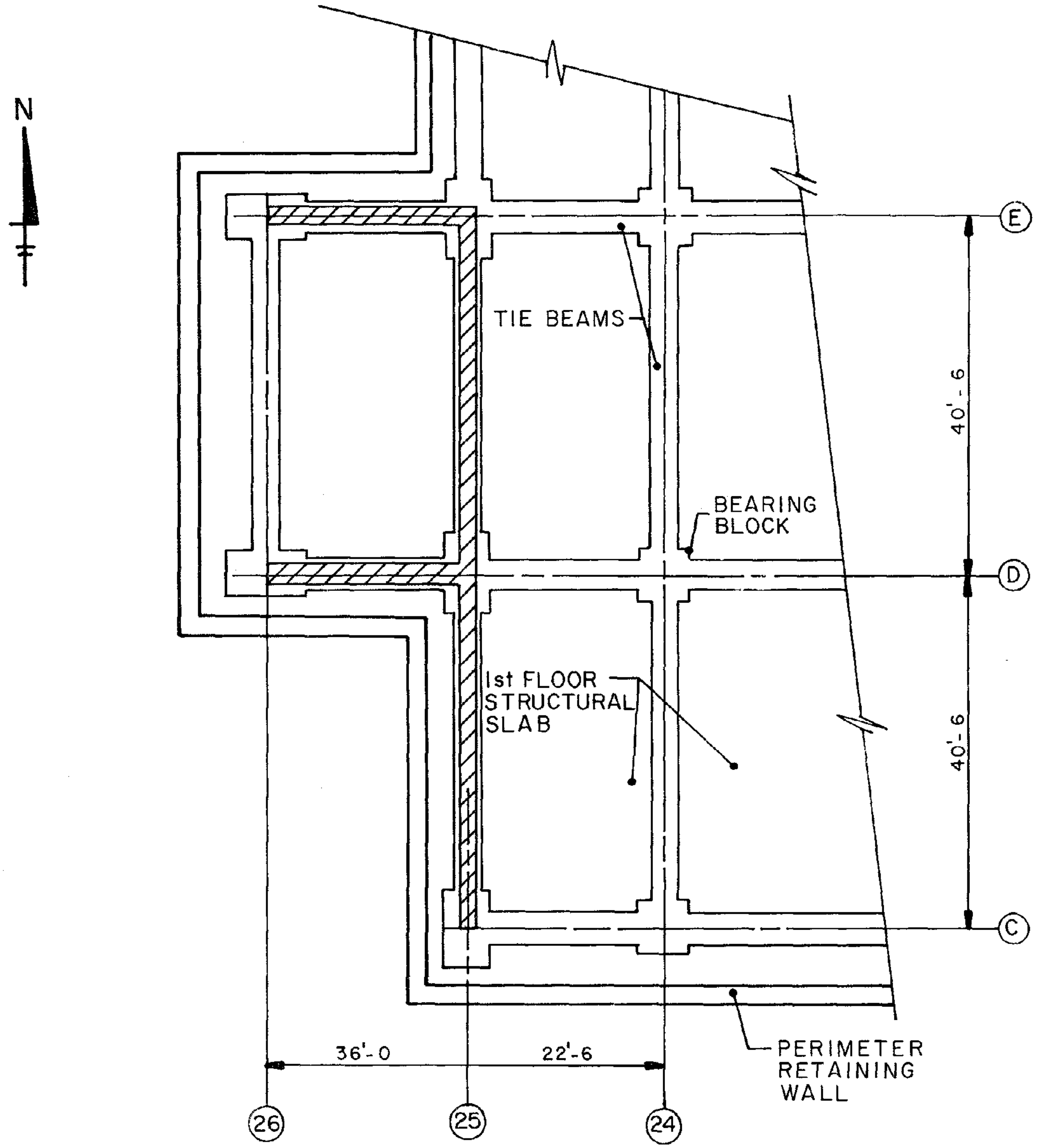


FIGURE 44 PARTIAL FOUNDATION PLAN-BASE
ISOLATION CONCEPT
LOMA LINDA VAH

F-44

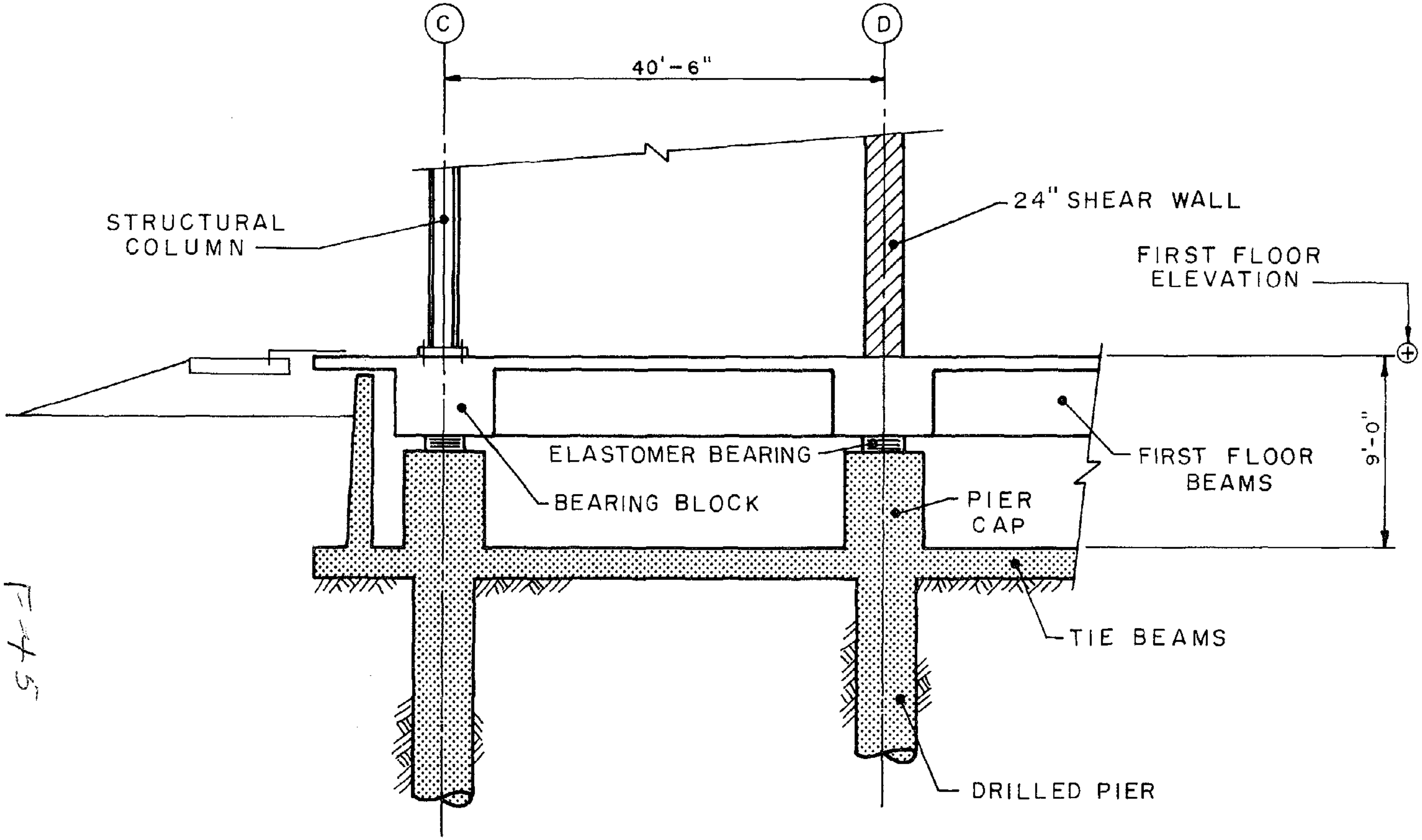
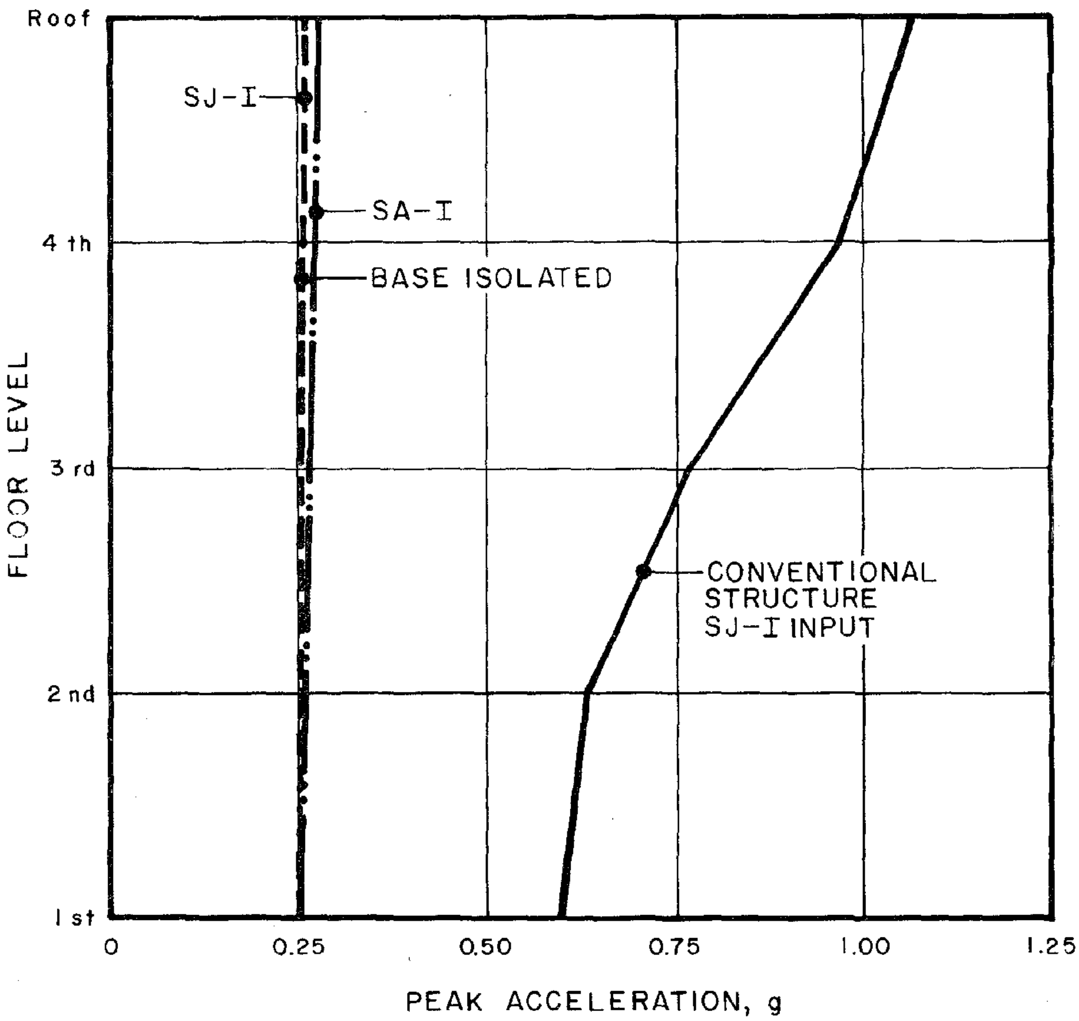


FIGURE 45 NORTH-SOUTH SECTION - BASE ISOLATION CONCEPT, LOMA LINDA VAH

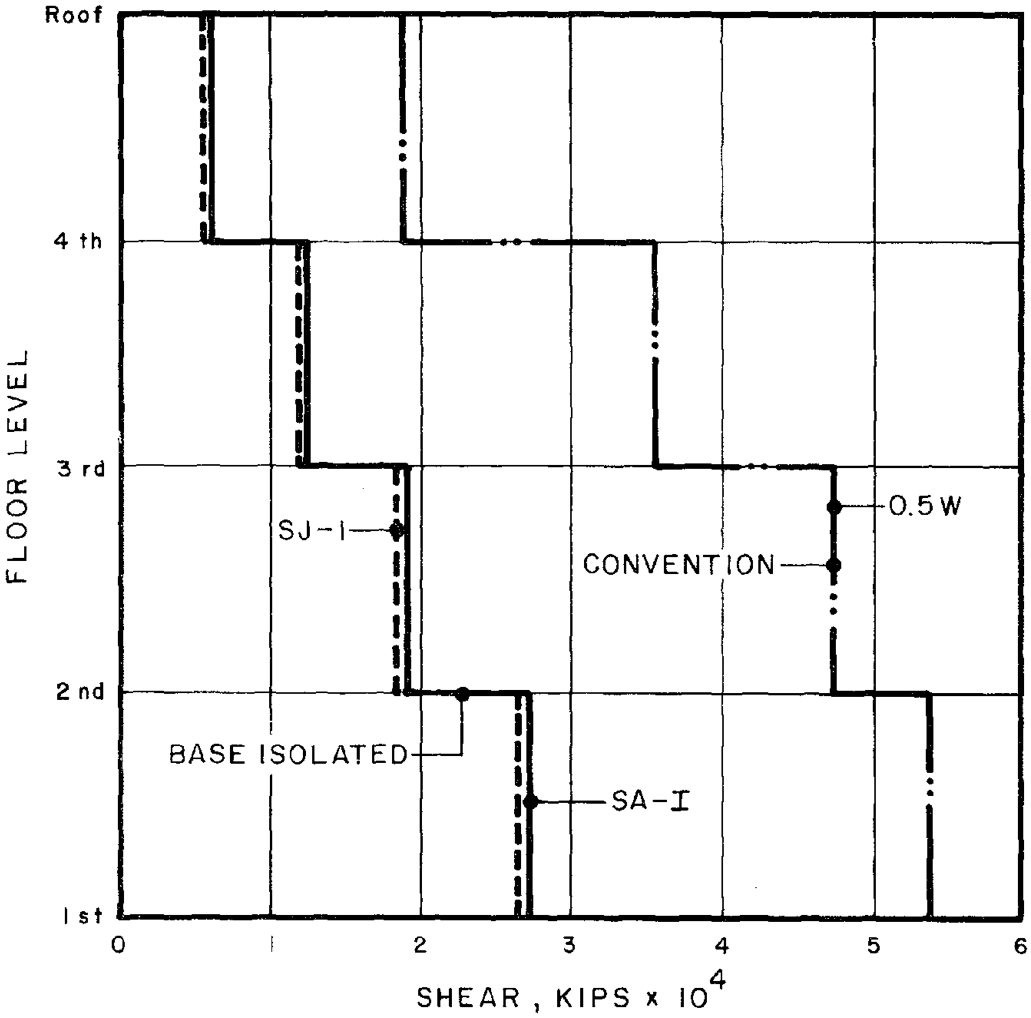
F-45



NOTE:

BASE ISOLATION USES BEARING WITH FRICTION SURFACE; PEAK GROUND ACCELERATION = 0.6g

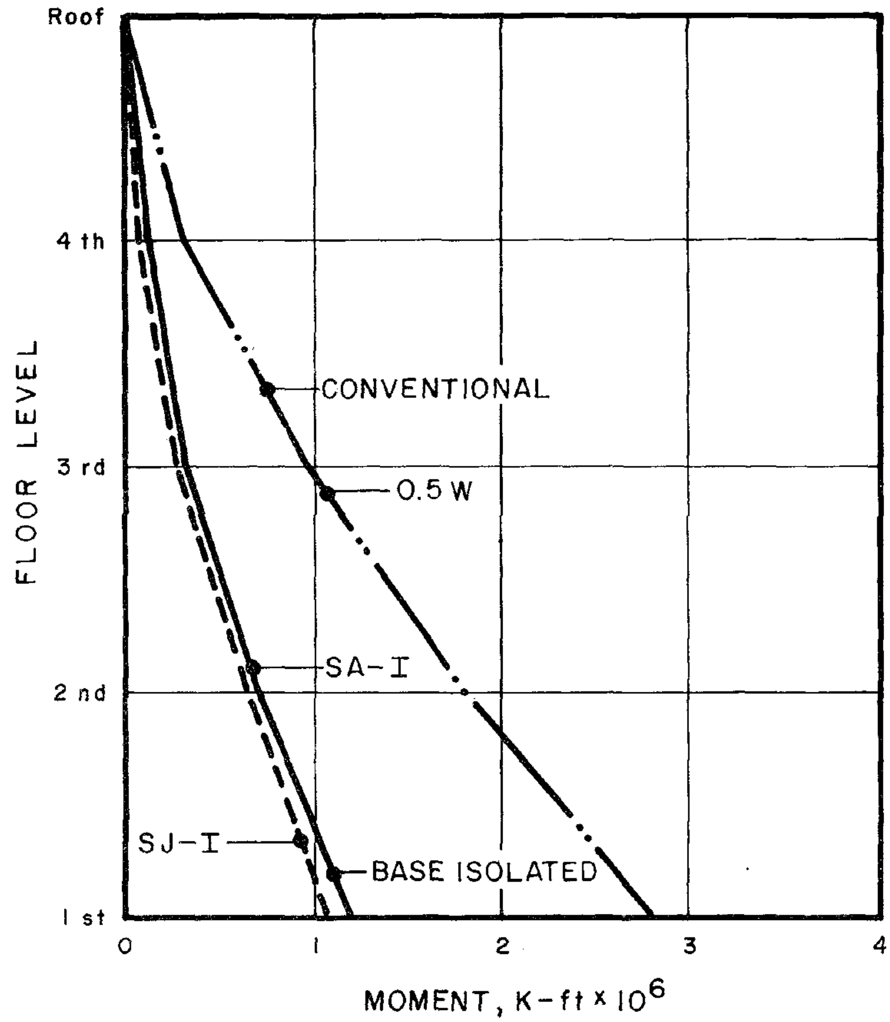
FIGURE 48 FLOOR ACCELERATIONS FOR SA-I INPUT LOMA LINDA VAH



NOTE:

BASE ISOLATION USES BEARING WITH FRICTION SURFACE; PEAK GROUND ACCELERATION = 0.6g

FIGURE 49 STORY SHEARS FOR SA-I INPUT LOMA LINDA VAH

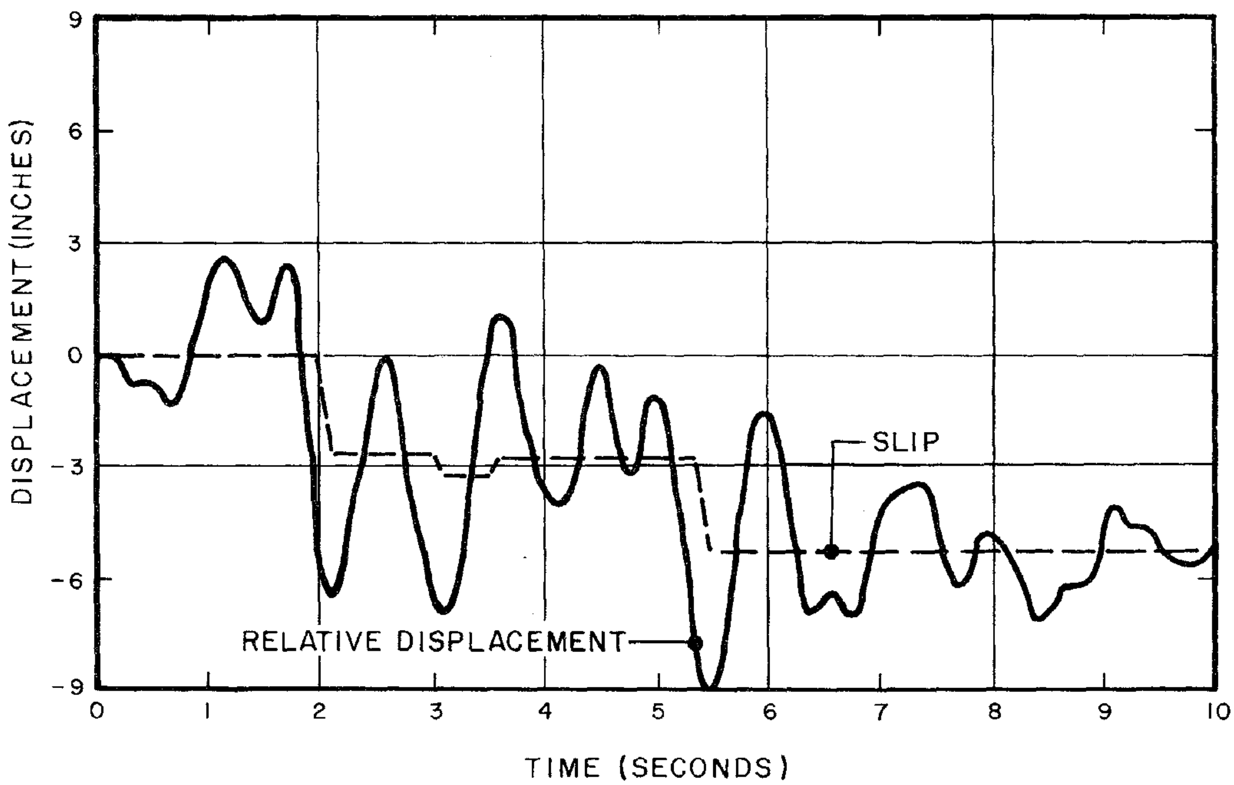


NOTE:

BASE ISOLATION USES BEARING WITH FRICTION SURFACE; PEAK GROUND ACCELERATION = 0.6 g

FIGURE 50 OVERTURNING MOMENTS FOR SA-I INPUT LOMA LINDA VAH

F-50



NOTE:

SA–I INPUT
PEAK GROUND ACCELERATION = 0.6 g

FIGURE 51 RELATIVE DISPLACEMENTS, SA-I INPUT LOMA LINDA VAH

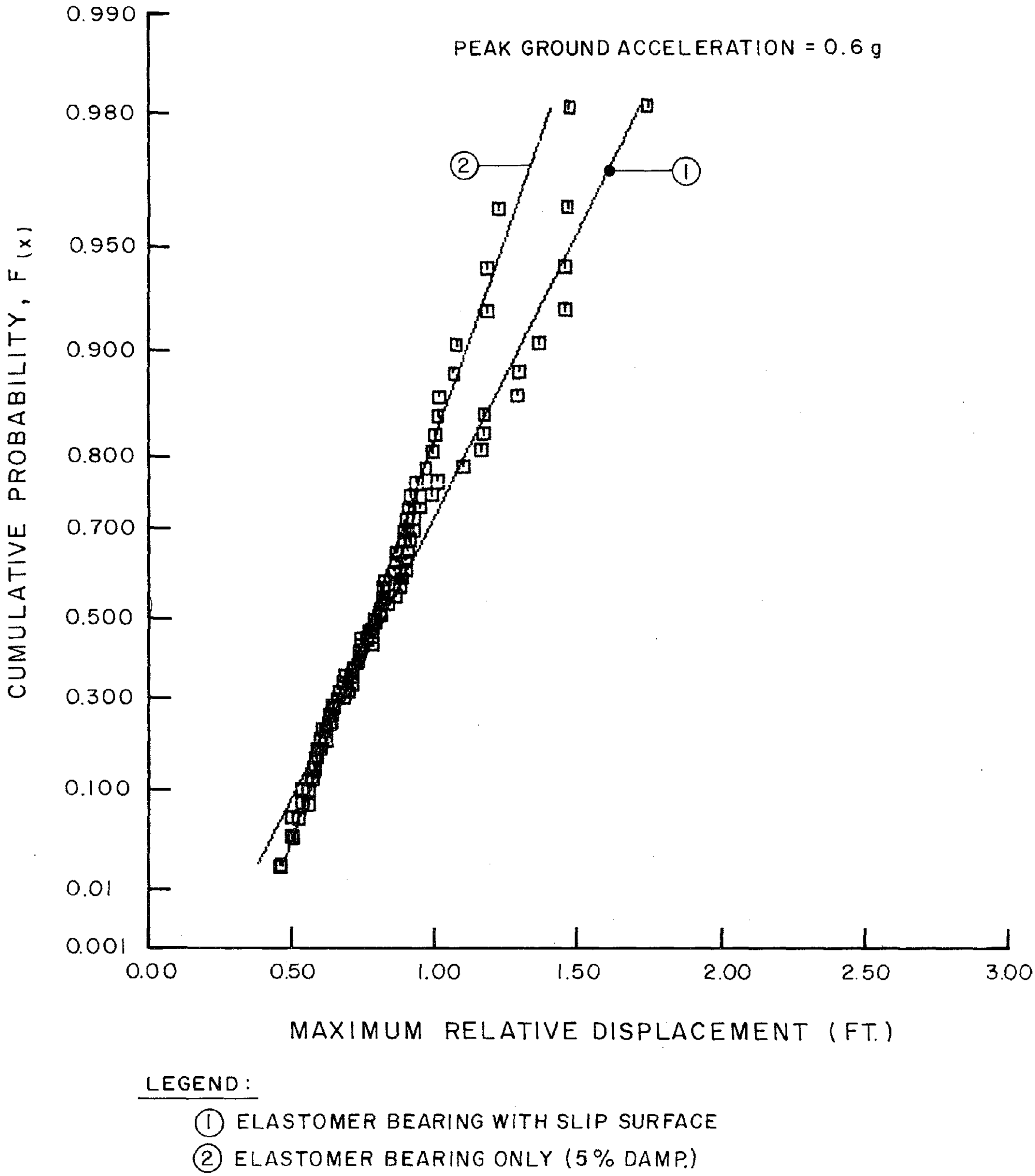
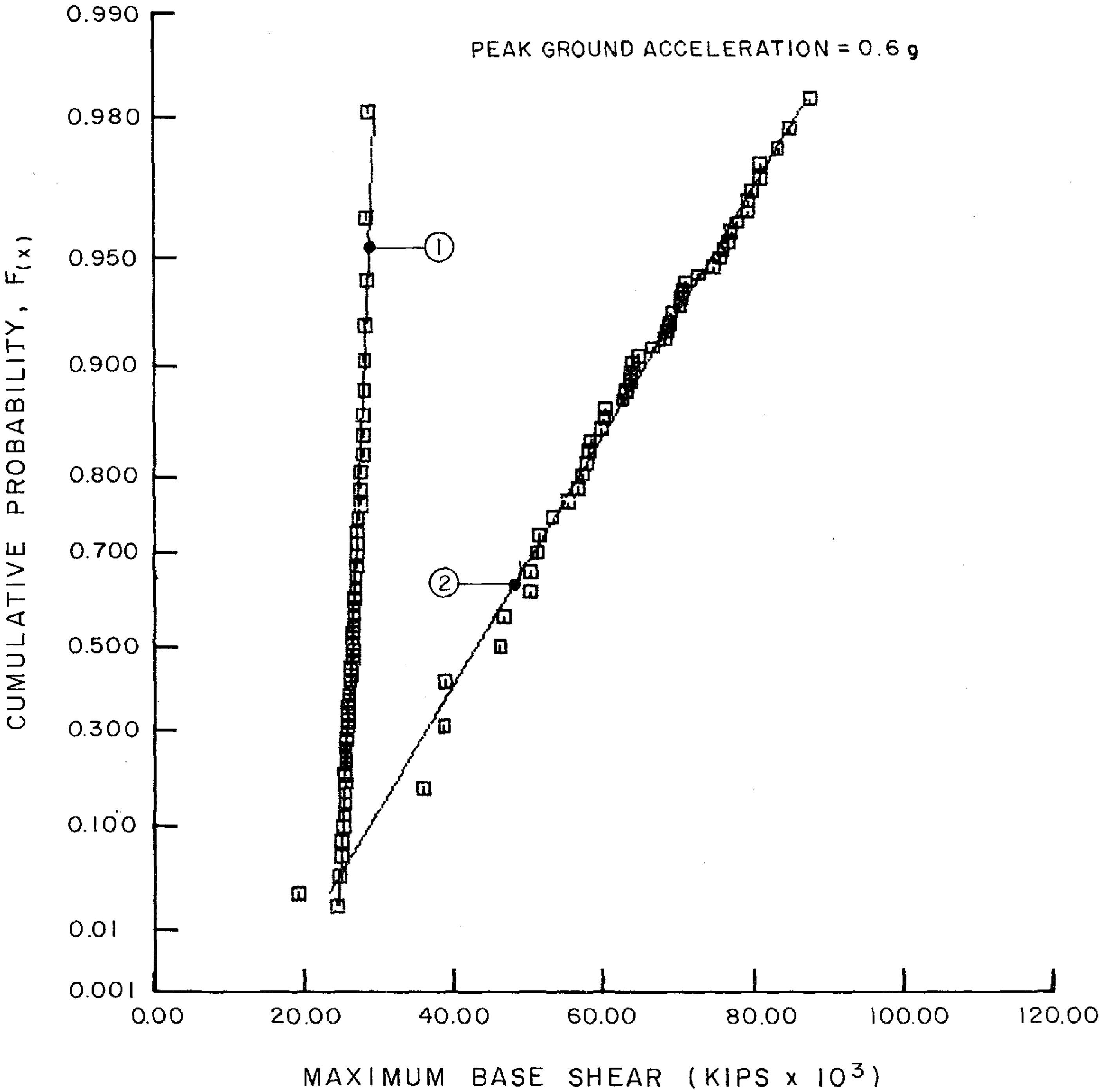


FIGURE 56 PROBABILITY DISTRIBUTION OF BASE ISOLATED RESPONSE, MAXIMUM RELATIVE DISPLACEMENT



LEGEND:

1. ELASTOMER BEARING WITH SLIP SURFACE
2. ELASTOMER BEARING ONLY (5% DAMP.)

FIGURE 57 PROBABILITY DISTRIBUTION OF BASE ISOLATED RESPONSE, MAXIMUM BASE SHEAR F-57

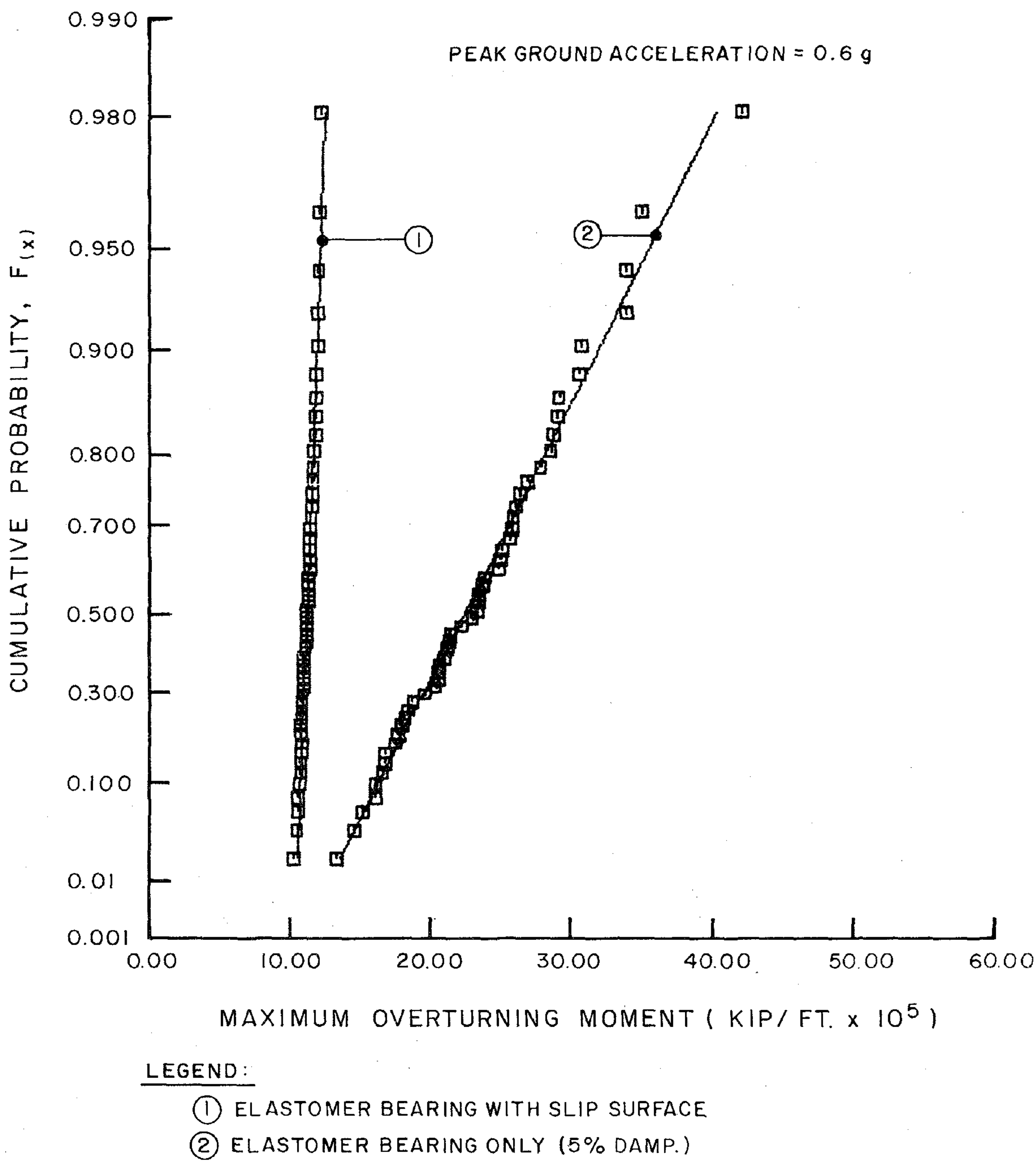


FIGURE 58 PROBABILITY DISTRIBUTION OF BASE ISOLATED RESPONSE, OVERTURNING MOMENT