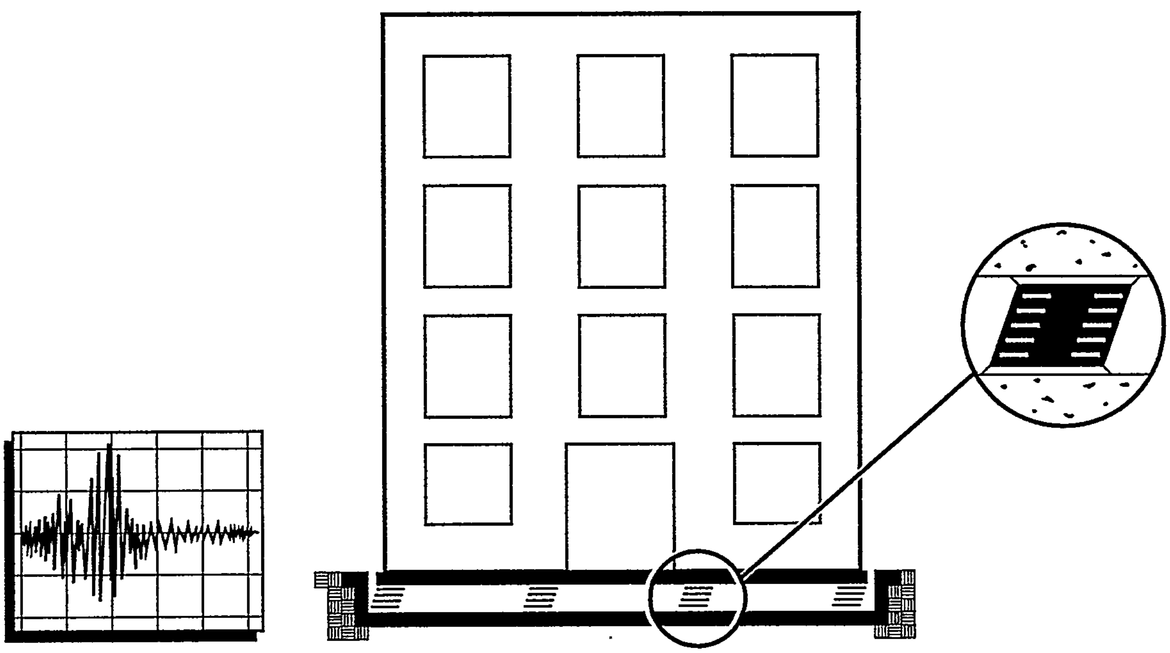
CONF-9205413

Technology Transfer Package on Seismic Base Isolation

Volume II



Hosted by:

Lawrence Livermore National Laboratory

Sponsored by:
U.S. Department of Energy

Offices of Risk Analysis and Technology and Nuclear Safety Policy and Standards

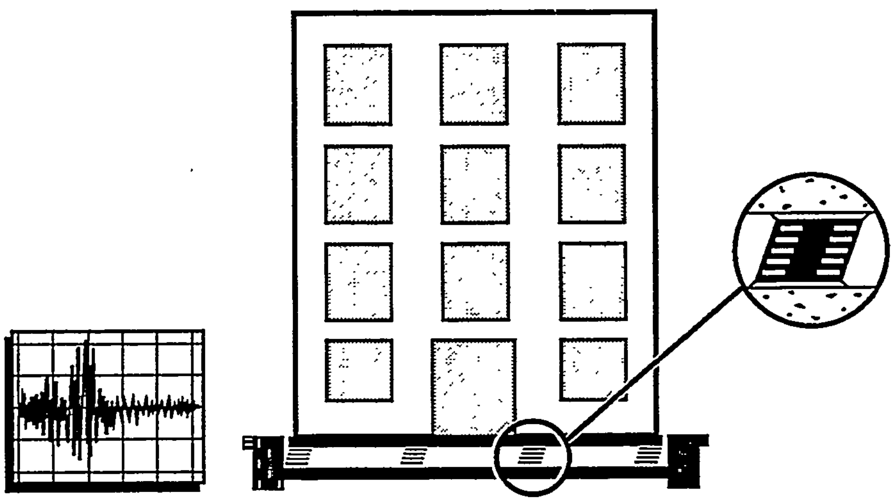
Work performed under the auspices of the U.S. Department of Energy by Lawrence Livermore National Laboratory under Contract W-7405-Eng-48.

DISCLAIMER

Table of Contents

Department of Energy

Short Course on Seismic Base Isolation



August 10–14, 1992
Berkeley Marina Marriott
Berkeley, California

Hosted by:

Lawrence
Livermore
National
Laboratory

Sponsored by:
U.S. Department of Energy

Offices of
Risk Analysis and
Technology
and Nuclear Safety
Policy and Standards

Berkeley Short Course

Berkeley Short Course

DEPARTMENT OF ENERGY SHORT COURSE ON SEISMIC BASE ISOLATION

AGENDA

Day 1 - Monday, August 10, 1992

7:30 am–8:30 am	Registration and Distribution of Materials	Lilian Decman Dawn Matz
8:30 am–9:00 am	Welcome	Stanley Sommer James Hill V. Gopinath and Richard Stark
9:00 am–10:00 am	The Development and Implementation of Base Isolation in the United States and Abroad	James Kelly
10:00 am–10:30 am	***Break***	
10:30 am–11:30 am	Base Isolation in Japan	James Kelly
11:30 am–12:00 pm	Applications of Isolation Systems in the U.S.	Charles Kircher
12:00 pm–1:15 pm	***Lunch***	
1:15 pm–2:30 pm	Linear Theory of Base Isolation	James Kelly
2:30 pm–3:00 pm	***Break***	
3:00 pm–4:30 pm	Extension of Theory to Buildings	James Kelly

Day 2 - Tuesday, August 11, 1992

8:00 am–8:45 am	Seismic Design Guidance for DOE Facilities	Thomas Nelson
8:45 am–10:00 am	Selection of Isolation as a Design Strategy	Charles Kircher
10:00am–10:30 am	***Break***	
10:30 am–11:30 am	Design Considerations for Isolated Structures	Charles Kircher
11:30 am–12:00 pm	Overview of Seismic Code Provisions	Charles Kircher
12:00 pm–1:15 pm	***Lunch***	
1:15 pm–3:00 pm	Current Building Code Provisions - SEAOC/UBC	Charles Kircher
3:00 pm–3:30 pm	***Break***	
3:30 pm–4:30 pm	Design Process for Isolated Structures	Charles Kircher
6:00 pm	**Reception**	

Day 3 - Wednesday, August 12, 1992

8:00am–9:15 am	Design Process for Multilayer Elastomeric Bearings	James Kelly
9:15am—10:15 am	Experimental Work on Base Isolation	James Kelly
10:15am–10:45 am	***Break***	
10:45 am–12:00 pm	Failure Mechanisms in Isolators	James Kelly
12:00 pm–1:30 pm	***Lunch***	
1:30pm–4:00 pm	Tour of Earthquake Engineering Research Center (EERC) Examples of Tests at EERC	James Kelly James Kelly

Day 4 - Thursday, August 13, 1992

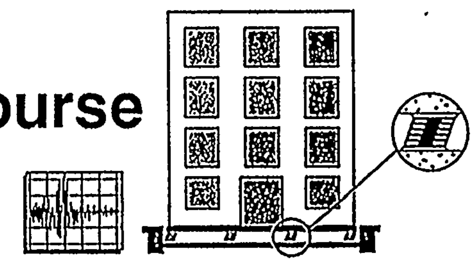
8:00 am–9:15 am	Seismic Isolation Decision Methodology and Application to the L.A. County FCCF	Robert Bachman
9:15 am–9:45 am	***Break***	
9:45 am–11:15 am	Applying UBC Code Design Requirements to the L.A. County FCCF	Robert Bachman
11:15 am–12:00 pm	Description of Student Design Problem	James Kelly
12:00 pm–1:15 pm	***Lunch***	
1:15 pm–5:00 pm	SEAOC/UBC Portion of Student Design Problem *3DBASIS* Portion of Student Design Problem	Ian Aiken and Peter Clark Ian Aiken and Peter Clark

Day 5 - Friday, August 14, 1992

8:00 am–9:15 am	Students Discuss Design Problem	James Kelly
9:15 am–9:45 am	***Break***	
9:45 am–10:15 am	Future Directions in the DOE	Stanley Sommer
10:15 am–11:30 am	Panel Discussion	James Hill James Kelly Charles Kircher

NSSP
Nuclear Systems Safety Program

Department of Energy Short Course on Seismic Base Isolation



WELCOME

Stanley C. Sommer
Lawrence Livermore National Laboratory
Nuclear Systems Safety Program

August 10, 1992
Berkeley Marina Marriott
Berkeley, California

LLNL/DOE Welcomes you to this Short Course.

- **Short Course Sponsors**

  U.S. Department of Energy
  Office of Risk Analysis and Technology
  Office of Nuclear Safety Policy and Standards

- **Short Course Host**

  Lawrence Livermore National Laboratory
  Nuclear Systems Safety Program

This Short Course will Instruct you on Engineering Details of Seismic Base Isolation.

- Overview of Base Isolation and its Many Applications
- Theory which Provides Technical Background

  Linear theory of MDOF systems
  Failure mechanisms in isolators

- Applying Theory to Practical Design with the UBC

  Seismic code (SEAOC) provisions
  Design process

- Student Design Problems

  Tour Earthquake Engineering Research Center (EERC)
  Static design with SEAOC provisions
  Nonlinear time history analysis with 3DBASIS

The Short Course Instructors are from Government, Universities, and Industry.

- Instructors

Robert E. Bachman - Fluor Daniel, Inc.
James M. Kelly - University of California Berkeley
Charles A. Kircher - Charles Kircher & Associates
Thomas A. Nelson - Lawrence Livermore National Lab.

- Graduate Students at EERC

Ian Aiken - University of California Berkeley
Peter Clark - University of California Berkeley

Several People are Responsible for Organizing the Short Course.

- Lawrence Livermore National Laboratory

  Stanley C. Sommer, Project Leader
  Robert C. Murray, Associate Program Leader
  Lilian S. Decman, Administrator
  Dawn S. Matz, Administrative Assistant

- U.S. Department of Energy

  James R. Hill, Project Manager
  V. Gopinath, Project Manager

This Short Course is a Follow-up to a Workshop held in May in Marina Del Rey, California.

- Workshop on Seismic Base Isolation for Department of Energy Facilities

- Participation from all Sectors of Technical Community

  10 instructors
  55 participants

- Successful and Useful for the DOE

  Discussed general concepts about base isolation
  Toured several base isolated facilities in Los Angeles

The DOE is Focusing on Reducing the Effects of Natural Phenomena Hazards on its Facilities.

DOE Supports the International Decade for Natural Disaster Reduction

- Natural Phenomena Hazards

  Earthquakes (Seismic)
  Wind
  Flood
  Volcano
  Lightning
  Others

Many DOE Facilities are Located in Areas of High Seismic Risk.

- Areas of High Seismic Risk

  California: LLNL, SLAC, LBL, SNL, ETEC
  Eastern U.S.: ORNL, Paducah, Savannah River

- DOE Facilities

  Non-critical: office building, storage building
  Critical: research laboratory, reactor

- Potential Uses of Base Isolation

  Seismic retrofit of existing facility
  Construction of new facility

LECTURE 1

The Development and Implementation of Base Isolation In the United States and Abroad

by

James M. Kelly
Professor of Civil Engineering
Earthquake Engineering Research Center
University of California
Berkeley, California 94720

Short Course on Seismic Base Isolation
Department of Energy

August 10 - August 14, 1992

J.M. Kelly

August 10-14, 1992

LECTURE 1

The Development and Implementation of Base Isolation In the United States and Abroad

Historical Development of Base Isolation

In the past several years base isolation has become an accepted structural design technique for buildings and bridges in highly seismic areas. Many buildings and other types of structures have been built using this approach, and many new buildings are under construction or in the design phase. Most of the completed buildings and those under construction use rubber isolation bearings in some way in the isolation system, but new techniques are being proposed and developed and as this seismic design technique spreads it is likely that alternative techniques will be applied.

The ideas behind the concept of base isolation are quite simple. There are two basic types of isolation systems. The system which has been adopted most widely in recent years is typified by the use of elastomeric bearings, the elastomer being either natural rubber or neoprene. In this approach, the system works by decoupling the building or structure from the horizontal components of the earthquake ground motion by interposing a layer with low horizontal stiffness between the structure and the foundation. This layer gives the structure a fundamental frequency that is much lower than its fixed-base frequency, and also much lower than the predominant frequencies of the ground motion. The first dynamic mode of the isolated structure involves deformation only in the isolation system, the structure above being to all intents and purposes rigid. The higher modes which produce deformation in the structure are orthogonal to the first mode, and consequently, also to the ground motion. These higher modes do not participate in the motion so that if there is high energy in the ground motion at these higher frequencies this energy cannot be transmitted into the structure. The isolation effect in this type of system is produced not by absorbing the earthquake energy, but rather by deflecting it through the dynamics of the system. It is worth noting that this type of isolation works when the system is linear and even when undamped. Some damping, however, is beneficial to suppress any possible resonance at the isolation frequency.

The second basic type of isolation system is typified by the sliding system. This works by limiting the transfer of shear across the isolation interface. The lower the coefficient of friction the less shear is transmitted. This is the earliest of proposed systems and is the simplest. However, it is not without problems. To sustain wind load and unnecessary slip under small earthquakes or other disturbances, a fairly high value of the frictional coefficient is needed. Many frictional surfaces have sliding characteristics that are sensitive to pressure and the relative velocity of slip, and the fact that the slip process is intrinsically nonlinear means that a proper dynamic analysis must also be nonlinear. A further problem is that the sudden change in the stiffness of the overall structure when slipping or sticking occurs has the effect of generating high-frequency vibrations in the structure, vibrations at frequencies which may not even be present in the ground motion. The action of the system is to transform low-frequency energy into high-frequency energy in the structure. Nevertheless, the advantages of the concept, the lack of resonance and the potential low cost, make sliding systems quite attractive. Systems have been recently proposed that are hybrids between the elastomeric systems and the sliding systems and it is possible that the best system for a wide class of building types might eventually evolve from a combination of these two simple types.

Early Efforts at Base Isolation

The severe earthquake in 1908 in the Italian region of Messina-Reggio killed 160,000 people, mainly by crushing them under fallen masonry. Following this earthquake a commission was formed consisting of practising engineers and university professors The Commission was charged with recommending structural engineering design methods for the reconstruction of the damaged area. Two proposals arose from the Commission's studies: one suggested separating a building from its foundation by a layer of sand or by use of rollers; the other favored a fixed foundation. The Commission finally decided on the fixed foundation approach with constraints on several construction details and a requirement that the building be able to support at least 8% of its weight as a horizontal force. At exactly the same time, a medical doctor, Dr. Calantarient, from Scarborough in England applied for a British patent on an earthquake-resistant design approach which proposed separating a building from its foundation by a layer of sand or talc. Dr. Calantarients' interest in earthquake engineering, although presumably amateur, was not superficial, for he designed wind restraints which would prevent the building from moving in high winds, and, recognizing that reducing damage to the building would require the possibility of a large shift between the building and the ground, he showed how the utilities and access to the building had to be designed for this. Why a medical doctor in England, not known as a seismic region, should be interested in earthquake engineering is not clear, but the name is Armenian and he may have been at the time a recent immigrant from the Near East where earthquakes and the collapse of masonry buildings are a recurrent threat.

Possibly the first person to use this concept in a building was Frank Lloyd Wright. His design of the Imperial Hotel completed in Tokyo in 1921 was in complete contrast to accepted practice at the time and was extremely controversial. Under the site was an 8 ft layer of fairly good soil and below that a layer of soft mud. This layer appeared to Wright as a ''good cushion to relieve the terrible shocks. Why not float the building on it?''. He tied the building to the upper layer of good soil by closely spaced short piles that penetrated only as far as the top of the soft mud. The building performed extremely well in the devastating 1923 Tokyo earthquake. It was a very highly decorated building with appendages of many kinds, and buildings of this sort generally are badly damaged in earthquakes. The only damage was to statuary in the courtyard of the hotel. The famous architect's intuitive idea of floating the building ''as a battleship floats on the ocean'' appears to have worked.

Since fortuitous layers of soft mud are unlikely at building sites, other ways to reduce earthquake damage were sought by engineers. In the late twenties and thirties, the concept of the flexible first story was proposed by structural engineers. In this approach the lateral stiffness of the columns of the first story would be designed to be much lower than that of the columns above, and under earthquake loading the deformations in the structure would be concentrated in these first-story columns. To be effective in reducing accelerations at the upper levels, however, the displacements in the first-story columns would be of the order of several inches, and the effect of the weight of the upper levels on this sideways displacement of the columns could produce severe damage to the columns, making collapse of the building a distinct possibility. The first-story columns in this approach behave elastically and the damping is low.

In a modified approach, called the soft first-story method, the first-story columns yield during an earthquake, producing an energy-absorbing action and controlling the displacements. However, to produce enough damping the displacements would still have to be several inches and a yielded column has a greatly reduced buckling load, so that column instability and collapse are inevitable.

The search for a mechanism that can carry the weight of the building and tolerate the large sideways movement caused by earthquakes has been unending. Many types of roller bearing systems have been proposed, and several have been patented. Since the ground movement may occur in any direction, it is necessary to use spherical bearings or two crossed layers of rollers. The damping of rollers is low and they have no inherent resistance to wind so that some other mechanism must provide wind restraint and energy-absorbing capacity. A permanent offset may result in such a system after an earthquake since there are no restoring forces.

Since a roller isolation system could sit unattended and unmaintained for several decades in the basement of a building, it is likely that its performance when called upon would be worse than that of mechanical bridge bearings which are in constant movement due to the daily temperature cycle. When steel presses against steel for a long period, there is a possibility of cold welding, which would cause the system to become rigid after a time. Nevertheless, roller bearing systems have been used. A demonstration building in Sebastopol in the Crimea has been built on steel bearings. The bearings are doubly spherical ovoids, egg-shaped and not quite spherical. When the building is displaced it is forced to rise and this produces a restoring force. The building is a seven-story reinforced concrete building and acts as an oscillator with a three-second period which provides considerable protection from earthquake attack as compared to a conventional seven-story reinforced concrete building which would have a period of about 0.5 second, the most intense frequency region of most earthquake ground motions. Not a great deal has been published in Western literature regarding this building, but it apparently experienced an earthquake in 1977 and performed to expectation.

A four-story school in Mexico City was completed in 1974 on a roller bearing isolation system. The school has a reinforced concrete frame with brick infill walls. Each column of the building is carried on two isolation elements. An isolation element has two steel disks separated by a hundred or more 1 cm ball bearings, the exact number in each element being fixed by the vertical load it must carry. There is a keeper ring which prevents them from rolling out and limits the relative displacement to about 12 cm. There is no evidence that the system was activated during the 1985 Mexico earthquake; the ground motions in the area of the city where the structure is located were of low intensity.

The first use of rubber for earthquake protection was in an elementary school in Skopje, Yugoslavia. The building is a three-story concrete structure and was completed in 1969. It rests on large blocks of natural rubber. In contrast with more recent rubber bearings, these blocks are completely unreinforced so that the weight of the building causes them to bulge sideways. The vertical stiffness of the system is about the same as the horizontal stiffness so that the building will bounce and rock backwards and forwards in an earthquake. These bearings were designed at a time when the technology for reinforcing rubber blocks with steel plates, as in bridge bearings, was not highly developed nor widely known and it is unlikely that this approach will be used again.

Research on Base Isolation at EERC

Research on the development of natural rubber bearings for the protection of buildings from earthquakes began at the Earthquake Engineering Research Center (EERC) of the University of California at Berkeley in 1976. At that time, the idea of isolating buildings from earthquake attack was not entirely new. A few methods using rollers or sliders had been proposed, but it was still considered then to be very esoteric and impractical by the structural engineering profession. The ten-year research program initiated then has led to a very substantial degree of acceptance for the concept by the profession and to the construction of several buildings and the design and planned construction of many others.

The research project began with a set of hand-made bearings of extremely low-modulus rubber used with a simple three-story, single-bay, twenty-ton model. The bearings and the model were far from ideal but the shaking table tests showed that isolation bearings could work. They showed that reductions in acceleration by factors of as much as ten as compared to conventional design were possible, and as predicted, the response of the model was as a rigid body with all deformation concentrated at the isolation system. It was also clear that a certain degree of damping was needed in the system and that the scale of the model was too small to allow more practical rubber compounds to be used.

Many ideas were explored in pursuit of increased damping; steel energy absorbing devices, hydraulic dampers and oil-extended rubber were tried. A practical system and more convincing demonstration of the concept was achieved in 1978 by the use of bearings made by commercial techniques and used with a more realistic five-story three-bay model weighing forty tons. A film was made of the tests showing graphically the substantial reductions in acceleration and the rigid-body movement of the structure. This film has been shown perhaps thousands of times to professional and lay audiences and has had a major effect in improving acceptance for the isolation concept.

A strong interest throughout the research program has been the influence of isolation on the response of equipment and contents in a structure. In many buildings the equipment is much more costly than the structure, and conventional methods of seismic resistant design tend to increase the attack of the earthquake on the contents. The research at EERC, through a considerable series of tests on the five-story frame, was able to demonstrate that isolation with rubber bearings could provide very substantial reductions in the accelerations experienced by internal equipment, exceeding the reductions experienced by the structure itself. However, the same tests showed that when additional elements were added to the isolation system to increase damping (elements such as steel energy-absorbing devices, frictional systems or lead plugs in the bearings), the reductions in acceleration to the equipment were not achieved. The effect of these additional elements, while acting to control displacements, is to induce responses in the higher modes of the structure and these in turn have a very deleterious effect on the equipment. It became clear that the optimum method of increasing damping is to provide it in the rubber compound itself and this was in fact done later for the compound used in the first building built on isolation bearings in the United States.

Before this development however, the EERC base isolation research program was involved in the design of an isolation system for the seismic rehabilitation of a large building in San Francisco. The building was constructed in 1912 as a Masonic Hall and was handsome with an elegantly finished interior. It was abandoned in 1951 as seismically unsafe and had remained unoccupied although well maintained since then. A proposal was made to use its many large rooms as concert halls and rehearsal rooms but to do so would require a complete seismic rehabilitation that would be expensive and possibly destructive to the fine interior and to the very features that would have made it an ideal performing arts center. A base isolation scheme for rehabilitation was developed and sample bearings were manufactured and tested in a specially built test rig at EERC to verify that they would perform as required for the design.

For reasons unrelated to the seismic rehabilitation, the proposal to convert the Masonic Hall to a performing arts center was abandoned, and the isolation rehabilitation design was not implemented. However, the project provided considerable experience in the practical details of isolation systems and a clear picture of the steps needed to design a cost-competitive isolation system using rubber bearings.

Modern Approaches to Seismic Isolation

Rubber bearings offer the simplest method of isolation and are relatively easy to manufacture. The bearings are made by vulcanization bonding of sheets of rubber to thin steel reinforcing plates. The bearings are very stiff in the vertical direction and are very flexible in the horizontal direction. Their action under seismic loading is to isolate the building from the horizontal components of the earthquake ground movement, while the vertical components are transmitted through to the structure relatively unchanged. Vertical accelerations are not normally a problem for most buildings. These bearings will in addition have the effect of isolating the building from high-frequency vertical vibrations that are produced by underground railways and local traffic. A building on rubber bearings will be simultaneously protected from unwanted vibration and from earthquake attack. Rubber bearings are suitable for stiff buildings up to seven stories in height. For this sort of building, uplift on the bearings will not occur and wind load will be unimportant.

The mechanics of isolation bearings is now very well understood and the design of a system for a building at a specific site is a straightforward task. When a building is analyzed on rubber bearings for earthquake attack, it is usual to model the combined system by a linear viscously damped model. If this model is used, very simple solutions will result. If the fixed-base fundamental frequency of the building is much higher than that of the isolated system, say 3 Hz as compared to 0.5 Hz for the isolated case, the first mode of the isolated building is mainly a rigid body mode with all deformation occurring in the rubber. The second mode has a frequency about 50% to 100% above the first fixed-base frequency. The seismic input to the structure can be treated as an equivalent lateral load which is proportional to the rigid body mode. Since it is a characteristic of a linear vibrating system that all modes are mutually orthogonal, all modes higher than the first will be orthogonal to the input motion, so that if there are high energies in the earthquake ground movement at the frequencies of these higher modes, this energy cannot be transmitted into the building. Thus, the isolation system works not by absorbing these energies but by deflecting them. This is the most attractive feature of the simple rubber isolation system. If other elements are added for the purpose of increasing the damping or controlling displacement, this simple result no longer holds. Accelerations will then be induced in the higher modes and will produce stresses in the higher levels of the building and cause accelerations in equipment items and other contents of the building.

A simple form of rubber bearing isolation system was used for a three-story school in the small town of Lambesc near Marseilles in France. The school includes three buildings, each separated by a seismic gap. There are 152 natural rubber laminated bearings in the isolation system. The school buildings had originally been designed to be built of prefabricated concrete but the seismic code for the region was changed before construction began. The system could not have satisfied the changed seismic requirements without a substantial increase in cost. The use of the isolators allowed the system to satisfy the new code and saved the community a great deal of money. In this building there are no wind restraints or additional elements to enhance the damping and the period of the isolated building is around 1.70 seconds.

Since this school was completed, three houses have been built in the neighboring community of Saint-Martin de Castillon. The houses are of masonry construction, have tile roofs and are supported on 15 cm diameter natural rubber isolators. An isolation system of this kind has been designed for a three-story building now under construction in Toulon for the French Navy. This building is to be used for the storage of radioactive waste. No wind restraints or damping devices are used in this system since the displacement under wind load and earthquake action is likely to be very small.

In New Zealand, a number of isolation concepts have been applied to highway bridges, railway bridges and two buildings. One of the buildings, a government office building in Wellington, uses as isolators laminated natural rubber bearings each of which has a cylindrical plug of lead in a central hole. This system was developed in the late 1970's, since it was felt that the intrinsic damping in the rubber compounds available in Australasia at that time was inadequate to control the displacements of the isolation system. The lead plug produces an increase in both resistance to wind loading and a substantial increase in damping, from approximately 3% of critical damping in the rubber then available to about 10-15%. The building is four stories high and has a reinforced concrete frame designed to withstand the earthquake forces which would be caused if the building had a conventional fixed foundation, but there are some architectural features which would not have been allowed if isolation had not been used.

Shaking table tests of a model structure on lead plug bearings have been performed. The experimental results show that the lead plugs generally reduce the system displacement but cause increased higher mode response. There is also evidence that the damping is dependent on the degree of confining pressure on the lead plug provided by the bearing. There have been problems with the lead working into the rubber and with the lead plug fracturing, reducing its effectiveness. Development work on the system continues and tests have been carried out on the use of materials which could be substituted for lead and produce the same degree of damping without the associated problems.

A twelve-story building in Auckland, New Zealand, has been built on a base isolation system called the sleeved-pile system. This uses long bearing piles within cylindrical sleeves, allowing a certain amount of lateral movement, in this case about 6 in. The isolation period on the piles is 4 seconds, and resistance to wind loading would be inadequate with this system. In addition the damping would be very low. To improve the behavior of the system, energy-absorbing devices in the form of mild steel tapered plate beams are included in the structure and these lower the period to around 2 seconds.

The sleeved-pile concept is similar to the soft first story design concept but without the risk of collapse due to excessive first story lateral deflections. If the structure should exceed the design lateral displacements, the sleeve itself will control the displacement, providing a fail safe action for the system. Although piles are an expensive foundation system they must be used if soil conditions make the use of footings unacceptable. When circumstances dictate the use of piles it may be cost effective to use the sleeved-pile concept and provide a substantial reduction in the lateral force requirements for the superstructure.

Isolation systems have been proposed in which the isolation mechanism is purely sliding friction. These are the simplest isolation systems of all, and there has been a large amount of theoretical analysis of sliding systems but very little experimental work and as far as is known no large-scale shaking table tests. The idea of a sliding joint as an isolation system is an attractive one for low-cost housing since it can be constructed using no more complicated technology or no more skilled labor than a conventional building. For this reason, it has been developed for housing in China. It was observed after the Tang Shan earthquake of 1976 that masonry block buildings in which the reinforcement was not carried through to the foundation performed better than buildings in which it did. In a structure which performed well in the earthquake a horizontal crack was observed at the foot of the wall as well as a residual displacement of about 6 cm.

As a result of these observations, the approach adopted in China is a separation layer under the floor beams above a wall foundation. A thin layer of specially screened sand is laid on the sliding surface and the building constructed on this layer. Since low-rise concrete block or masonry buildings are very stiff, heavy structures, they are susceptible to earthquake damage. The presence of the sliding layer allows a degree of flexibility which reduces the seismic risk. Four demonstration buildings have been built in China using this technique. Three of these are one-story brick houses and one a four-story brick dormitory in Beijing for the Strong Motion Observatory Center.

A nuclear power plant in Koeberg, South Africa has been built on an isolation system by the French nuclear construction company, Framatome. This company supplies a standard power plant designed for a seismic input of 0.2g peak acceleration. For a site where the design requirements exceed this, as at Koeberg, the power plant is built on an isolation system which will reduce the accelerations experienced by the structure and components. The French nuclear isolation system uses laminated neoprene bridge bearings with lead bronze-stainless steel slip plates on top of each bearing. The neoprene bearings act as conventional isolators for small earthquakes, but cannot accept large displacements since they have only a few layers of elastomer. If a large earthquake should occur, sliding will take place on the slip plates. These have been designed to have a friction coefficient of 0.2 and to maintain this value for the life-time of the plant. The construction costs for this system are very high but are justified in that it allows a standardized plant to be built at any site with no additional costs for redesign, strengthening and requalification of components for seismic loads. The Koeberg plant began commercial operation in 1987.

Neoprene pads without slip plates are used under the reactor buildings of a four-unit nuclear power plant at Cruas-Meysse in the Rhone Valley. The pads are similar to standard neoprene bridge bearing pads, with three layers of elastomer reinforced with steel plates. An isolation system is used for this site since there is a probability of shallow earthquakes of low magnitude (Richter 4 to 4.5) occurring close to the site and producing high accelerations and high-frequency ground motion.

The first base-isolated building in the United States is the Foothill Communities Law and Justice Center located in the municipality of Rancho Cucamonga in San Bernardino County. The building is a thirty-million dollar legal services center for the county. It is four stories high with a full basement and sub-basement for the isolation system. The building sits on 98 isolators which are multilayer natural rubber bearings reinforced with steel plates. The superstructure of the building has a structural steel frame stiffened by braced frames in some bays.

The site of the building is 20 kilometers from the San Andreas fault, and the County, which has the most thorough earthquake preparedness program in the country, asked that it be designed for the maximum credible earthquake for that site. The design was based on a Richter magnitude 8.3 earthquake on the fault. The design selected for the isolation system, which took into account possible torsion of the building, led to a maximum displacement demand of 15 in. (380 mm) in the isolators at the corners of the building. Tests of full-scale sample bearings verified that 15 in. relative horizontal displacement is within the capacity of the bearings.

The rubber from which the isolators are made is a highly filled natural rubber with mechanical properties that make it ideal for a base isolation system. The shear stiffness of this rubber is high for small strains but decreases by a factor of about four or five as the strain increases, reaching a minimum value at a shear strain of 50%. For strains greater than 100% the stiffness begins to increase again. Thus, for small loading caused by wind or low-intensity seismic loading the system has high stiffness and short period and as the load intensity increases the stiffness drops. For very high load, say above the maximum credible earthquake, the stiffness increases again providing a fail-safe action. The damping follows the same pattern but less dramatically, decreasing from an initial value of 20% to a minimum of 10% and then increasing again. In the design of the system, the minimum values of stiffness and damping are assumed and the response is taken to be linear. The high initial stiffness is invoked only for wind load design and the large strain response only for fail-safe action.

This high-damping rubber system has been adopted for a recently completed building in Los Angeles. The building is a Fire Department Command and Control Facility for Los Angeles County. The building houses the computer systems for the emergency services program of the county and the facility is required to remain functional after an extreme event. The decision to use seismic isolation for this project was based on a comparison of conventional and isolation schemes designed to provide the same degree of protection. On this basis, the isolated design was estimated to cost less, by 6%, than the conventional design. It is worth noting that in most projects the isolated design is compared with conventional code design and generally the isolated design is estimated to cost around 5% more. However, code design is a minimal level of design, requiring only that the building should not collapse. It does not mean that there will be no structural damage under strong ground shaking. When equivalent levels of protection are compared, isolated designs are always less expensive. These costs are first costs. Life cycle costs are even more favorable for isolated designs.

The high-damping rubber bearings were also used in a building recently completed in Italy. This very large office building for the Italian telephone company, S.I.P., is located in Ancona. and was completed in 1992. It is the first modern base isolated building in Europe.

Base-Isolated Buildings and Projects in the United States
(as of June 1992)

New Base-Isolated Projects

Foothills Communities Law and Justice Center

Location:	Rancho Cucamonga, California
Status:	New
Owner:	County of San Bernardino
Size:	230,000 sq. ft.
Total Cost:	$36 million
Completed:	1985
Engineers:	Taylor & Gaines; Reid & Tarics Assoc.
System:	HDR
Supplier:	Oil States Industries (now LTV)

University of Southern California Hospital

Location:	Los Angeles, California
Status:	New
Owner:	USC and National Medical Enterprises
Size:	250,000 sq. ft.
Cost:	$50 million
Completed:	1988
Engineers:	KPFF
System:	LRB
Supplier:	DIS/Furon

Kaiser Computer Center

Location:	Corona, California
Status	New
Owner:	Kaiser Foundation Health Plan
Size:	120,000 sq. ft.
Cost:	$32 million
Completion:	Under construction 1992
Engineers:	Taylor & Gaines
System:	LRB & HDR
Supplier:	DIS/Furon

Aircraft Simulator Manufacturing Facility

Location:	Salt Lake City, Utah
Status:	New
Owner:	Evans and Sutherland, Corp.
Size:	140,000 sq. ft.
Cost:	$8 million
Completed:	1988
Engineers:	Reavely Engineers and Assoc.; DIS
System:	LRB
Supplier:	DIS/Furon

Fire Command and Control Facility

Location:	East Los Angeles, California
Status:	New
Owner:	County of Los Angeles
Size:	32,000 sq. ft.
Cost:	$6.3 million (excludes installed equipment)
Completed:	April 1990
Engineers:	Fluor-Daniel Engineers, Inc.
System:	HDR
Supplier:	Fyfe Assoc./Dynamic Rubber

Titan Solid Rocket Motor Storage

Location:	Vandenburg Air Force Base, California
Status	New
Owner:	U.S. Air Force
Size:	N.A.
Cost:	N.A.
Completion:	1992
Engineers:	Bechtel Corp.
System:	HDR
Supplier:	LTV

San Bernardino Medical Center

Location:	Colton, California
Status	New
Owner:	County of San Bernardino
Size:	Five buildings, totaling 900,000 sq. ft.
Cost:	N.A.
Completion:	Design & review 1992
Engineers:	KPFF; Taylor & Gaines
System:	HDR
Supplier:	DIS

M.L. King Jr.-C.R. Drew Diagnostics Trauma Center

Location:	Watts, California
Status	New
Owner:	County of Los Angeles
Size:	140,000 sq. ft.
Cost:	$40 million
Completion:	Final design OSHBD review 1992
Engineers:	John Martin Assoc.; BIC
System:	HDR & Bronze Alloy Sliders
Supplier:	Not selected

Emergency Operations Center

Location:	East Los Angeles, California
Status	New
Owner:	County of Los Angeles
Size:	33,000 sq. ft.
Cost:	$6 million
Completion:	Construction starts 1992
Engineers:	DMJM
System:	HDR
Supplier:	Not selected

San Francisco Main Public Library

Location:	San Francisco, California
Status:	New
Owner:	City & County of San Francisco
Completed:	Design in progress 1992
Cost:	N.A.
Completion:	N.A.
Engineers:	Olmm Structural Design; Forell/Elsesser Eng.
System:	Not selected
Supplier:	N.A.

Water Control Center-Water Quality Laboratory

Location:	Portland, Oregon
Status:	New
Owner:	Portland Water Bureau
Size:	28,000 sq. ft.
Cost:	N.A.
Completion:	Design phase 1992
Engineers:	Harris Group; DIS
System:	LRB
Supplier:	DIS

Two Residences

Location:	West Los Angeles, California
Status:	New
Owner:	David Lowe
Size:	4,700 sq. ft. each
Cost:	$20,000 for each base
Completion:	1992
Engineers:	David Lowe
System:	GERB Resistant Base
Supplier:	GERB

Retrofit Base-Isolated Projects

Salt Lake City and County Building

Location:	Salt Lake City, Utah
Status:	Retrofit
Owner:	Salt Lake City Corp.
Size:	170,000 sq. ft.
Cost:	$30 million (inc. non-seismic rehab.)
Completed:	1988
Engineers:	E.W. Allen and Assoc.; Forell/Elsesser Engineers
System:	LRB
Supplier:	DIS/LTV

Mackay School of Mines

Location:	Reno, Nevada
Status	Retrofit
Owner:	University of Nevada, Reno
Size:	27,000 sq. ft.
Cost:	$7 million
Completion:	Under construction 1992
Engineers:	Jack Howard and Assoc.; BIC
System:	HDR & PTEF sliders
Supplier:	Furon

Channing House Retirement Home

Location:	Palo Alto, California
Status	Retrofit
Owner:	Non-profit corporation
Size:	260,000 sq. ft.
Cost:	N.A.
Completion:	In design phase 1992
Engineers:	Renne & Peterson; DIS
System:	LRB
Supplier:	DIS

Rockwell Seal Beach Facility

Location:	Seal Beach, California
Status:	Retrofit
Owner:	Rockwell International
Size:	300,000 sq. ft.
Cost:	$14 million
Completed:	1991
Engineers:	Englekirk & Hart
System:	LRB
Supplier:	DIS/Furon

Marina Apartments

Location:	San Francisco, California
Status	Retrofit
Owner:	Dr. Hawley
Size:	20,000 sq. ft.
Cost:	N.A.
Completion:	1991
Engineers:	EPS
System:	FPS
Supplier:	EPS

Long Beach Hospital

Location:	Long Beach, California
Status	Retrofit
Owner:	Veteran's Administration
Size:	350,000 sq. ft.
Cost:	N.A.
Completion:	Final Design 1992
Engineers:	A.C. Martin & Assoc.
System:	Not selected
Supplier:	N.A.

Hayward City Center

Location:	Hayward, California
Status	Retrofit
Owner:	City of Hayward
Size:	145,000 sq. ft.
Cost:	$7 million
Completion:	Final design 1992
Engineers:	EQE, San Francisco; C. Kircher Assoc.
System:	FPS & HDR
Supplier:	EPS & Bridgestone

Asian Art Museum

Location:	San Francisco, California
Status	Retrofit
Owner:	City and County of San Francisco
Size:	170,000 sq. ft.
Cost:	N.A.
Completion:	Conceptual design in progress
Engineers:	Rutherford & Chekene; C. Kircher Assoc.
System:	Not selected
Supplier:	N.A.

50 United Nations Plaza

Location:	San Francisco, California
Status	Retrofit
Owner:	U.S. General Services Administration
Size:	345,000 sq. ft.
Cost:	N.A.
Completion:	Architect/Engineer selection 1992
Engineers:	N.A.
System:	N.A.
Supplier:	N.A.

U.S. Customs House

Location:	San Francisco, California
Status	Retrofit
Owner:	U.S. General Services Administration
Size:	142,000 sq. ft.
Cost:	$14 million (estimate)
Completion:	Conceptual design in progress
Engineers:	URS Blume
System:	Not selected
Supplier:	N.A.

San Francisco City Hall

Location:	San Francisco, California
Status	Retrofit
Owner:	City and County of San Francisco
Size:	N.A.
Cost:	N.A.
Completion:	Conceptual design in progress
Engineers:	Forell/Elsesser Eng.
System:	Not selected
Supplier:	N.A.

Oakland City Hall

Location:	Oakland, California
Status	Retrofit
Owner:	City of Oakland
Size:	153,000 sq. ft.
Cost:	$47 million (estimate)
Completion:	Final design 1992
Engineers:	Forell/Elsesser Engineers; DIS
System:	LRB
Supplier:	DIS

State of California Justice Building

Location:	San Francisco, California
Status	Retrofit
Owner:	State of California
Size:	250,000 sq. ft.
Cost:	$40 million (est., inc. non-seismic renovation)
Completion:	Conceptual Design 1992
Engineers:	Rutherford & Chekene; C. Kircher Assoc.
System:	Not selected
Supplier:	N.A.

U.S. Court of Appeals

Location:	San Francisco, California
Status	Retrofit
Owner:	U.S. General Services Administration
Size:	350,000
Cost:	N.A.
Completion:	Conceptual Design 1992
Engineers:	Skidmore, Owings & Merrill
System:	FPS
Supplier:	EPS

Kerckhoff Hall, UCLA

Location:	Los Angeles, California
Status	Retrofit
Owner:	Regents, Univ. of Calif.
Size:	100,000 sq. ft.
Cost:	$15.3 million
Completion:	December, 1994
Engineers:	Brandow & Johnston
System:	Not selected
Supplier:	N.A.

Educational Services Center

Location:	Los Angeles, California
Status	Retrofit
Owner:	L.A. Community College District
Size:	90,000 sq. ft.
Cost:	$450,000
Completion:	August 1992
Engineers:	Fleming Corp.
System:	Earthquake Barrier
Supplier:	N.A.

Lecture One

List of Slides

1 Experience in the Design and Implementation of Seismically Isolated Structures
2 Base Isolation - Potential Applications
3 Base Isolated Condition / Fixed-base Condition
4 Acceleration Response Spectrum
5 Displacement Response Spectrum
6 Comparison of Non-Isolated and Isolated Response of El Centro Earthquake, 1940
7 Orthogonality of Input to Higher Modes for Base-isolated Structures
8 Dr. Calantarient's Designs
9 Dr. Calantarient's Designs
10 Olive View Hospital - Soft Story
11 Olive View Hospital - Psychiatric Ward, Collapsed Columns
12 Early Efforts and Isolation Using Natural Rubber
13 View of School in Skopje, Macedonia
14 School, Skopje - Cross-section
15 School, Skopje - Natural Rubber Isolators in School
16 View of School in Marseilles Area, Southern France
17 School, France - Natural Rubber Isolators for School, Shown Under Construction
18 School, France - Natural Rubber Isolators for School, Completed Structure
19 School, France - Seismic Gap for Isolated School
20 View of Isolated Dormitory Building in Beijing, PRC
21 Dormitory, PRC - View of
22 Dormitory, PRC - View Showing Seismic Gap
23 Dormitory, PRC - Cross-section of Isolation System
24 View of Isolated School Building in Mexico City
25 School, Mexico City - Front View of School Showing Seismic Gap
26 School, Mexico City - Additional View of School Showing Seismic Gap
27 School, Mexico City - View of Isolators in School Basement
28 Modern Approaches to Base Isolation
29 View of Isolated Office Building in Wellington, New Zealand
30 Office Building, NZ - Cross-section
31 Office Building, NZ - Isolators Shown During Construction
32 Office Building, NZ - Isolators Shown During Construction
33 Office Building, NZ - View of Construction
34 Office Building, NZ - Isolators in Completed Building
35 Office Building, NZ - View of Completed Building
36 View of Union House, Aukland - Sleeved-Pile Isolation System
37 Cross-section of Union House Showing Sleeved Piles
38 Union House, NZ - Sleeved Piles During Construction
39 Union House, NZ - Sleeved Piles During Construction
40 Union House, NZ - Steel Energy Dissipators

LECTURE 2

Base Isolation In Japan

by

James M. Kelly
Professor of Civil Engineering
Earthquake Engineering Research Center
University of California
Berkeley, California 94720

Short Course on Seismic Base Isolation
Department of Energy

August 10 - August 14, 1992

Department of Energy
Short Course on Seismic Base Isolation

J.M. Kelly
August 10-14, 1992

LECTURE 2

Base Isolation In Japan

Introduction

After a slow start, base isolation research and development in Japan has increased rapidly. The first large base-isolated building was completed in 1986. Base-isolated buildings in Japan require special approval from the Ministry of Construction. Even so, by mid-1992, a total of 65 buildings had been completed, under construction, or had been approved.

Base isolation has advanced rapidly in Japan for several reasons. The expenditure for research and development in engineering is high, with a significant amount - $25 million - designated specifically for base isolation; the large construction companies aggressively market the technology; the approval process for constructing a base-isolated building is a straightforward and standardized process; and the high seismicity of Japan - severe earthquakes are common - encourages the Japanese to favor the long-term benefits of life safety and building life-cycle costs when making seismic design decisions.

Implementation of Base Isolation in Japan

Active development of base isolation began in the United States in the mid-1970's. Experimental research was carried out at the Earthquake Simulator Laboratory of the Earthquake Engineering Research Center (EERC) of the University of California at Berkeley. Several types of isolation systems were studied using very large structural models representing low- and medium-rise steel frames or reinforced concrete buildings. This research at EERC led to further experimental work at other American institutes. Acceptance of this new seismic design approach has, however, been slow. The first base-isolated building in the United States, the Foothill Communities Law and Justice Center in San Bernardino County, was completed in December 1985, and another in Salt Lake City, Utah was completed in late 1988.

In contrast, base isolation research and development was slow to start in Japan, but since the first large base-isolated building was completed in 1986, the use of base isolation has increased very rapidly. This is due in large part the the large construction companies that have taken the leading role in developing base isolation technology. These companies have decided that base isolation is superior to conventional seismic design and can provide a competitive edge in the construction industry. Therefore, they are aggressively marketing the technology to potential clients. The first buildings to use base isolation were, accordingly, demonstration projects with the construction companies building them for their own use. More recent projects have been for a variety of clients have have included office buildings, manufacturing facilities, and apartment blocks.

The most common form of base isolation used in Japan is steel-laminated natural rubber bearings with additional devices to enhance the damping in the system. The rubber compounds used have very low intrinsic damping and the additional elements, such as steel rods, viscous dampers, hydraulic shock absorbers, and lead plugs, among others, have been used to produce the necessary damping in the system. In a way, these designs mirror the research history in the United States where in extensive shake table testing, many energy-absorbing mechanisms were tried in parallel with the rubber bearings. As the experimental research progressed over a period of ten years, the various additional dampers were shown to be unnecessary, inconvenient, and sometimes deleterious and the damping was incorporated in the elastomer itself through appropriate compounding. The high-damping natural rubber isolation system used in the San Bernardino building evolved from this research program. Recently, Bridgestone Corporation in Japan developed a natural rubber with high damping and several of the construction companies in Japan are evaluating this approach. These isolators have been installed in one demonstration building as described below.

Demonstration Buildings on Base Isolation Systems

There is a long history of innovative earthquake-resistant design in Japan. Many ingenious devices and structural systems have been proposed and some implemented. A review of these unconventional approaches has been given by Izumi [1]. None of these has achieved enough acceptance by the structural engineering profession in Japan to achieve widespread use.

The acceptance of base isolation as an earthquake-resistant design strategy has also been very slow in Japan, although proposals for its used were made in the late seventies. In 1978, a construction company called Unitika advertised a natural rubber isolation system called the Yurine bearing but no use was made of the technique.

The first isolated building in Japan was built in 1982. It is a small two-story house built on six natural rubber bearings produced by Bridgestone Corporation. The building was constructed by Tokyo Kenchiku Structural Engineers and research on its seismic response has been carried out by the faculty of Fukuoka University. Details of the structure, the isolations system, and a bearing under test are shown in Figs. 1-4. It is approximately 10 m by 5 m in plan and was constructed in a conventional way so as to allow its use as housing. It is located in Yachiyo City in Chiba Prefecture, an area that experiences relatively frequent earthquakes. In fact, during a three year period, seventeen earthquakes were recorded on the system of strong motion accelerographs installed in the building.

The rubber bearings used in this building are 30 cm in diameter and 8.2 cm high. The elastomer is a relatively unfilled natural rubber providing a natural period of approximately 1.8 seconds and damping of approximately 3%. A variety of different types of additional damping components was investigated in the building including an elasto-plastic type using a cantilevered steel rod, a sand type using friction between a steel rod and sand, a frictional type, and a damper using precast concrete plates that rub against the side of the building. The damper with the most satisfactory performance in force-vibration and free-vibration tests was the elasto-plastic damper; this provided an equivalent damping of around 20% for the system.

Observations of the earthquake response of the building over a three-year period after completion and installation of the seismometers were reported by Tada [2], and have been highly favorable, although the ground motions experienced have, in most cases, been quite small. The roof accelerations have always been less than the ground accelerations, sometimes dramatically so, and as expected, the acceleration reduction ratio increases with increasing ground motion. The highest recorded input during the initial monitoring period occurred in late 1985 when a peak-ground motion of 10.2% produced a peak-roof acceleration of 2.4%. A summary of the recorded data is shown in Fig. 5. The observation program is continuing and in December 1987 an earthquake with a ground motion of 13.1% was recorded. The maximum roof acceleration in this case was only 3.5% g, demonstrating the remarkable attenuation of the isolation system.

Almost certainly as a result of the entirely favorable response of the building in Yachiyo City, a number of much larger buildings using base isolation were constructed in Japan with many being completed in late 1986 and 1987. In almost every case, these buildings were designed and built by construction companies as demonstration

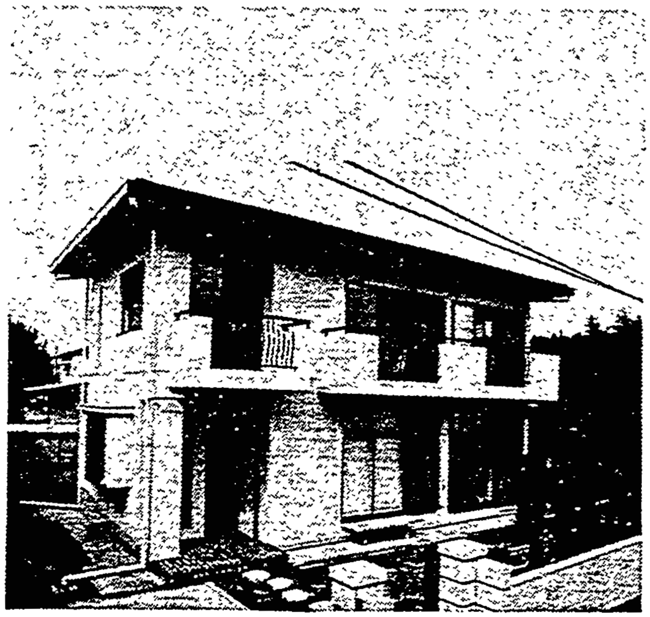


Figure 1 Base-Isolated Dwelling in Yachiyo City

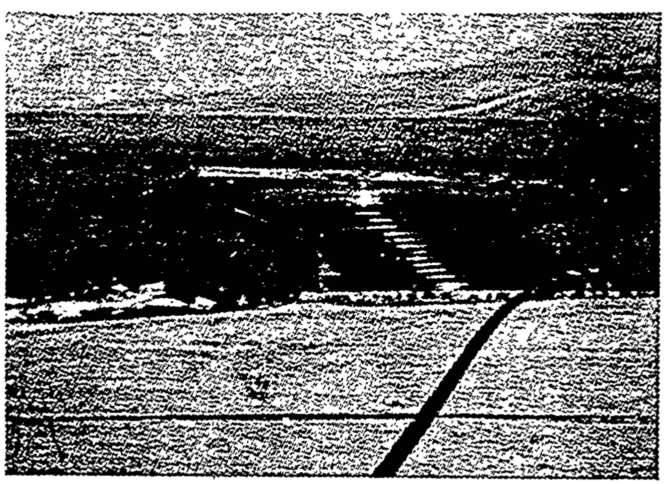


Figure 2 Deformation of Isolation Bearing in Pull-Back Test of Building

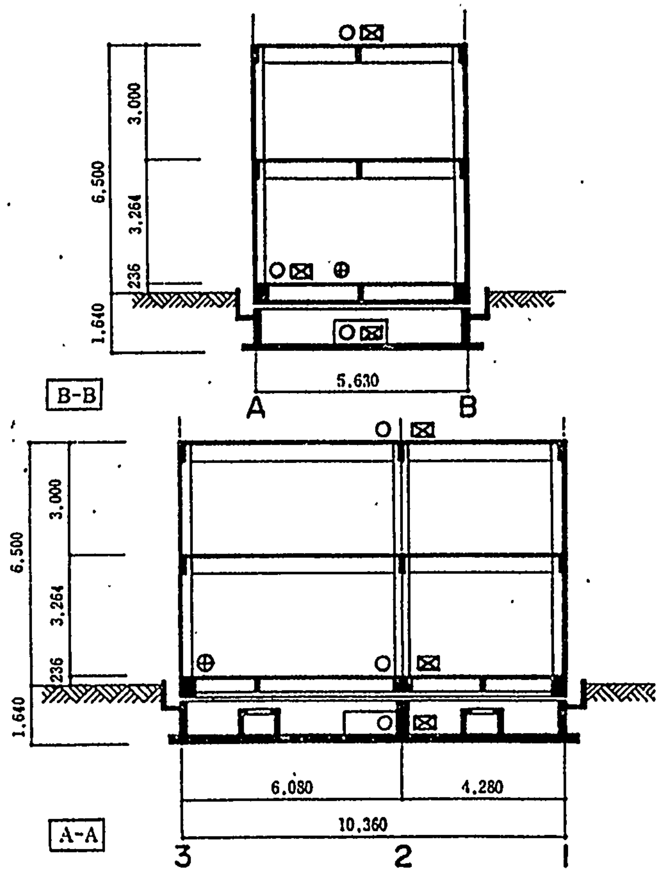


Figure 3 Cross Sections of Building

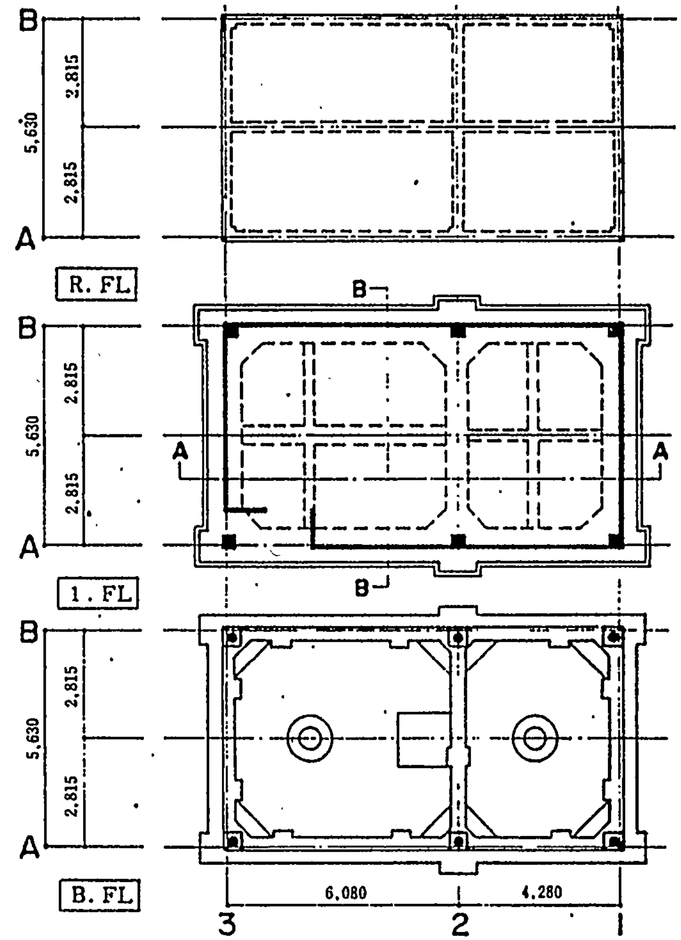


Figure 4 Building Plan and Bearing Layout

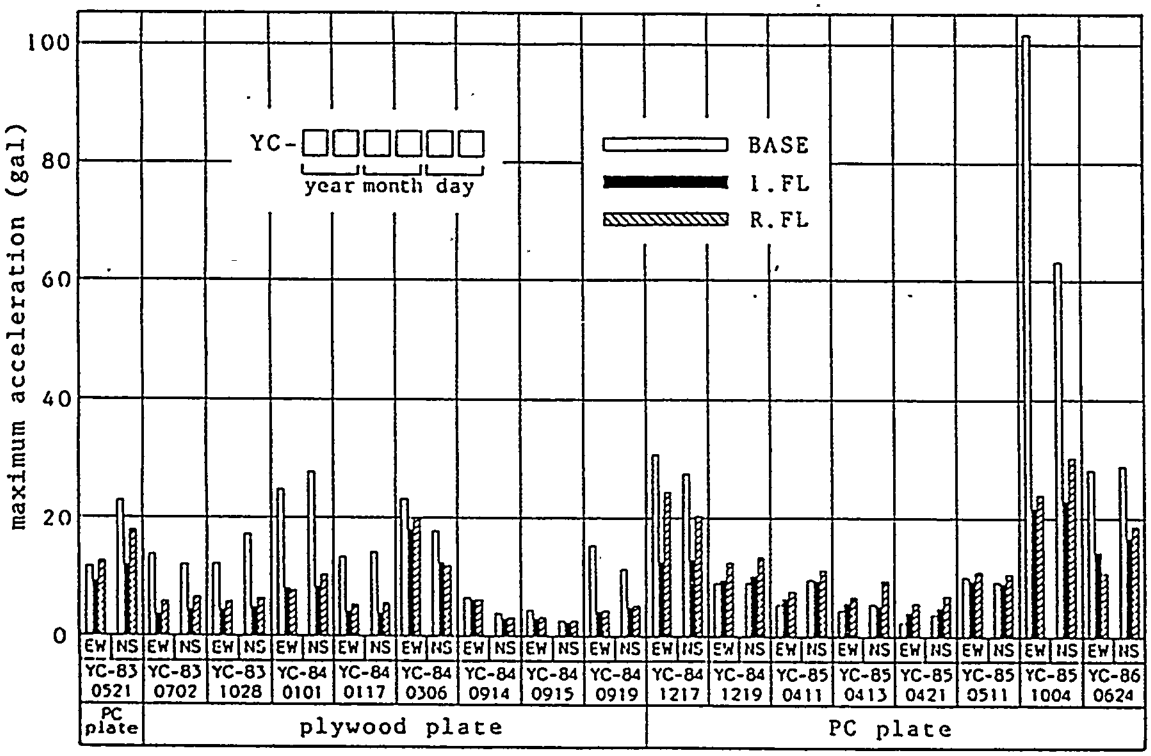


Figure 5 Peak Acceleration Distribution at the Yachiyo City Building

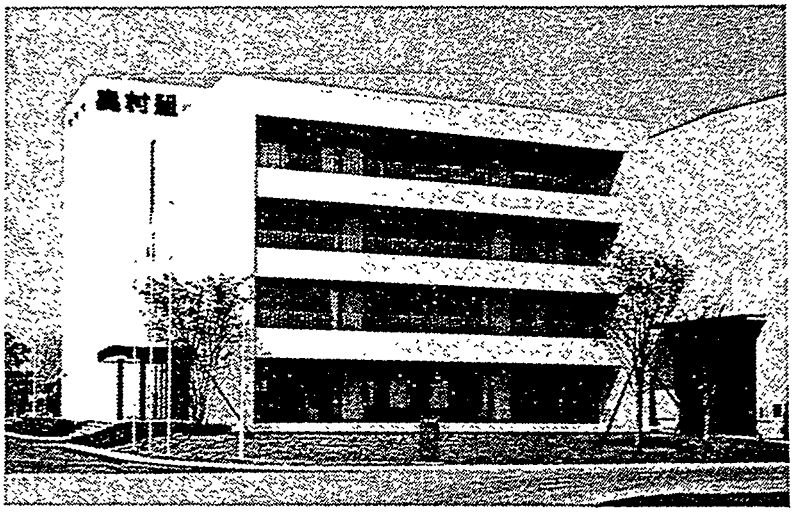


Figure 6 Okumura Company Demonstration Building in Tsukuba Science City

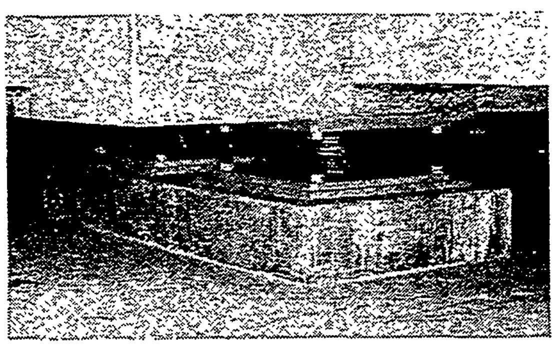


Figure 7 Isolators in Okumura Building

buildings and are used by the companies as offices and laboratories. These buildings were built to test the feasibility of the isolation systems, to observe their responses to earthquakes, and to demonstrate the technology to potential clients.

The first of these larger buildings to be completed was the Okumura Construction Co. Technical Center in Tsukuba City, Ibaraki Prefecture. The building is a four-story reinforced concrete building 20 m in length and 15 m wide by 14 m high. The isolation system is comprised of natural rubber bearings provided by Showa-Densen-Denran, and a number of steel elasto-plastic coil type dampers similar to those used in the Yachiyo City house. Photographs of the these bearings and dampers are shown in Figs. 6-10. The bearings are 500 mm in diameter by 140 mm high. A total of 25 bearings were used in the building. Twelve of the steel dampers are included in the system.

The building is used as an office building by the Okumura Corporation Research Institute. It has a total floor area of 1,330 m^2 and a total weight of 2,250 tonnes. A test program on the building has been jointly carried out by the Okumura Corporation and the Central Research Institute of the Electric Power Industry (CRIEPI). Forced- and free-vibrations tests were carried out on the building and static tests were carried out on the elements and the building. The building was extensively instrumented with seismometers and its response to earthquakes has been observed as part of this experimental program. The results of the various studies have been reported by Abe et al. [3] and Aoyagi et al. [4].

At small strains, the bearings and dampers provide a period of about 1.1 seconds with a damping ratio of 2.5%. The dampers are elastic up to a displacement of about 3 cm and provide no damping for the displacements below this level. The predicted maximum damping which can be provided by the dampers under large excursions is about 18%, at which stage the period lengthens to about 1.8 seconds. The building has responded well in earthquakes. Twenty-six events were recorded in 1987, with the maximum peak-ground acceleration measuring 20% g, while the measured peak amplification of the roof was 2.0% g, a remarkable attenuation of response.

The Oiles Technical Center (Fig. 15) is a five-story reinforced concrete frame building, 30 m x 30 m in plan (Fig. 16), with column grids at 6.0 m and 9.0 m centers. A vertical section of the building is shown in Fig. 17. As a technical center, the building houses the Departments of Research, Technical Development, Quality Assurance, Technical Information, and Mechanical Development. The Oiles Patent Office and Computer Center are also located in the building.

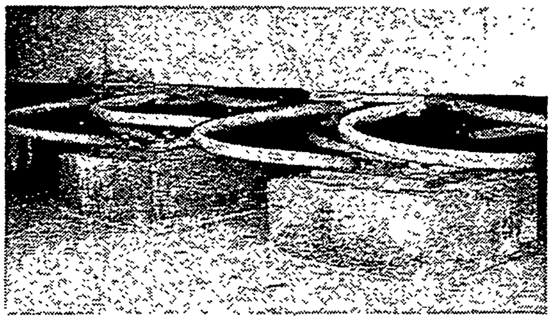


Figure 8 Coil Type Elasto-Plastic Steel Dampers

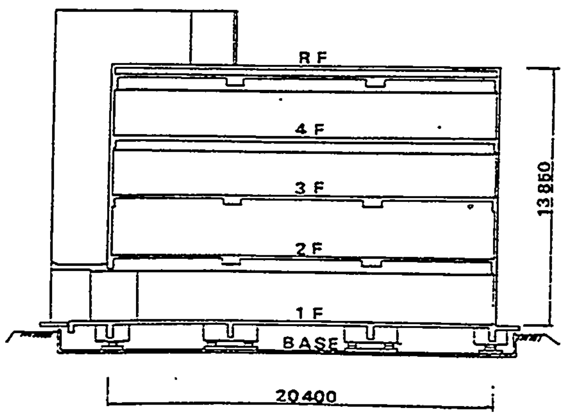


Figure 9 Section through Building

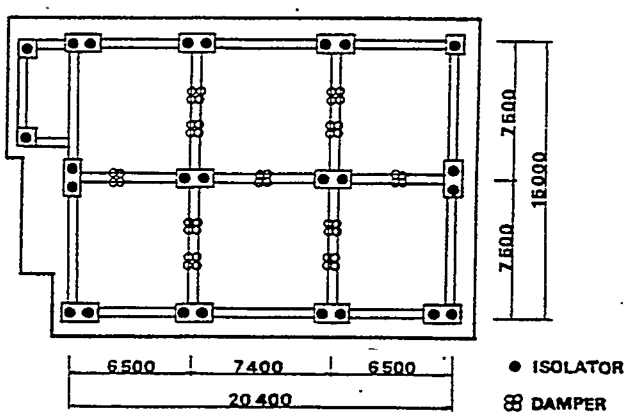


Figure 10 Plan of Base-Isolated Building Showing Layout of Isolators and Dampers

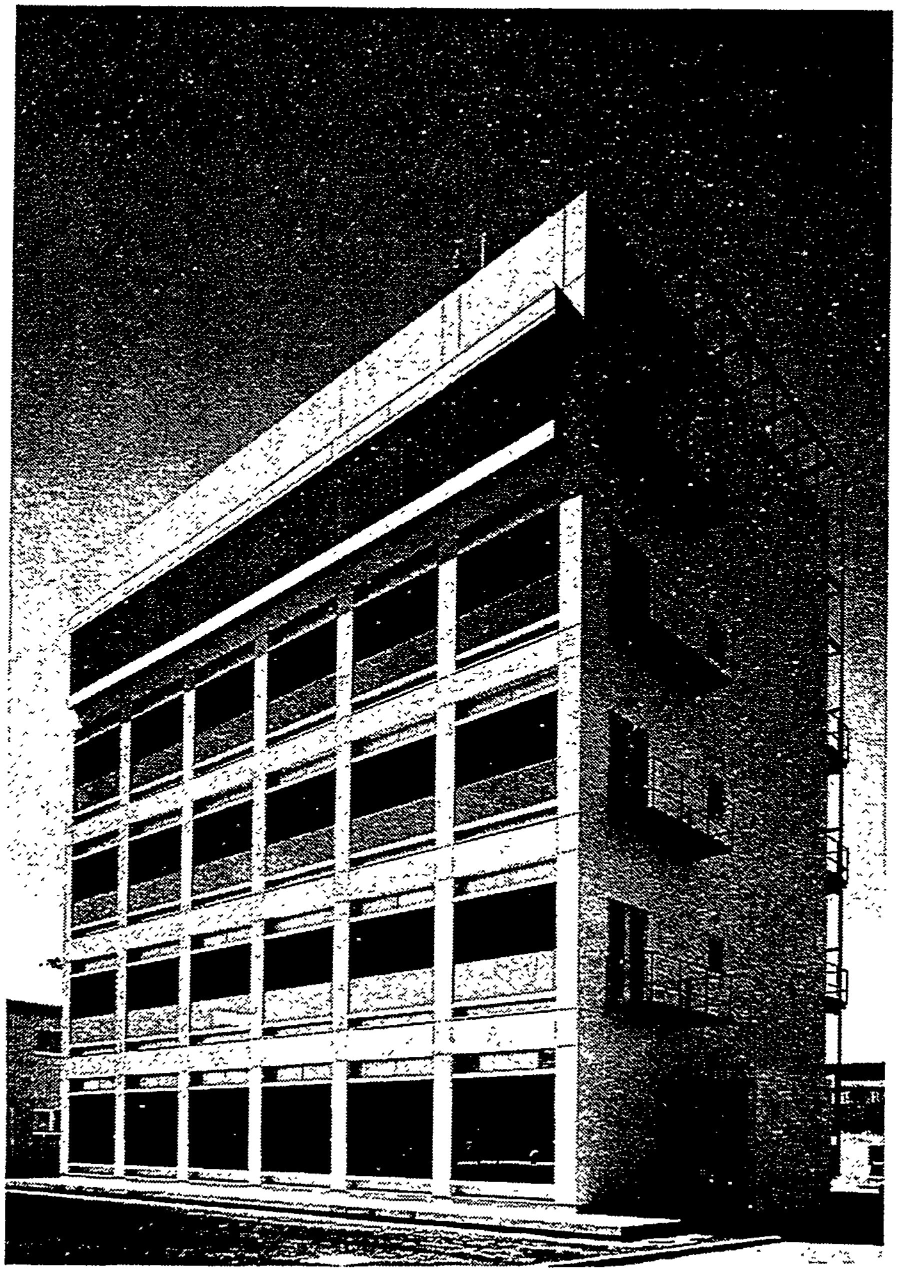


Figure 11 Ohbayashi-Gumi Ltd. Demonstration Base-Isolated Building (High Tech Research and Development Center)

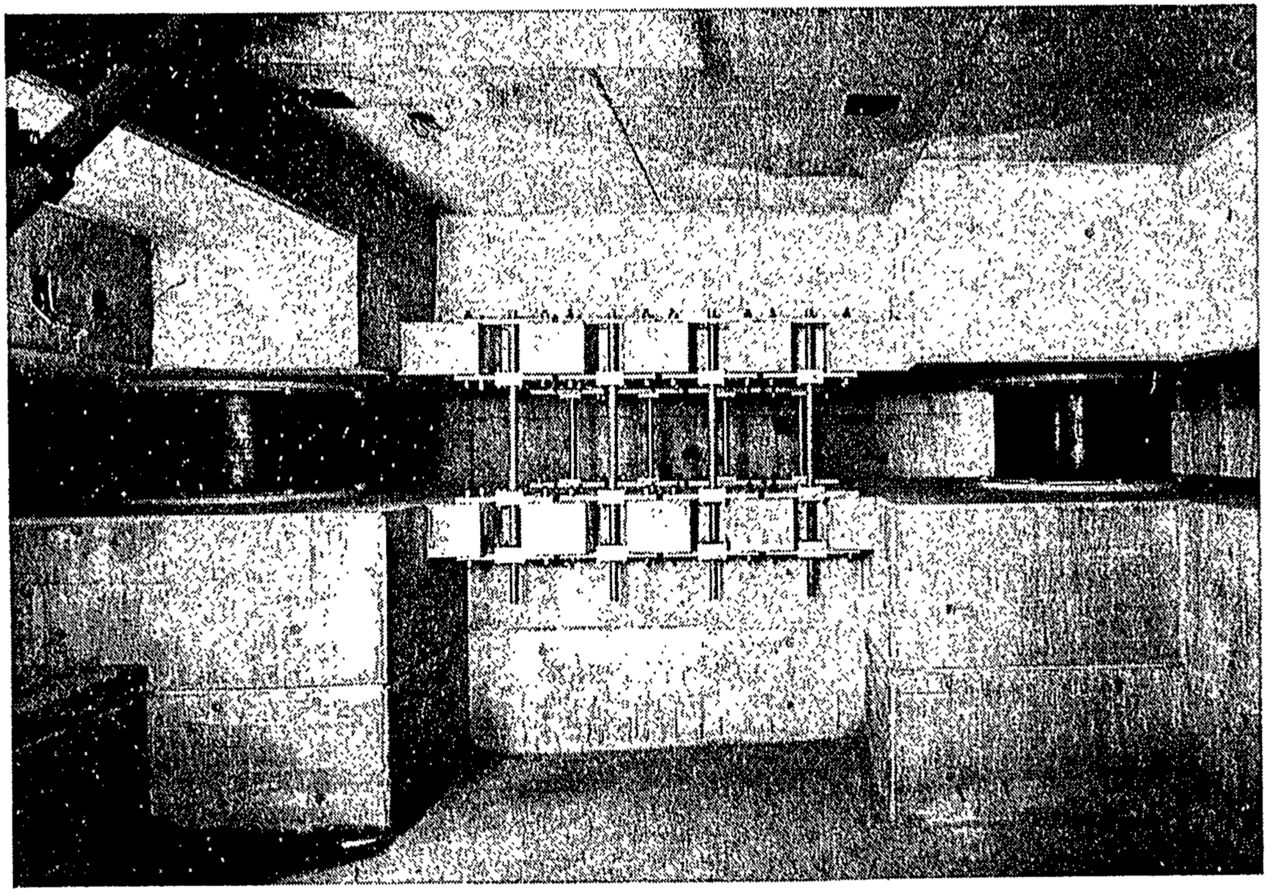


Figure 12 Isolators and Cantilever Type Elasto-Plastic Steel Dampers

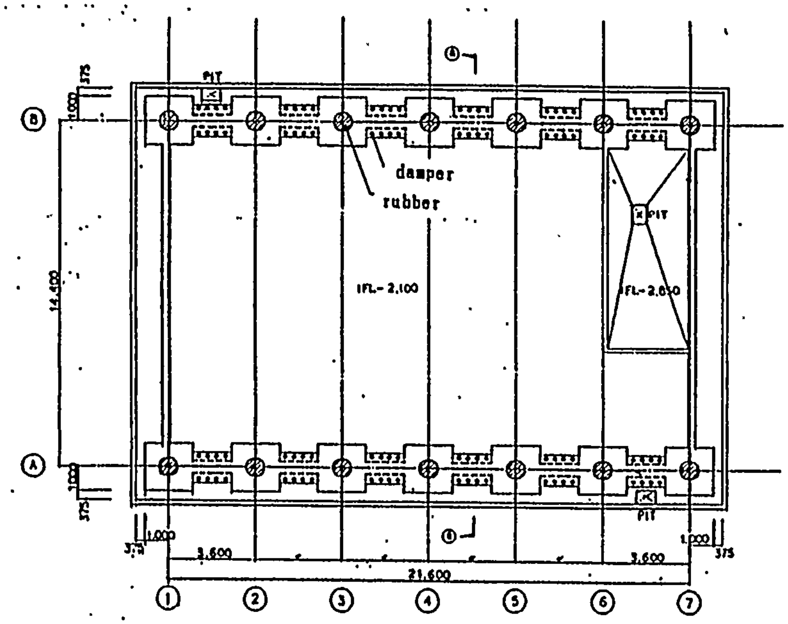


Figure 13 Plan of Building—Layout of Isolators and Dampers

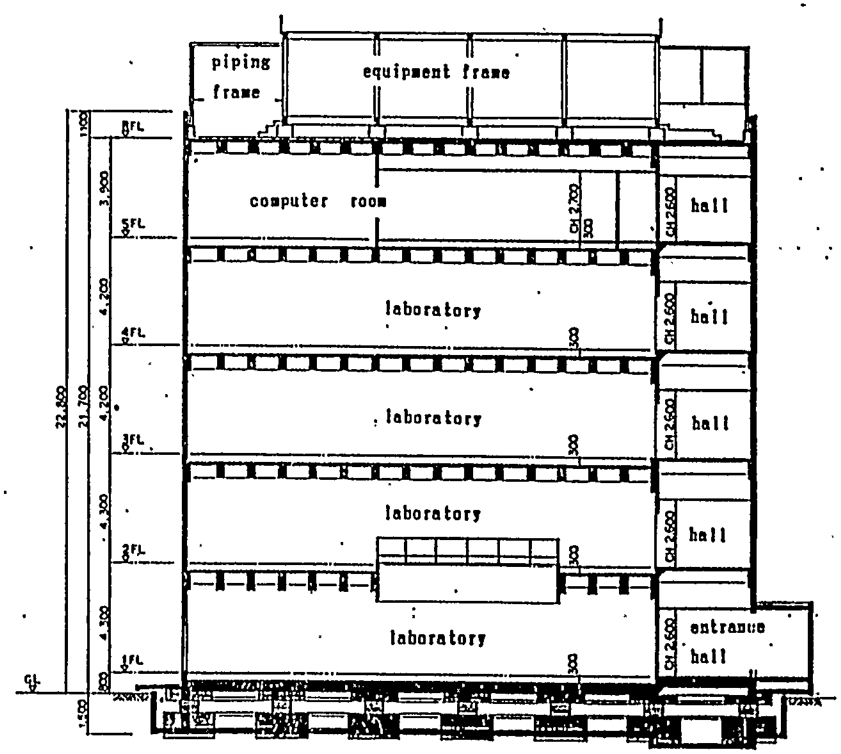


Figure 14 Section through Building

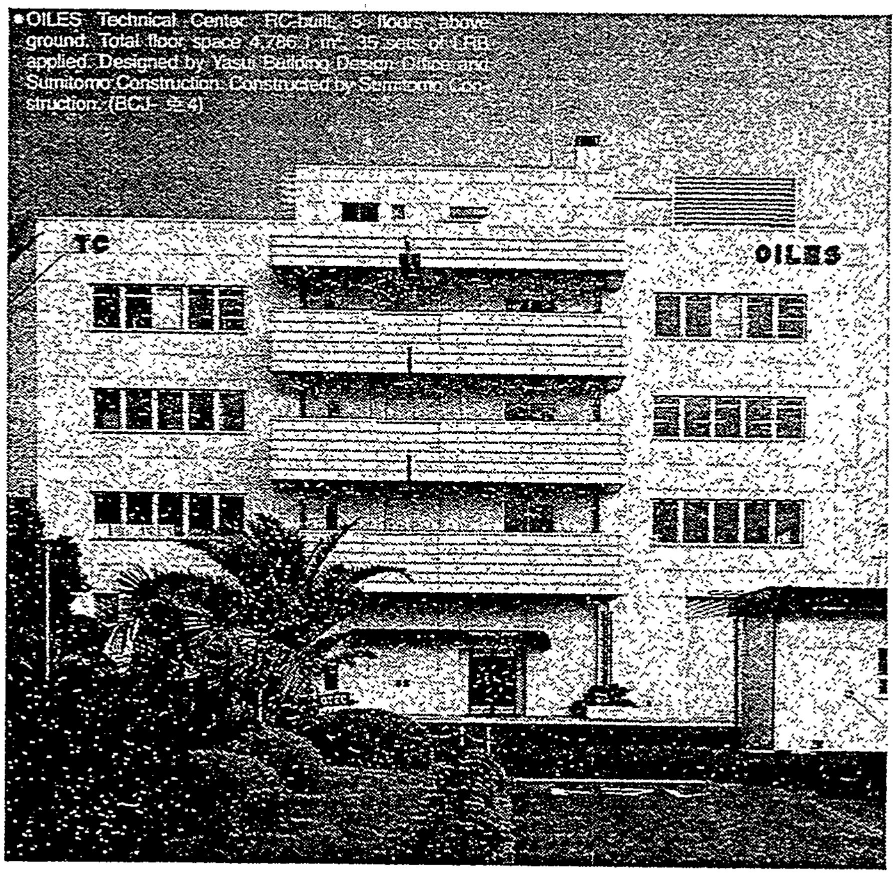


Figure 15 Oiles Industries Ltd. Demonstration Building in Fujisawa City

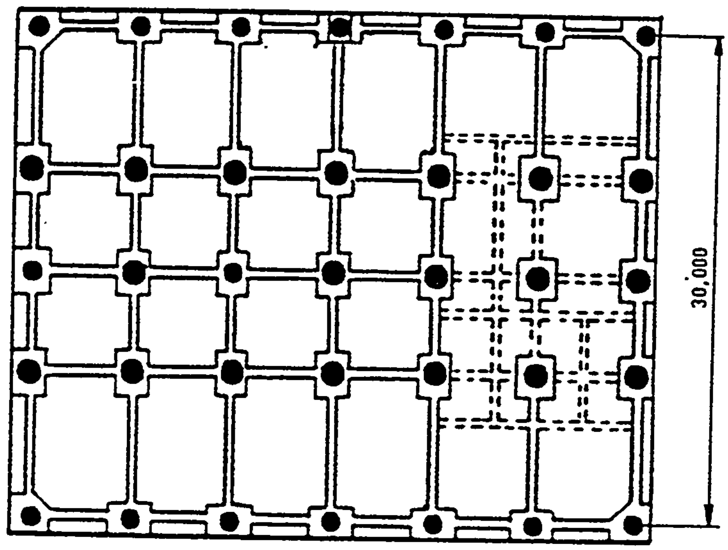


Figure 16 Plan of Building Showing Location of Isolators

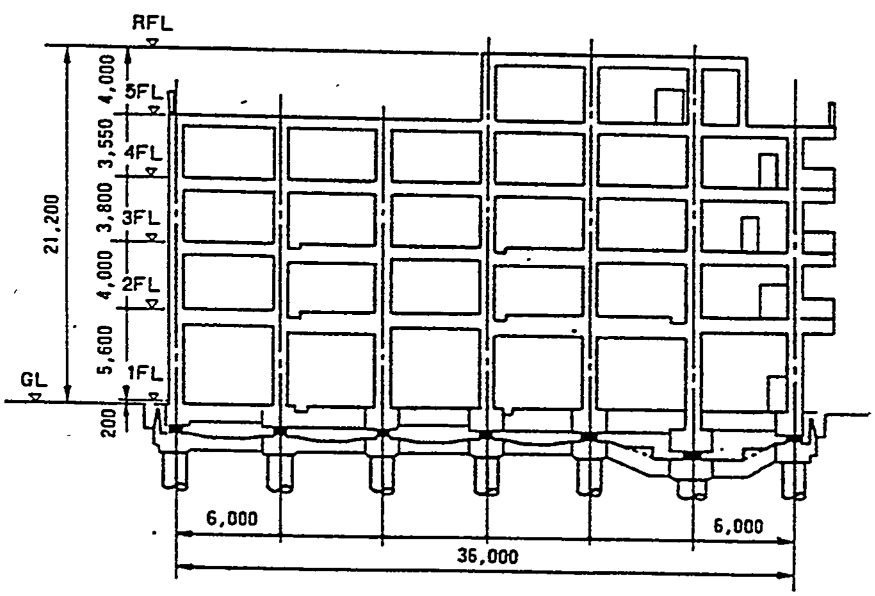


Figure 17 Section through Building

Construction began in April 1988 and was completed in February 1987. It has a floor area of 4,800 m^2 and a total weight of 7,500 tonnes above the isolators. Thirty-five lead-filled elastomeric isolators support the structure with diameters ranging from 650 mm to 800 mm. The lead plugs vary from 130 mm to 160 mm in diameter. All bearings are 363 mm in height.

The building was designed with a base shear coefficient of 0.2, the same as that required for a conventional building. Therefore, no economy was possible in the design and the rationale for construction was improved seismic safety, damage-free performance in major earthquakes, and its use as a demonstration building. For this last purpose, the building serves three useful functions. First, it is used to show building owners, architects, and structural and building service engineers, typical details and configuration necessary to construct and maintain a base-isolated building. Second, it is used as a full-scale isolation test laboratory. Both forced and free-vibration tests have been conducted on the building and these are repeated from time to time to check the durability and reliability of the isolation system. Useful data on the recovery of hysteretic damping after free-vibration testing have also been obtained. Third, the building is instrumented with 27 strong motion accelerographs, 8 displacement transducers, and 9 stress sensors in the piles. Records of structural response have been obtained for several earthquakes [6]. While most of these have been low-level events for which the hysteretic damping have barely yielded, measurable attenuation in floor accelerations has been demonstrated and the design assumptions have been confirmed. Figure 18 illustrates acceleration records from the building the during east Tokyo earthquake of March 18, 1988. The measured peak-ground acceleration was 6.2% g and at the roof, 4.3% g. The relative displacement at the isolation system for this input was just under 1 cm.

A laboratory building at the research institute of the Kajima Corporation was completed at about the same time as the Oiles Technical Center. This structure, the Environmental Engineering Laboratory located near Tokyo, is a functional building that doubles as a full-scale experimental model for advanced construction technology. The facility consists of two adjacent buildings, an acoustic/environmental vibration laboratory, and a thermal control/air flow/equipment laboratory. The two buildings are connected by a corridor and an atrium roof (Fig. 19). The thermal control laboratory was constructed using conventional precast concrete technology and is supported on a standard concrete foundation. The acoustic laboratory is of similar construction but is supported on a base isolation system consisting of 18 laminated rubber bearings, 14 hysteretic dampers, and an unknown number of oil buffers (energy-absorbing devices)

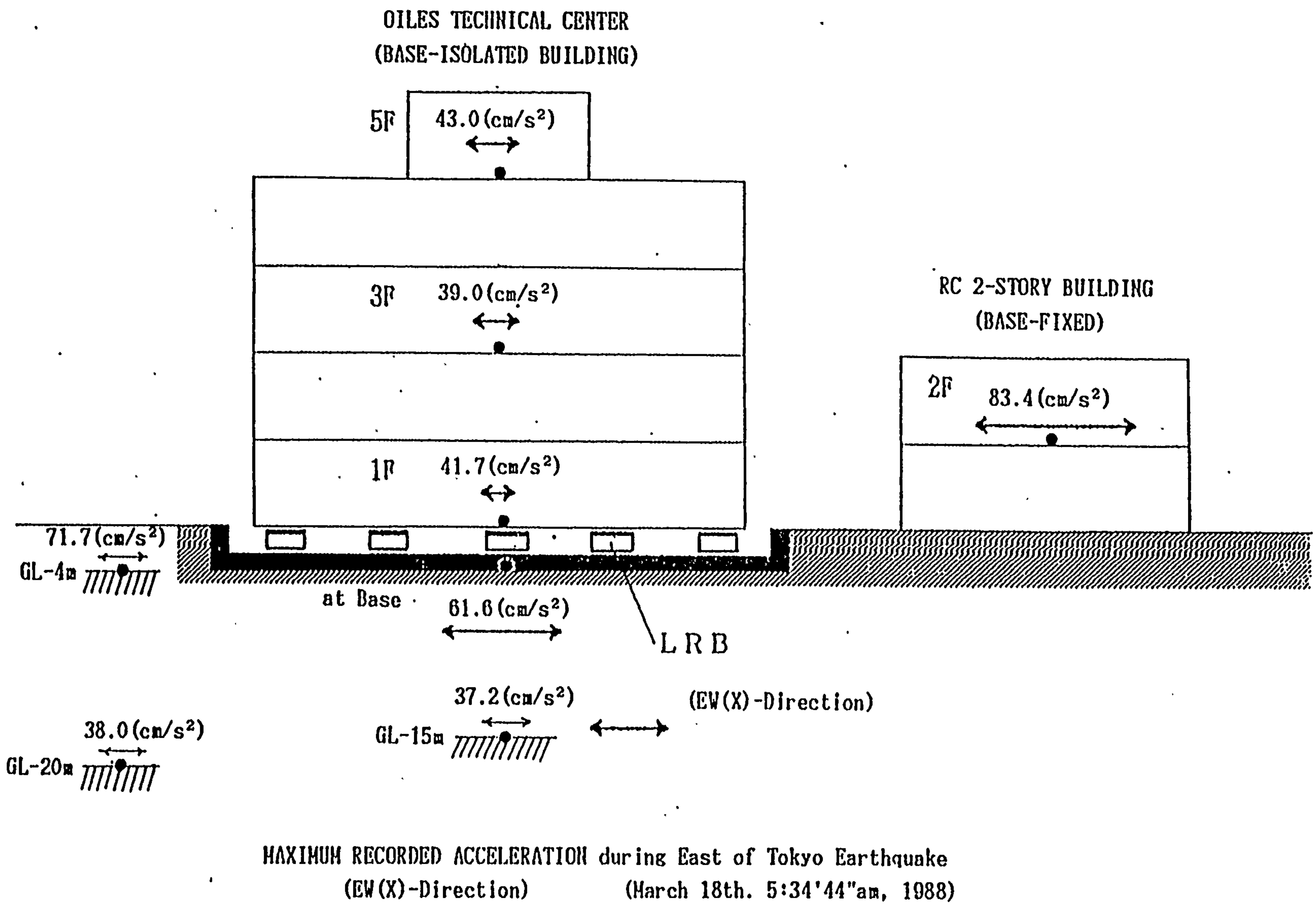
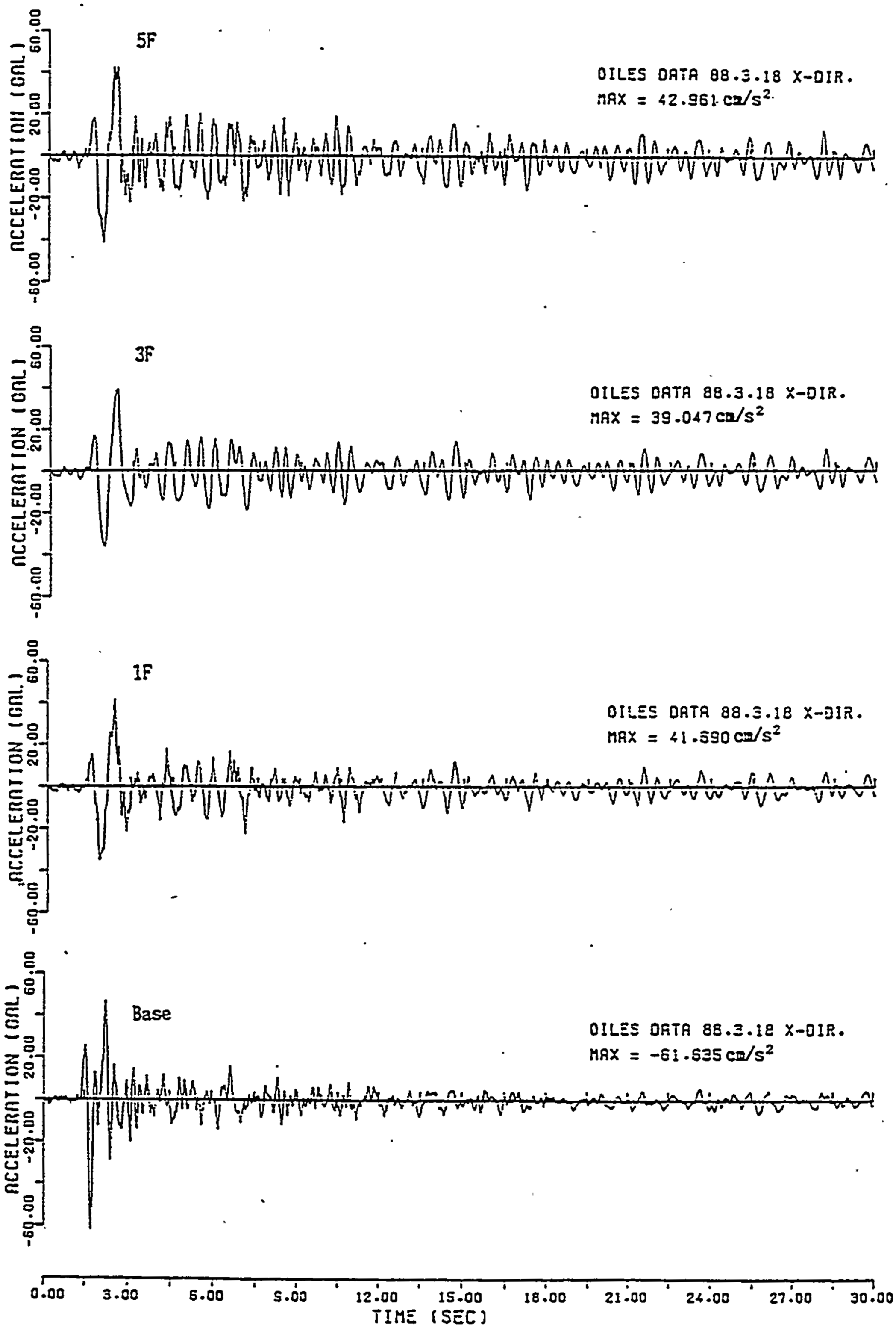


Figure 18(a) Location of Strong-Motion Accelerographs and Maximum Recorded Accelerations during East Tokyo Earthquake, March 18, 1988



Observed Acceleration of OILES TC Building(X-Direction) 1988.3.18

Figure 18(b) Observed Time Histories during East Tokyo Earthquake, March 18, 1988

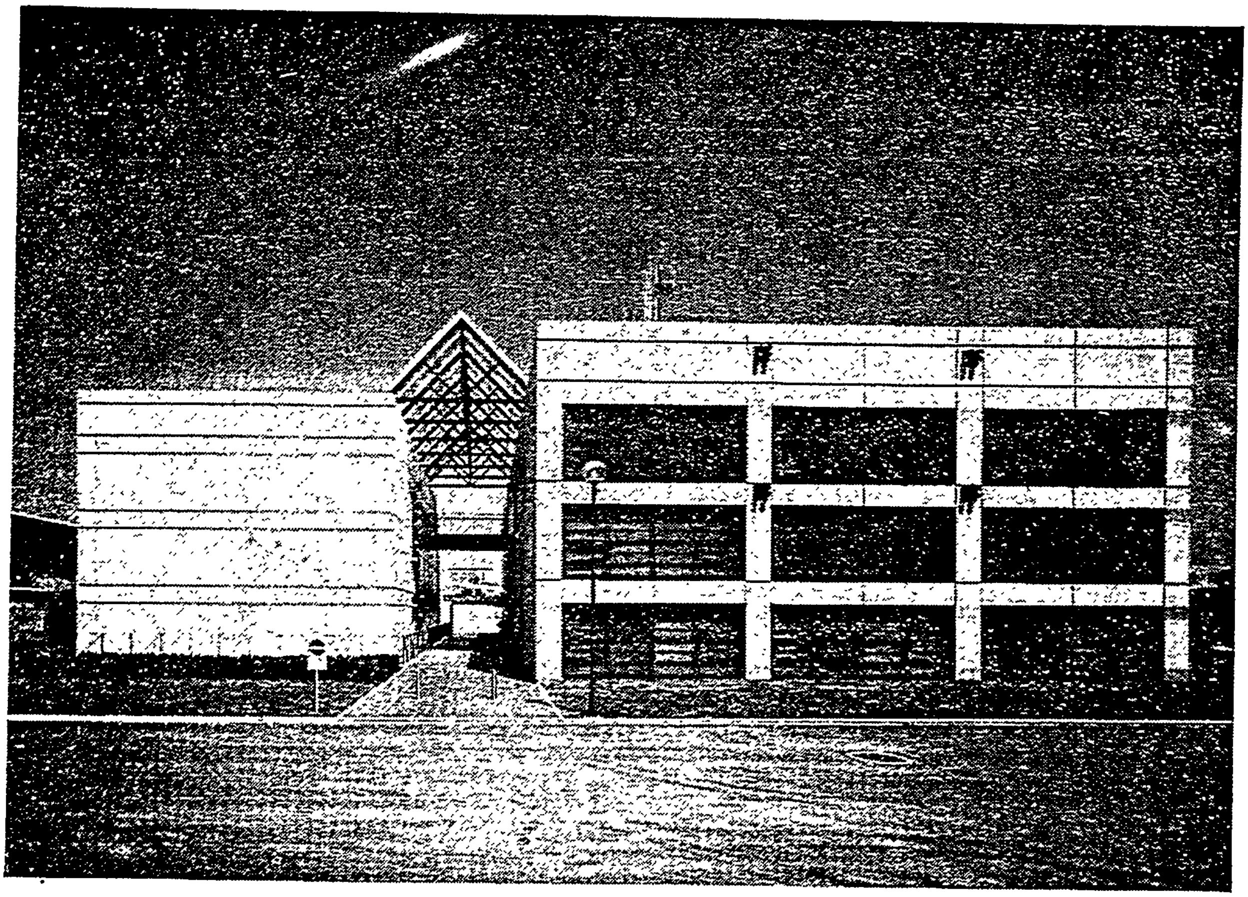


Figure 19 Kajima Corporation Demonstration Building
(Environmental Engineering Laboratory)

as shown in Fig. 20. The foundations and the base floors of the two building, therefore, differ fundamentally. Figure 21 shows the location of the various supports and isolation devices used for the acoustic laboratory and Fig. 22 shows a section through the building.

The structure represents a full-scale model of an operational laboratory designed for earthquake motion isolation. The building is supported on a foundation that provides a complete mechanism for isolation against earthquake motions and other ambient vibrations. Acoustic and vibration experiments can be performed inside the isolated structure with a high degree of accuracy.

Soft vertical rubber springs are utilized to isolate the building and filter out micro-tremors caused by external ground-transmitted vibrations. Fail-safe blocks, which limit ultimate system displacements, are used to provide secondary protection against primary system failure. The horizontal frequency of the building is 0.5 Hz and the vertical frequency at the design weight is 5 Hz. This low vertical frequency is achieved by thick layers of soft natural rubber. The bearing shown in Fig. 23 has shape factor of 5.2 and consists of 5 rubber layers, each 48 mm thick and 4 steel insert plates, each 5 mm thick. The bearing has a design load of 3.1 MPa and a design shear displacement of 20 cm.

The thick-layered laminated bearings do not provide sufficient energy absorption and dampers are incorporated to increase overall system damping. The hysteretic damper and concrete deformation retainer are shown in Fig. 24. The steel rod at the center dissipates horizontal vibration energy through inelastic flexural deformation. The deformation retainer protects the steel rod from damage at the fixed end and improves the fatigue strength by a factor of 4. According to a Kajima spokesperson, the shape of the deformation restrainer was determined through trail and error.

Oil buffers are incorporated to reduce vertical vibrations and rocking motions during an earthquake. Figure 25 illustrates an oil buffer damping device. Analytical methods have been used to demonstrate that vertical response can be reduced by over 50% with these buffers. The reaction force of the oil buffer is proportional to its velocity.

Two ancillary components are included in addition to the primary isolation components. Fail-safe blocks are used adjacent to the rubber bearings to provide secondary protection against failure of the primary system. The block shown in Fig. 26 is provided with a mechanism to adjust clearance between the device and the foundation. Slide bearings (Fig. 27) are used to support the passageway and atrium roof joining the two laboratories.

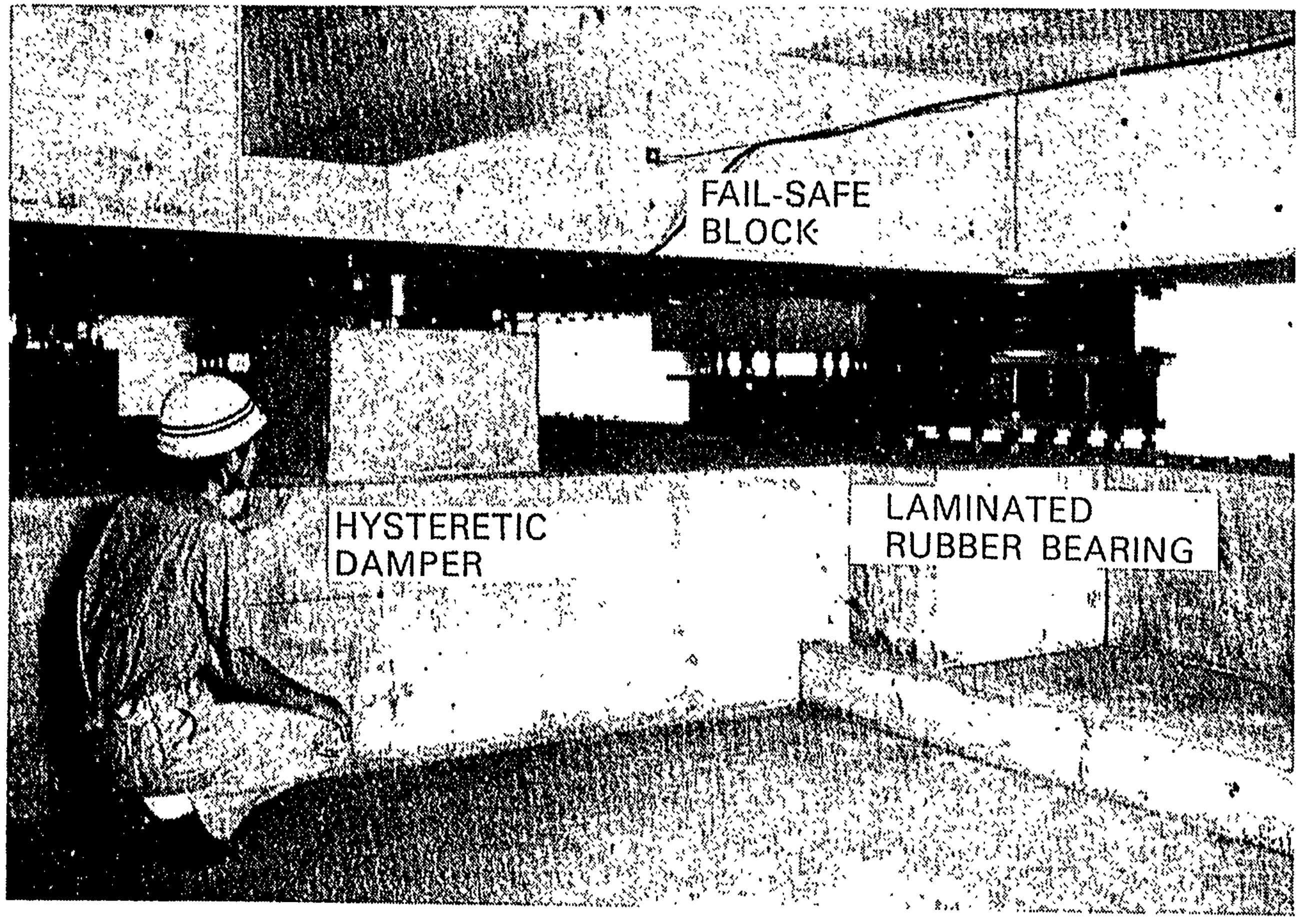


Figure 20 Isolators, Dampers, and Oil Buffers as Installed in the Kajima Building

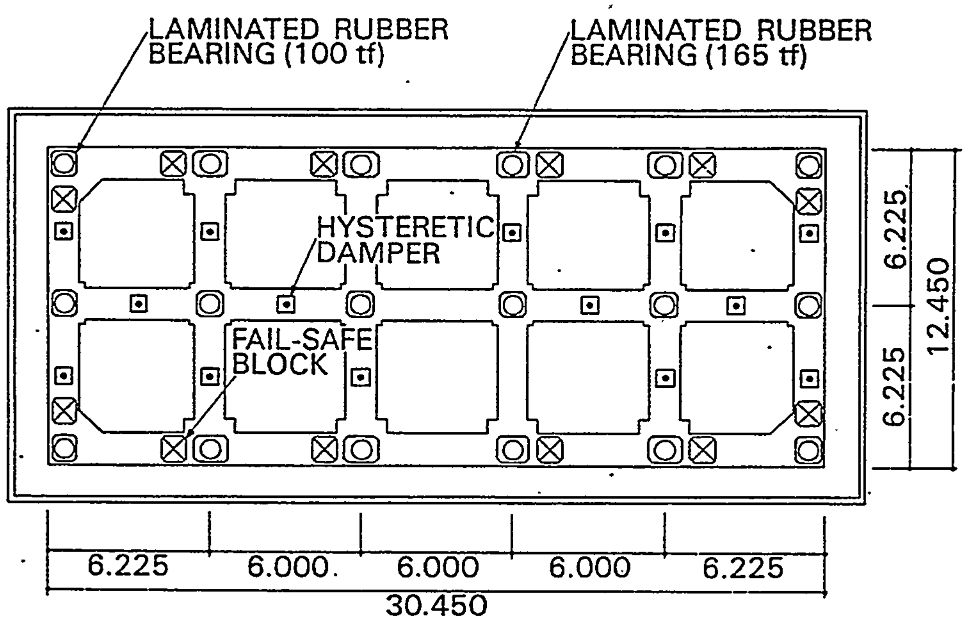


Figure 21 Plan of Kajima Building Showing Layout of Isolation System Components

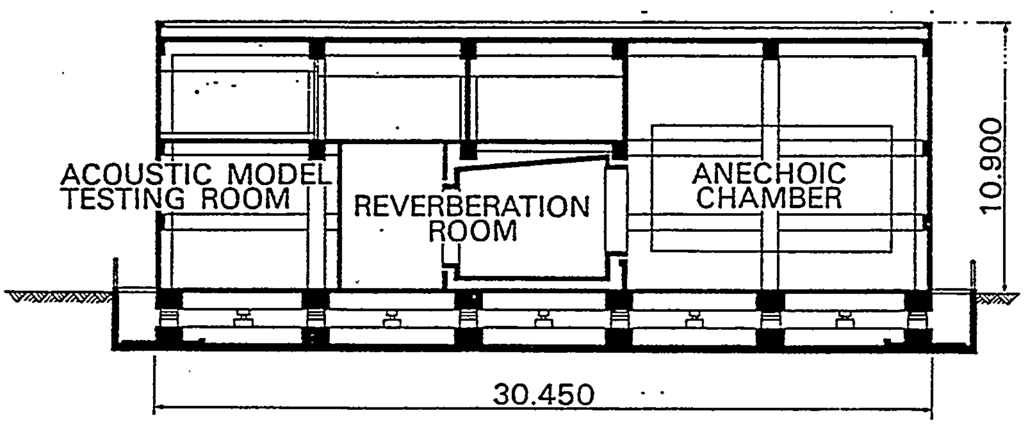


Figure 22 Cross-Section of Kajima Laboratory

165tf RUBBER BEARING

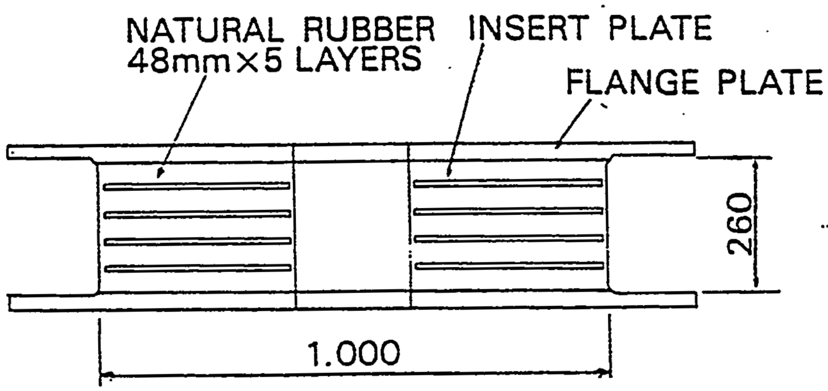


Figure 23 Rubber Bearing in Kajima Building

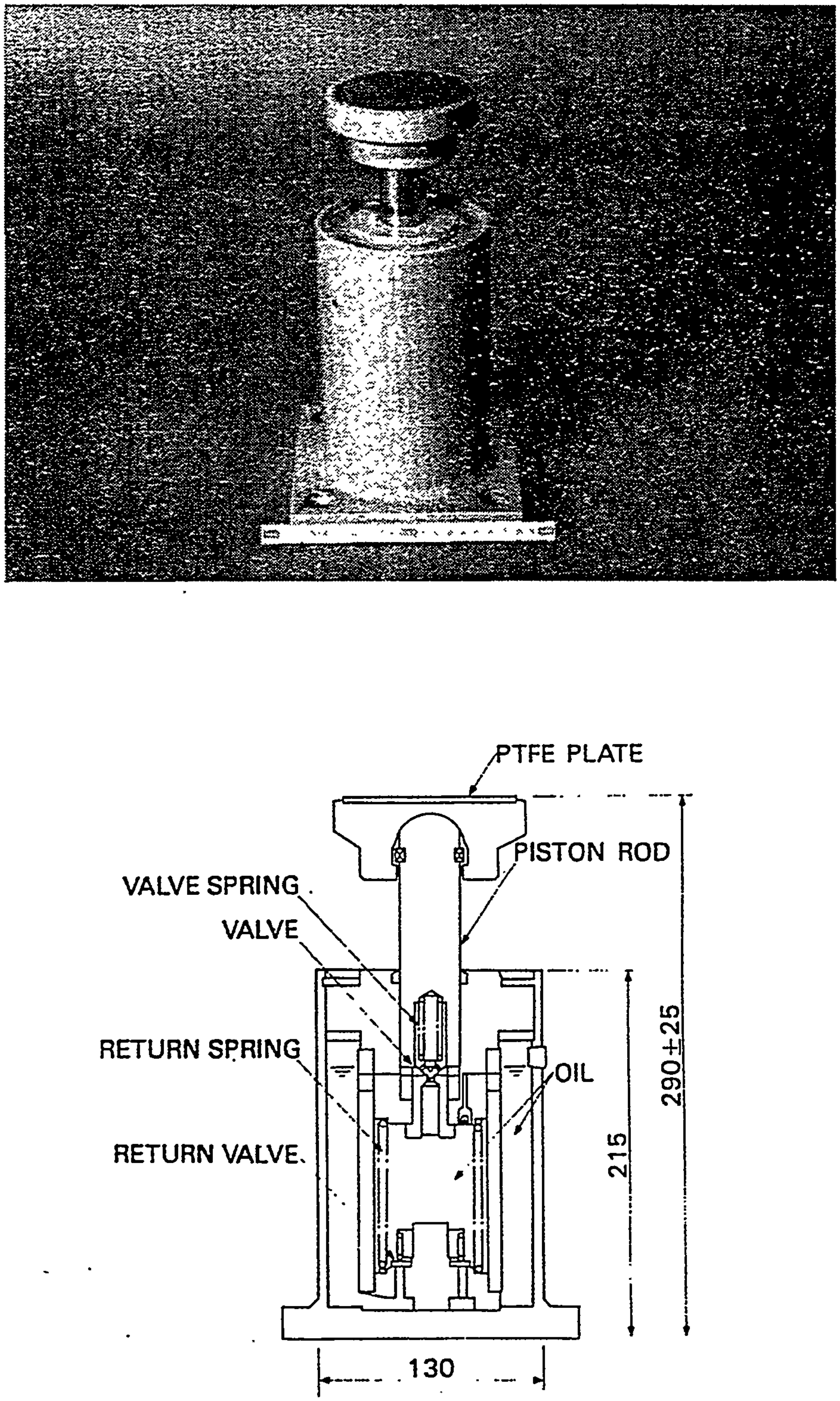
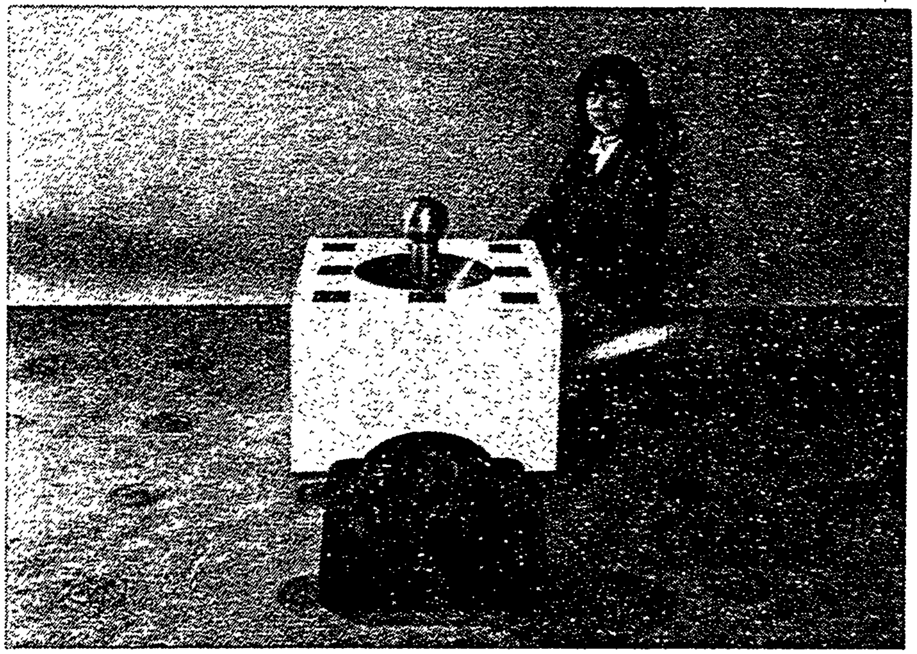


Figure 25 Oil Buffer for Kajima Building



HYSTERETIC DAMPER

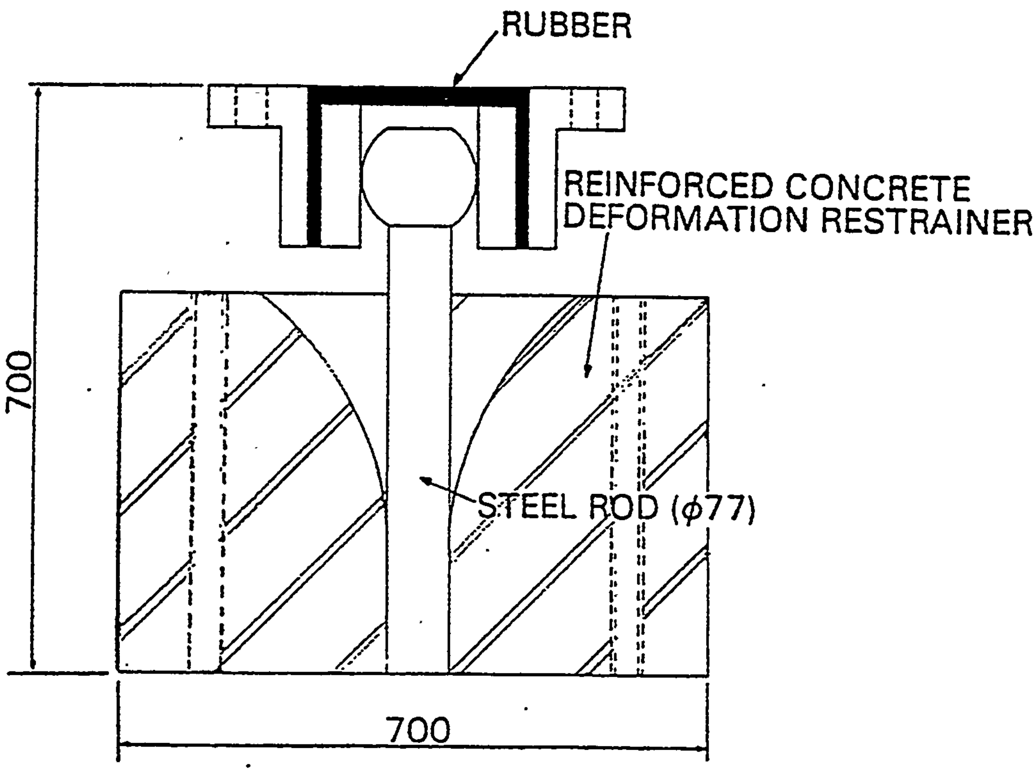


Figure 24 Hysteretic Steel Damper in Kajima Building

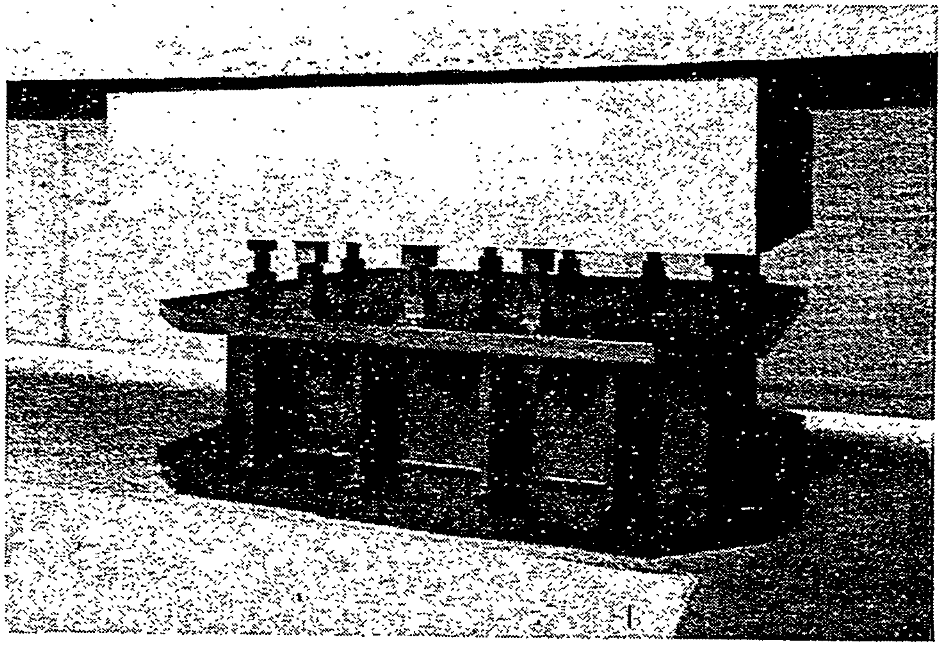


Figure 26 Fail-Safe Concrete Blocks in Kajima Isolation System

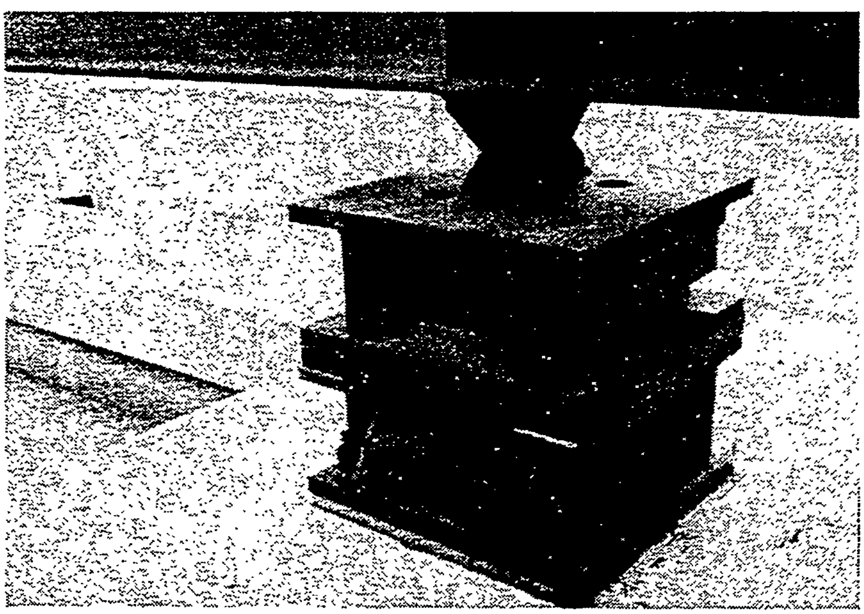


Figure 27 Sliding Bearings for Passageway between Isolated and Fixed Laboratories

Analysis, testing, and recorded earthquake data [7] have demonstrated that the isolation system can reduce peak building accelerations by a factor of 4 to 5. Measurements of micro-tremors have demonstrated 20 db reductions for frequencies greater than 10 Hz. The elastic stiffness of the hysteretic damper acts as a wind brace for the building. The system is designed to limit wind response to less than 1 gal for winds with a one-year return period. The validity of the analytical model predicting wind response has been demonstrated by measurements. Motions of 27 mm and accelerations of 0.41 gal have been recorded with 16m/sec winds.

The Takenaka Construction Company began a study of base isolation systems in 1982 and in 1984 constructed a full-scale test model using a 550-ton coal silo (Figs. 28-30) mounted on natural rubber bearings provided by the Bridgestone Corporation. The damping in the isolators was very low, equivalent to about 2.4% for an isolated period of 2.0 sec, and to provide adequate damping; viscous fluid dampers (Fig. 31) provided by Oiles Industries were included in the system. Free-vibration tests were carried out and the observed damping was about 12% [8]. The performance of the system in the tests was so satisfactory that the company designed and built its own demonstration building using this system. Known as the Funabashi Dormitory (located near the Nishi-Funabashi station about an hour's train ride from Toyko), the building was designed and built as a research facility. The Takenaka Technical Research Laboratory has instrumented the building and is presently collecting performance data.

The building (Fig. 32) is a three-story structure housing quarters for single male employees of the Takenaka Corporation. The structural framework and the lateral load-resisting system consist of shear walls and reinforced concrete frames. The architectural design (Fig. 33) for the facility included an open atrium area on the second floor between two wings of residential rooms. This required that the second and third floors overhang the first.

If conventionally founded, the resulting structural system could lead to poor seismic performance. Base isolation removed this design obstacle. In addition, base isolation allowed a relatively large open area on the first floor to be unencumbered by shear walls that would typically have been required to transmit seismic forces to the foundation.

The foundation and the first floor dimensions are 9 m by 37.2 m, while the second and third floor dimensions, including the overhangs, are 16.2 m by 44.2 m. The total height of the building above the basement is about 11 m. The basement, within which the base isolation system is situated has a clear height of about 1.5 m, not enough clearance to stand comfortably but high enough to enable inspection of the

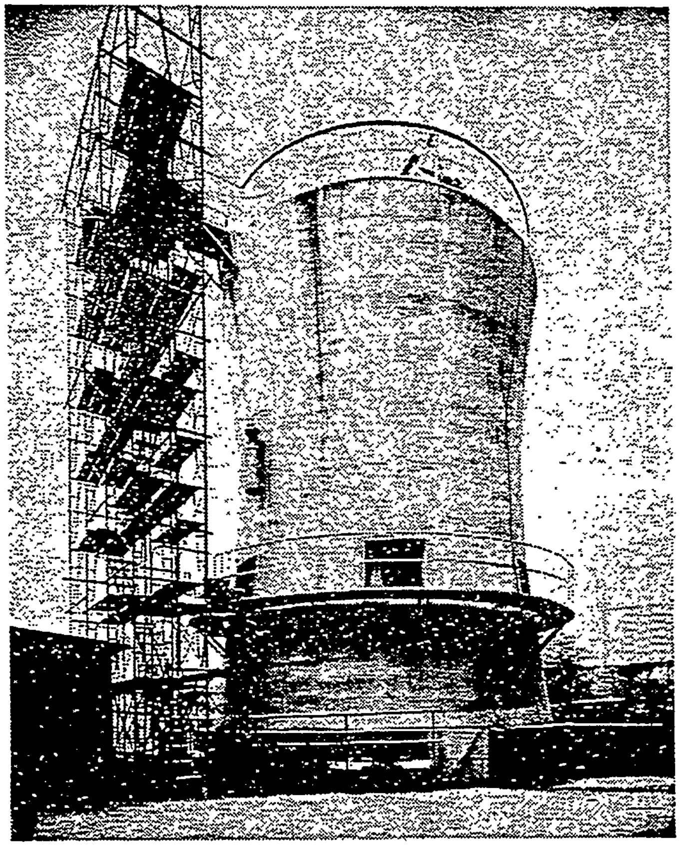


Figure 28 Takenaka Company Isolated Test Model (Coal Silo)

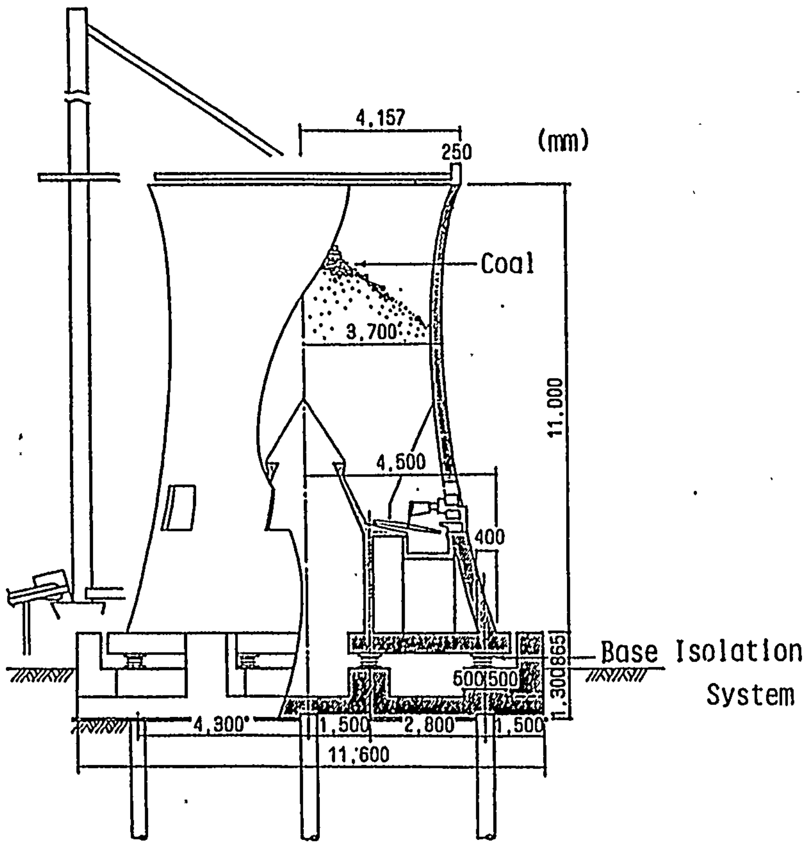


Figure 29 Section through Takenaka Test Model

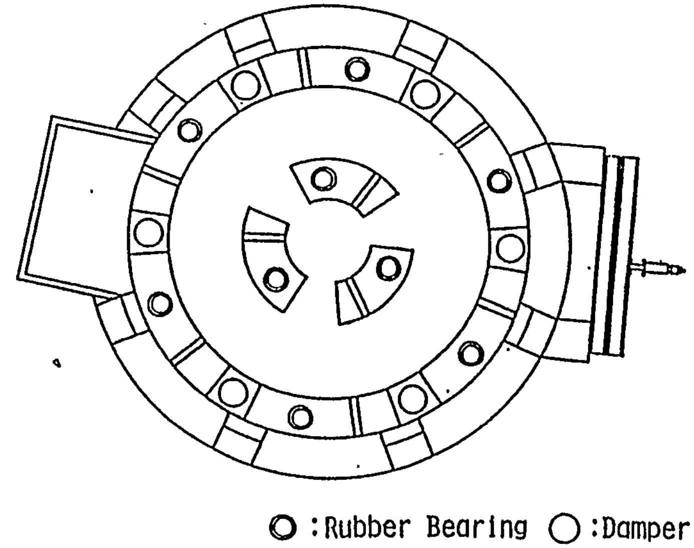


Figure 30 Plan of Test Model Showing Location of Isolators and Dampers

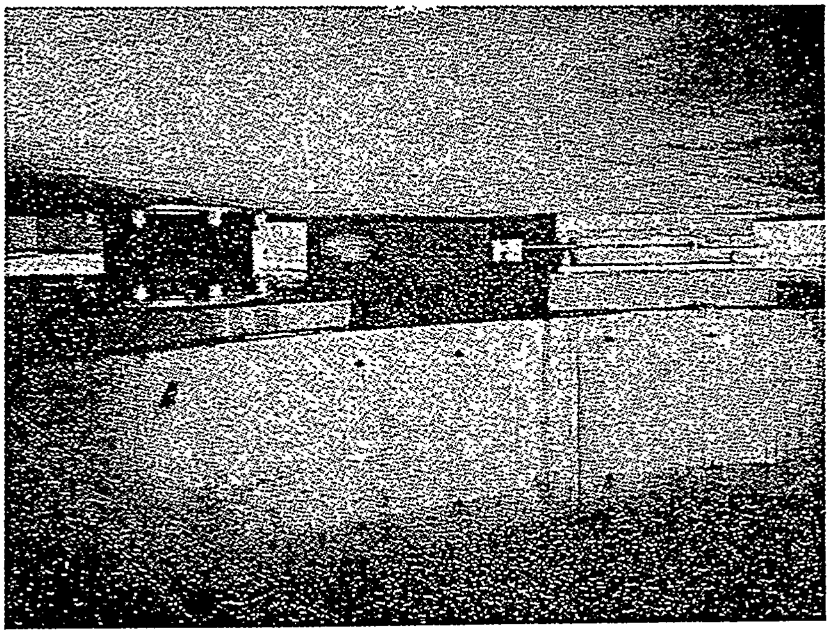


Figure 31 Rubber Bearings and Oil Dampers in Takenaka Test Model

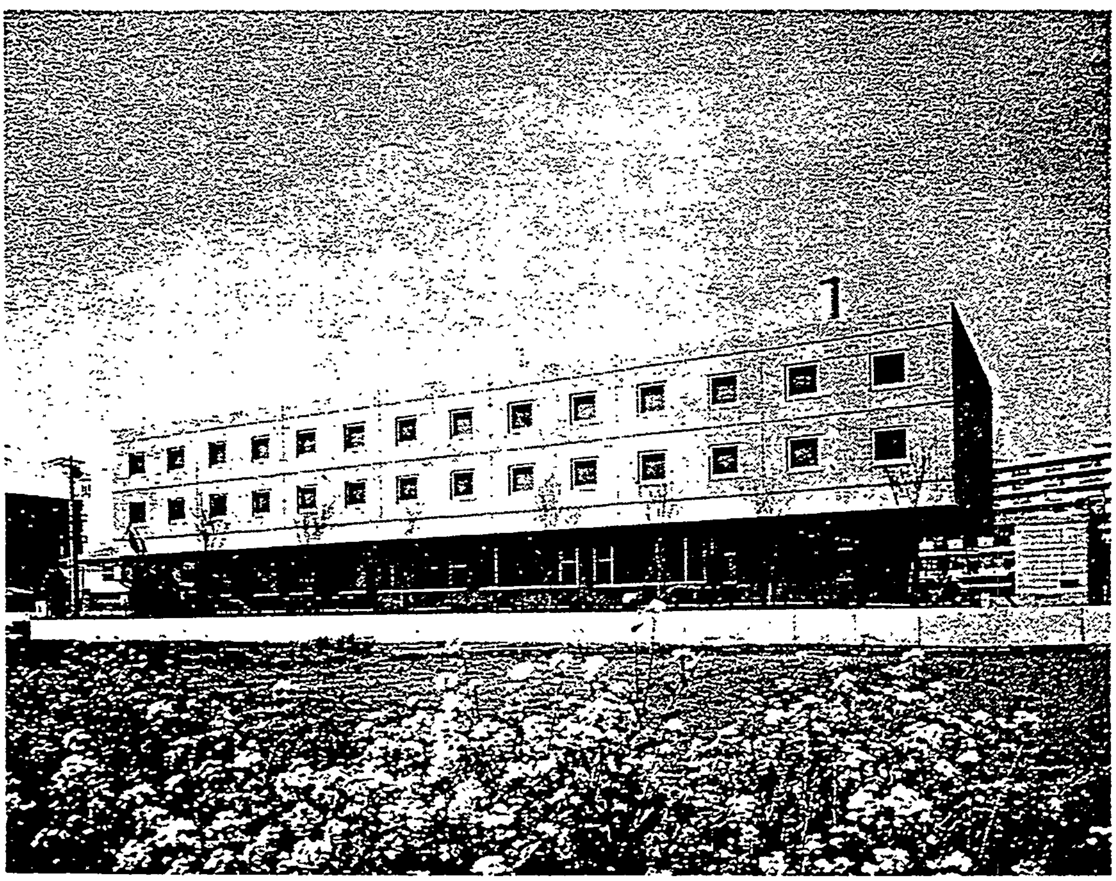


Figure 32 Funabashi Dormitory: Takenaka Company Demonstration Building

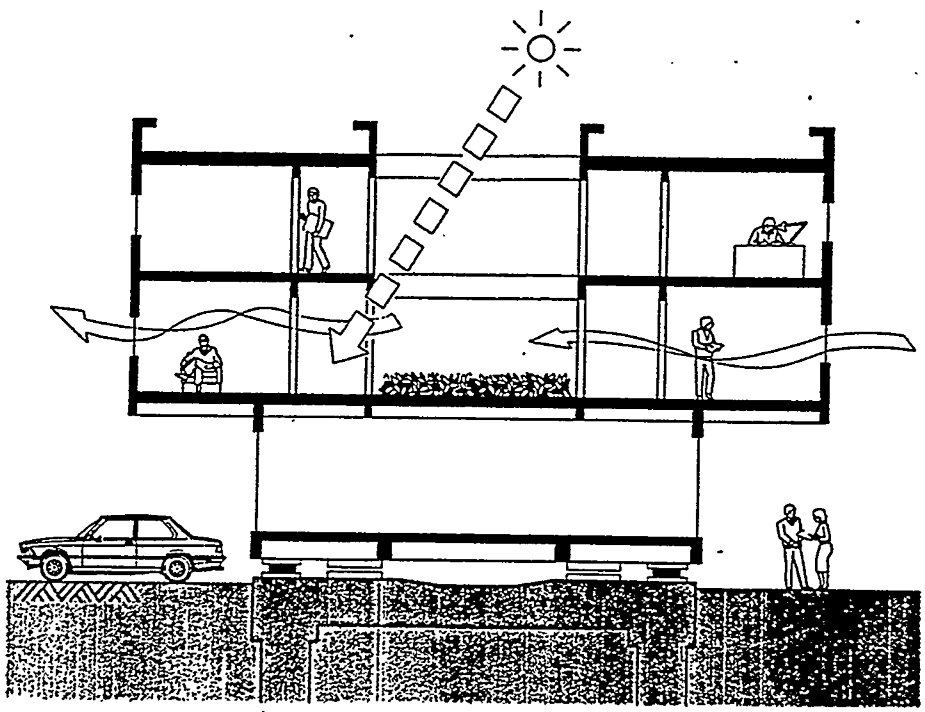


Figure 33 Cross-Section of Funabashi Dormitory
Showing Architectural Design

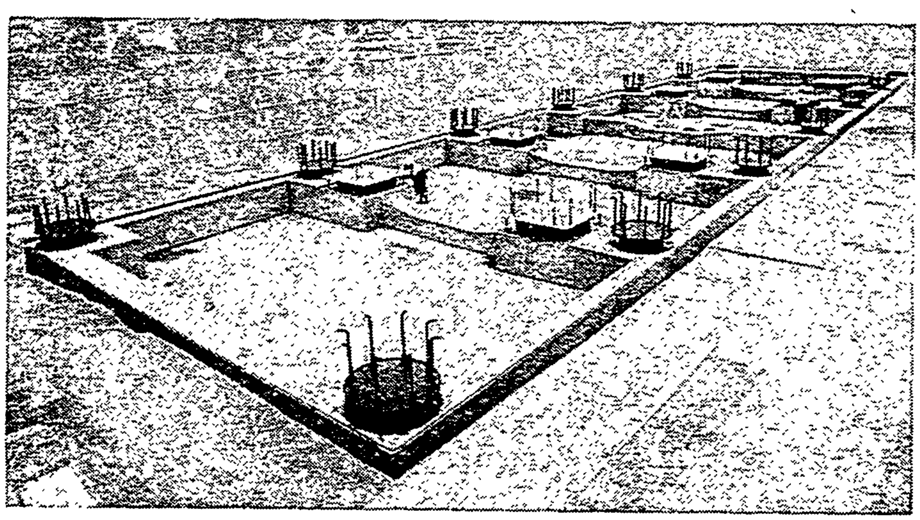


Figure 34 Isolators and Dampers in Funabashi Dormitory

base isolation system.

Poor soil conditions are the site required that the building be supported on piles placed to a depth of about 23 m bearing on a subsoil layer of find sand. A total of 14 piles, one located at each building column, support the entire weight of the structure. The pile caps are tied together by a grid of grade beams. The base isolation bearings and the energy-absorbing devices are located on top of the grade beams.

The base isolation system for the building consists of 14 steel-reinforced natural rubber bearings (Figs. 34 and 35) provided by the Bridgestone Corporation, one at each pile location, supplemented by eight dampers. Of the 14 bearings, 8 carry a vertical load of 150 tonnes, and 6 carry a vertical load of 200 tonnes. The 150-tonne bearings are 686 mm in diameter and 186 mm thick, which the 200-tonne bearings are 766 mm in diameter and 166 mm thick. Thus, the vertical pressure on the bearings is 4.06 MPa and 4.34 MPa, respectively.

The viscous polymer based dampers are unique to this system. Each damper is about 150 cm^2 in plan with a total height of about 30 cm. A viscous polymer is contained within the lower part of the damper. The upper bearing plate of the damping mechanism (see Fig. 36) is fixed to the superstructure and positioned on top of the viscous fluid. The drag of the viscous fluid on the bearing plate as it displaces horizontally during an earthquake produces a damping force proportional to the relative velocity. The building and the base isolation system and its performance are described in Ref. [9].

The rubber bearings in this system serve predominantly to support the building's weight, reduce the fundamental frequency of the base-isolated structures, and to provide a lateral restoring force. The dampers serve to absorb seismic energy and control displacements. While the damping devices do not support vertical load, the viscous damper adds to the horizontal stiffness of the base isolation system. The design shear strain in the bearings is 300%.

The use of base isolation systems for the seismic protection of museums is potentially a very large market. Priceless artifacts have been damaged during severe earthquakes in museums in Greece, Yugoslavia, and Italy, and there are many museums located in highly seismic areas. The first museum to be built on a base isolation system is the Japanese Christian History Museum in Osio, Kanagawa Prefecture. The region is subject to frequent earthquakes and is designated by the government as required positive counter-measures against seismic attack. A base isolation system was adopted for this museum to protect the many exhibits concerning the history and development of Christianity in Japan.

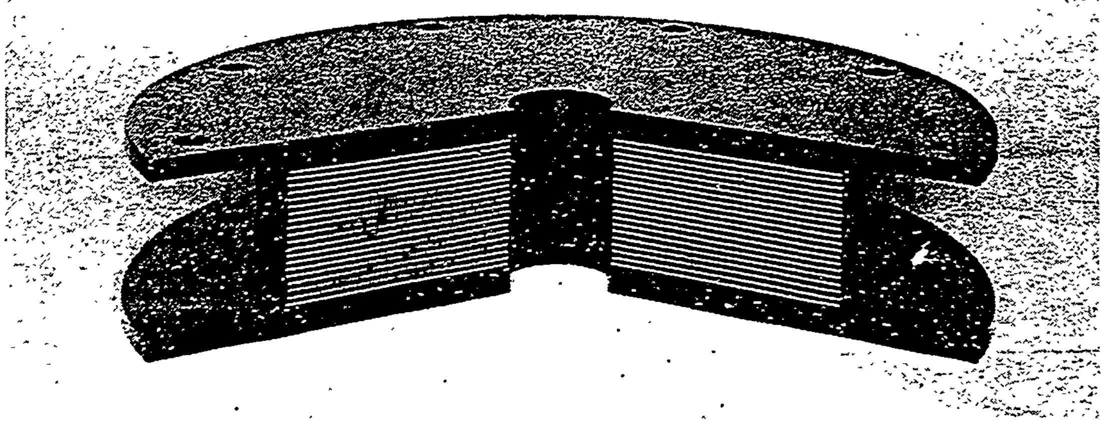


Figure 35 Rubber Bearings in Funabashi Dormitory

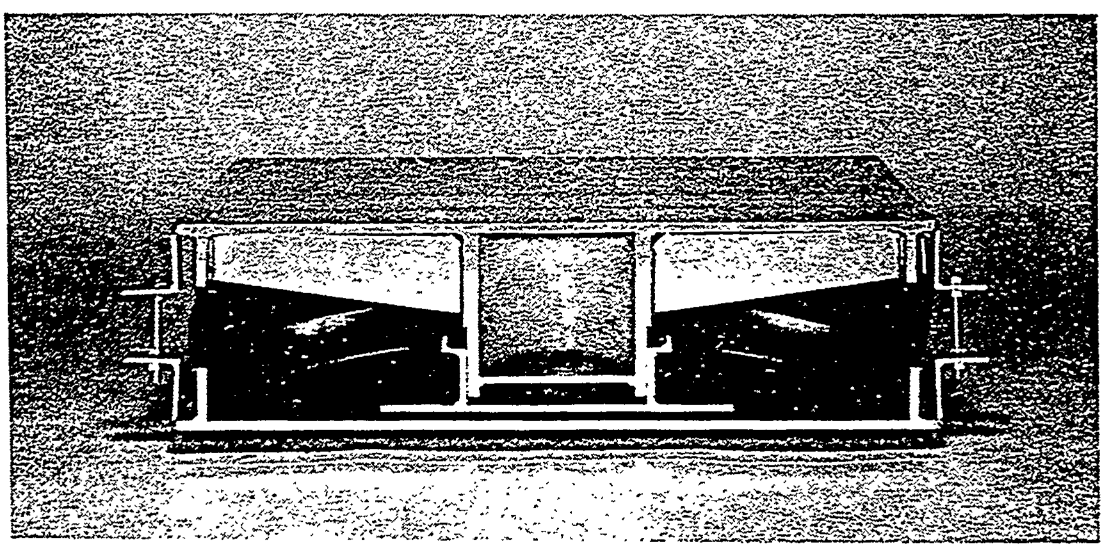


Figure 36 Viscous Dampers Used in Funabashi Dormitory

The building, shown in Fig. 37, is a reinforced concrete structure with two-stories above ground and one at basement level. The building has a total floor area of 550 m^2, weighs 830 tonnes, and is carried on 12 natural rubber isolators provided by the Bridgestone Corporation. They are 435 mm in diameter and 220 mm in height and provide a natural period of 1.9 seconds. The system includes 12 steel dampers of the type used in the Yachiyodai House and the Okumura Corporation building. With these features the period of the isolated structure is 1.3 seconds. Another type of damper is incorporated to provide damping at small displacements when the steel dampers are less effective. This damper consists of lead bars in the form of a *J* attached between the foundation and the superstructure. A view of the complete isolation system is shown in Fig. 38 and details are given in Ref. [10].

The building was constructed by TCRI and Yunichika Corporation and was opened to the public in April 1988. The response of the building to earthquakes is being studied by researchers are the Fukuoka University. As of this writing, no earthquake response has been reported.

In a joint research program, Tohoku University and Shimizu Construction Company constructed two full-sized test buildings side by side on the Sendai campus in northern Japan. Construction was completed in May 1986. The buildings are three-story reinforced concrete structures, one conventionally founded and the other base isolated. The dimensions and construction methods for the superstructure were otherwise identical. Each building has a rigid frame structure, and the outer walls consist of light-weight concrete panels. The plan dimensions of each building are 6 m by 10 m, the total combined area being 180 m^2. The site consists of an 18 m layer of loam with gravel with an average shear wave velocity of 310 m/sec overlying sandy tuff. The site frequency from micro-tremor observations is about 4 Hz.

A general view of the buildings is shown in Fig. 39; the structure on the right is base isolated. Plan and elevation views are shown in Fig. 40. The isolation system consists of 6 laminated unfilled elastomeric bearings and 12 oil dampers. Figure 41 shows the bearing dimensions and oil dampers. The oil dampers have an equivalent damping ratio of 15% for wide range of shear strain including small strains. The viscous oil used is expected to have a life of at least ten years.

Several static tests have been performed on the isolated building to confirm the bearing stiffness and damping properties. A static horizontal load was applied at the base of the isolated building using jacks to displace the building by ± 170 mm. Forced-vibration tests were performed to confirm the dynamic properties of the buildings. Vibration exciters were placed on the center of the roof slab and the natural

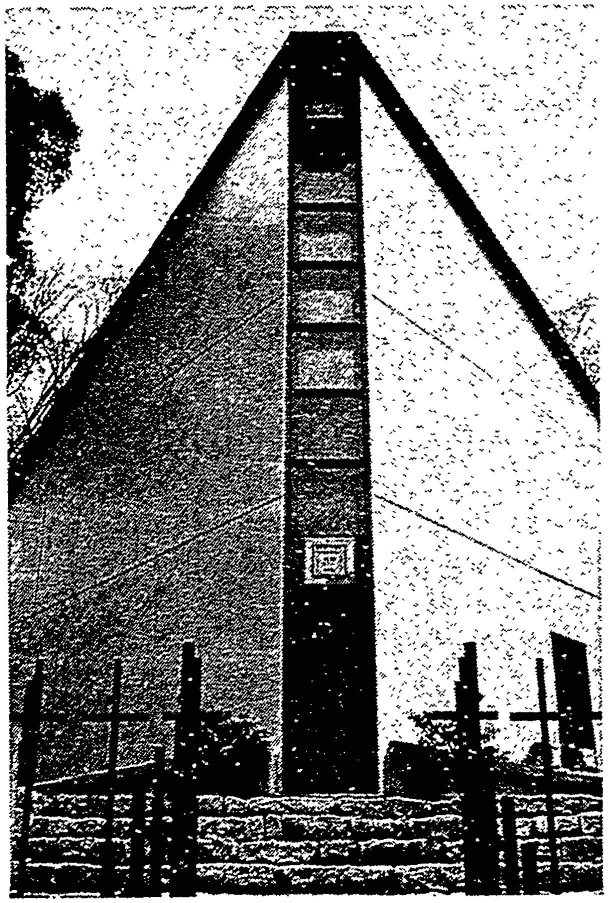


Figure 37 Base-Isolated Christian History Museum in Osio

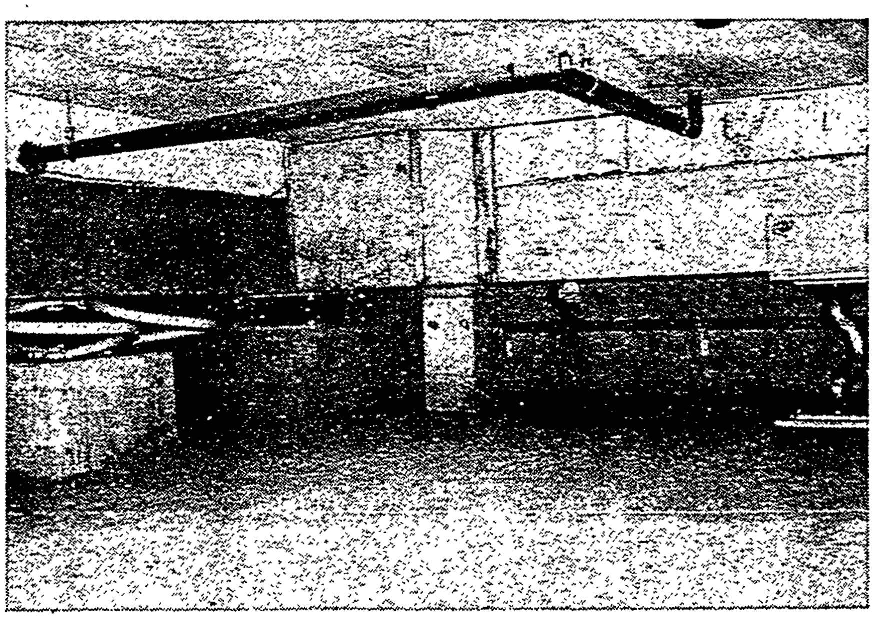


Figure 38 Isolation System for the Christian History Museum (Bearings, Coil Dampers, Lead Dampers)

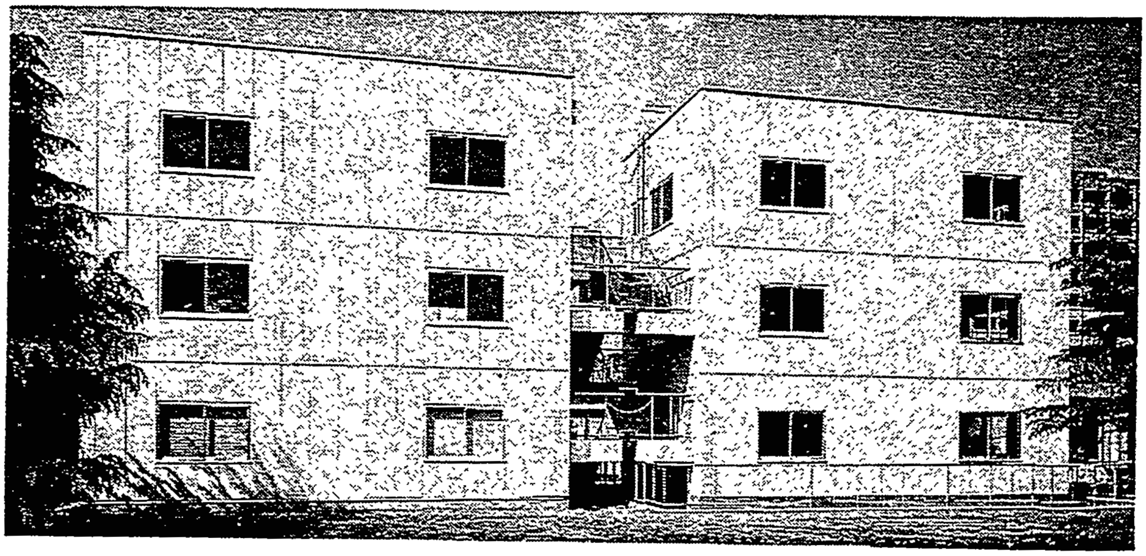


Figure 39 Shimuzu Company Test Buildings at Tohuku University

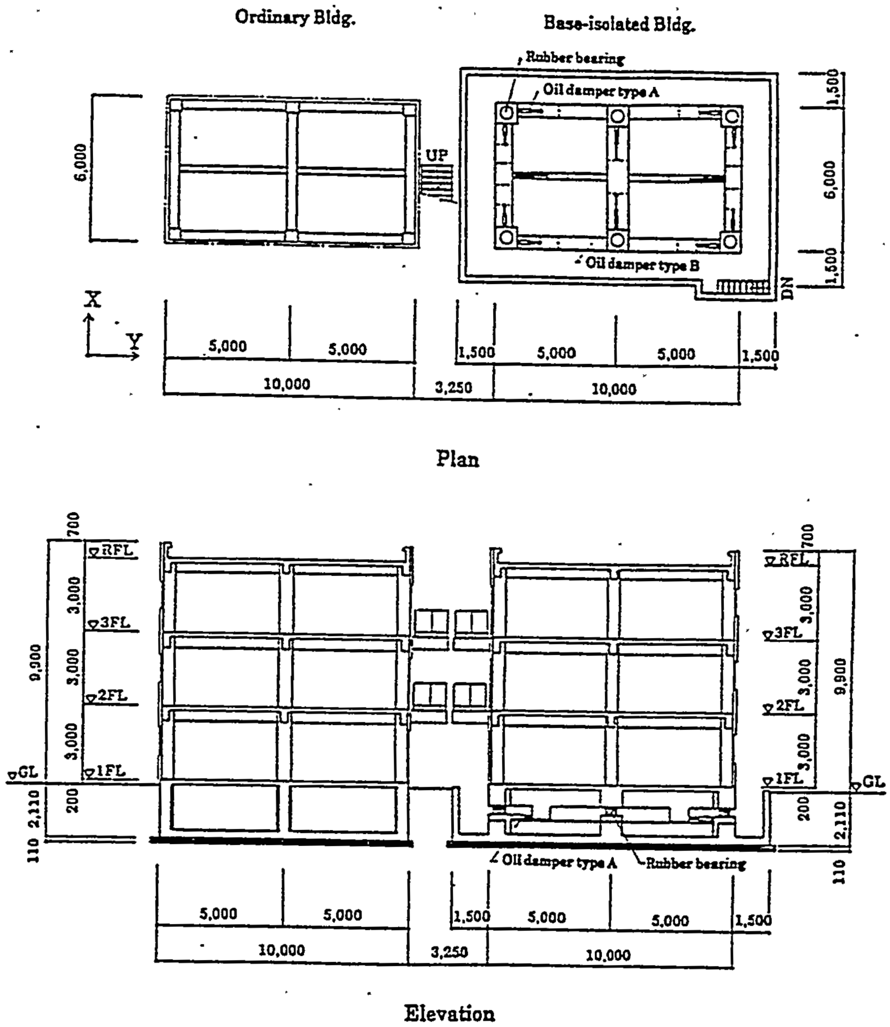
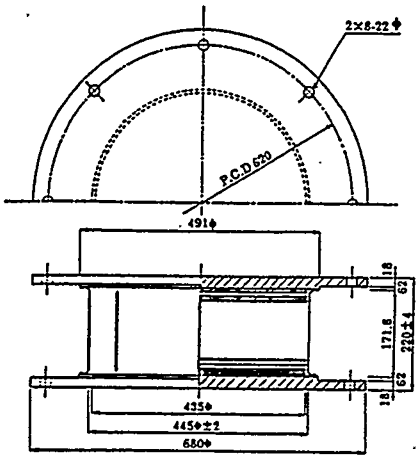
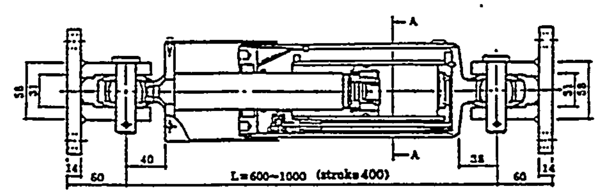
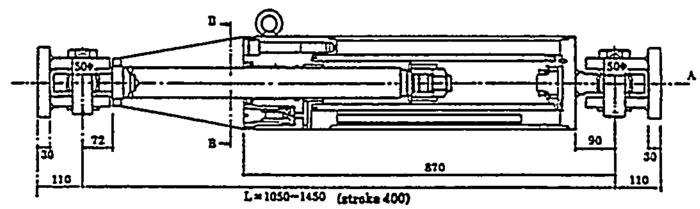


Figure 40 Layout of Test Buildings Showing Isolators and Dampers





Oil Damper Type A



Oil Damper Type B

Figure 41 Dimensions of Elastomeric Bearings and Oil Dampers

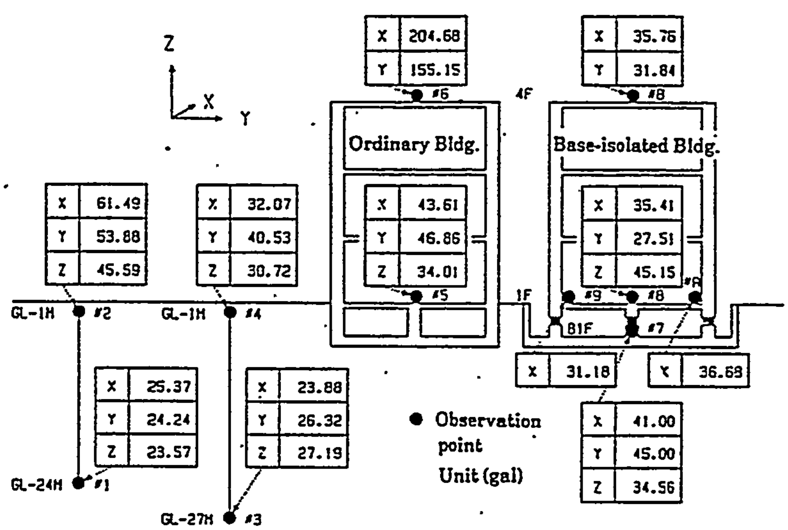


Figure 42 Recorded Maximum Accelerations during Earthquake of February 6, 1987

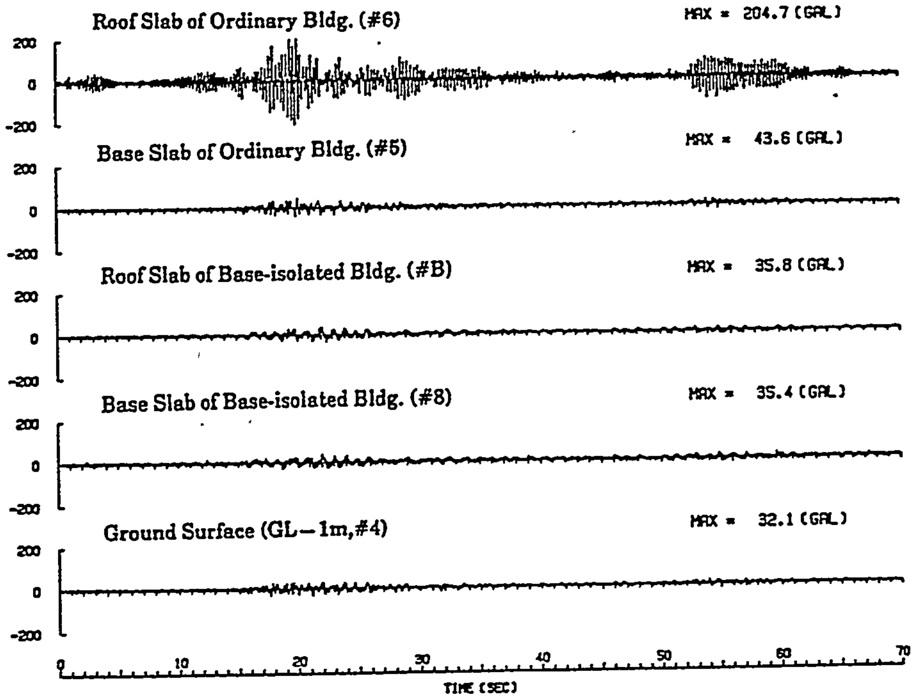
frequencies and damping rations were obtained experimentally. The natural frequencies of the conventional building were 3.6 Mz in the x-direction and 4.4 Hz in the y-direction. The frequencies of the isolated building with the oil dampers were 0.72 and 0.73 Hz, respectively. The damping ratios of the isolated building were 16% in the x-direction and 15% in the y-direction. The damping of the bearings alone is less than 2%.

The building have been instrumented with 11 strong motion accelerographs installed in the base rock at elevation -27 m, near the surface in the free field, on the base slab, and on the roofs of the two buildings. Thirty earthquakes were recorded [11] between June 1, 1986 and July 20, 1987, most of which were low-level events. The largest acceleration recorded on the ground surface was 87.27 gals (0.091g) and the corresponding peak accelerations at the roofs were 272 gals (0.28g) for the conventional 42 gals (0.04g) for the isolated structure. One of the strongest earthquakes recorded at the site was the earthquake of February 6, 1987 (magnitude 6.7, epicentral distance of 168 km). The values of maximum acceleration for this earthquake are shown in Fig. 42 and the time histories of acceleration are shown in Fig. 43.

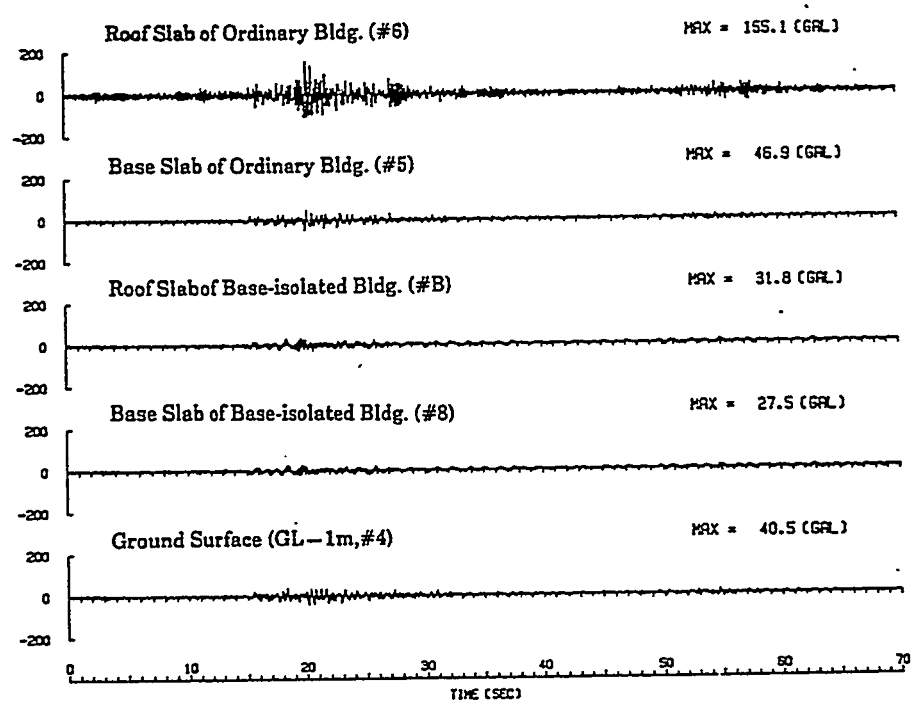
The amplification functions in the x-direction for the thirty earthquakes are plotted in Fig. 44(a). The mean value of amplification is 5.95 in the conventional building and 0.99 in the isolated structure. The amplification factors in the y-direction are shown in Fig. 44(b). The mean amplification values are 3.08 and 0.89 in the conventional and isolated buildings, respectively. The results for these earthquakes demonstrate the effectiveness of the isolation system in reducing accelerations induced in the superstructure.

The isolation system described above has since been replaced by 6 high-damping rubber bearings manufactured by Bridgestone Corporation. The relationship between lateral stiffness and damping versus shear strain for these bearings is shown in Figs. 45 and 46, respectively. The damping ratio at low strain levels is 19% and decreases to about 15% for higher strain. The buildings are being monitored for earthquakes to study the performance of these high-damping bearings.

A departure from the previous base-isolated demonstration buildings is that of the Taisei Corporation. This building was due to be completed in late 1988 on the grounds of the Taisei Technological Research Center in Yokohama. In contrast to other base isolation systems, which use natural rubber bearings as primary isolation elements, the isolation system for this building uses a sliding system called TASS (Taisei Shake Suppression System).

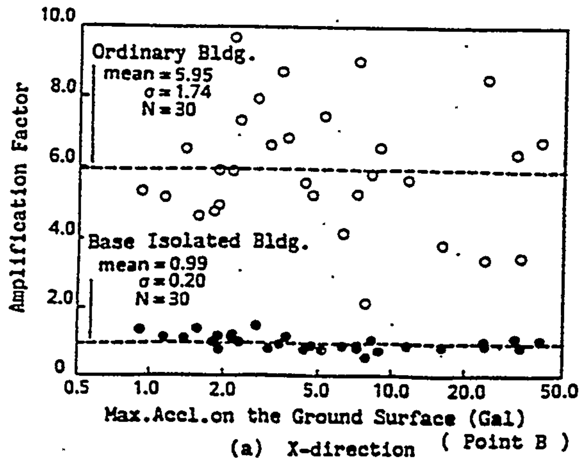


X-DIRECTION



Y-DIRECTION

Figure 43 Acceleration Time Histories for Earthquake of February 6, 1987



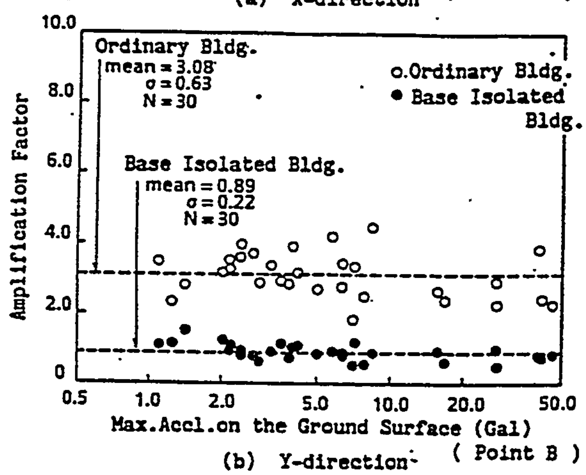


Figure 44 Amplication Factors for Test Buildings

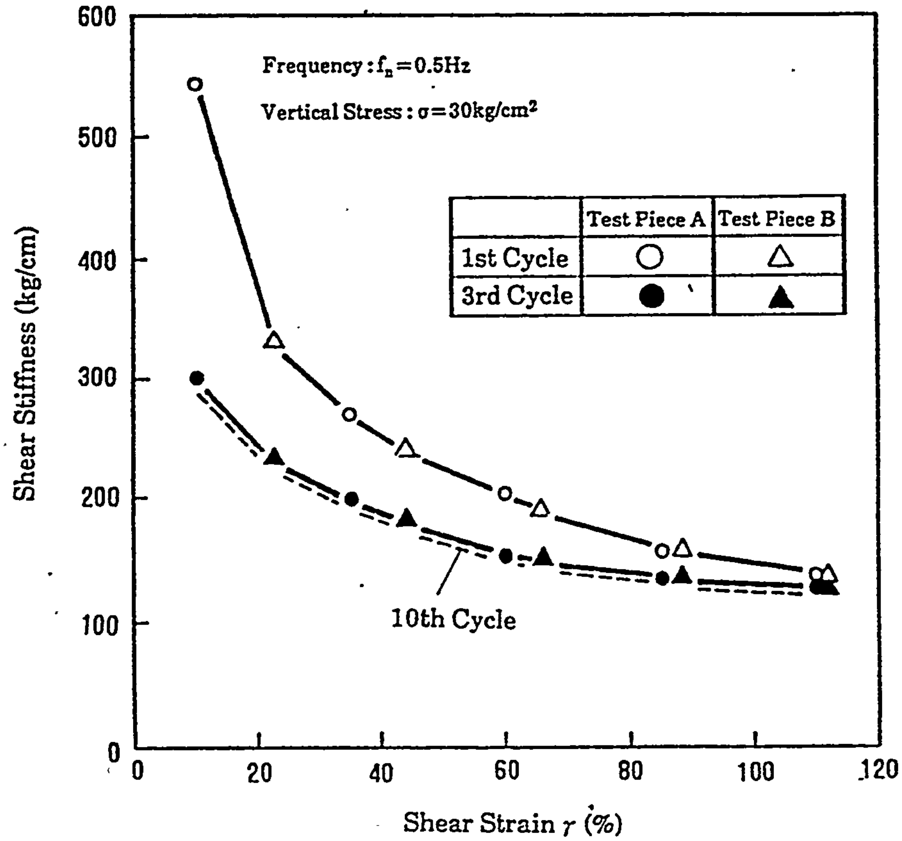


Figure 45 Relationship between Shear Stiffness and Shear Strain for High Damping Elastomer

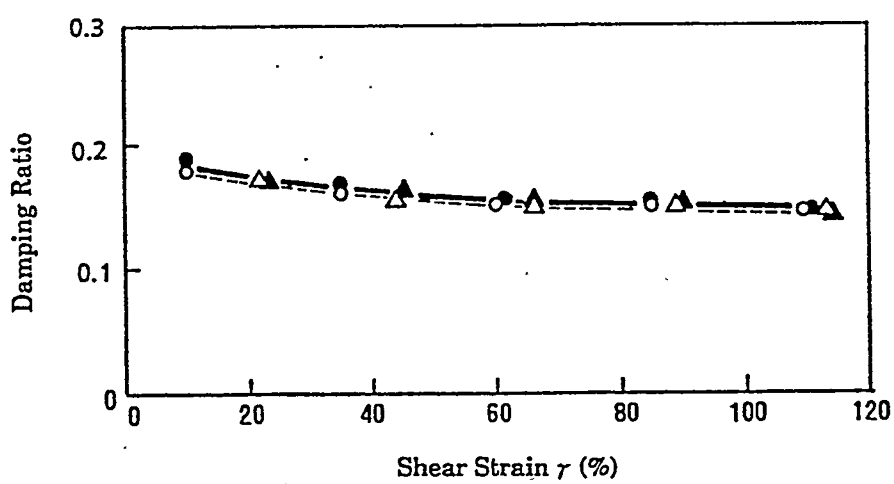


Figure 46 Relationship between Damping and Shear Strain

The building shown in Fig. 47 is a cast-in-place, four-story reinforced concrete structure with a plan area of 323 m^2. The building is supported on eight TASS bearings which use a teflon/stainless steel sliding surface and carry the entire vertical load of the building. A separate rubber bearing that carries no vertical load is positioned near each sliding bearing (Figs. 48 through 50) which act as a restoring force and to control sliding displacement.

The sliding bearing has 6 layers of rubber under the teflon surface permitting a certain amount of movement before the interface begins to slide. Each bearing is 85 cm in diameter as loaded to 7 MPa. The friction coefficient varies from 0.1 to 0.16 and the isolation period before sliding occurs is about 1.2 seconds.

The building above the isolators was designed for a base shear coefficient of 0.15. Analyses of the response of the structure to El Centro 1940 and to Hachinohe 1968 were performed assuming a constant friction factor of 0.10 and it was concluded that the system would reduce the peak acceleration in the building considerably with a predicted maximum displacement of around 25 cm. Shaking table testing of a model for demonstration was carried out and the results are presented in Ref. [12].

Commercial Buildings Using Base Isolation

There is at present no design code for base-isolated buildings in Japan. Each design is reviewed by two committees in a review process that takes three months. A seven-member committee of the Japan Building Center acting as a consultant to the Ministry of Construction reviews the design and it is further reviewed by a committee of the Ministry of Construction before approval is granted. In the absence of a code, the design criteria acceptable to the Ministry of Construction are as follows:

(a) soft ground sites are to be avoided (liquefaction potential);

(b) basin-shaped areas should be carefully studied for wave amplification and enhancement of long-period motions;

(c) height is restricted to a maximum of twenty stories;

(d) eccentricities should not be excessive;

(e) base shear coefficient should be greater than 0.15 and superstructure should be designed for ductility;

(f) the design philosophy is to allow for no damage for level 1 earthquakes (PGV = 25 cm/sec), minor damage for level 2 earthquakes (PGV = 50 cm/sec), and to activate fail-safe systems for level 3 earthquakes (PGV = 75 cm/sec);

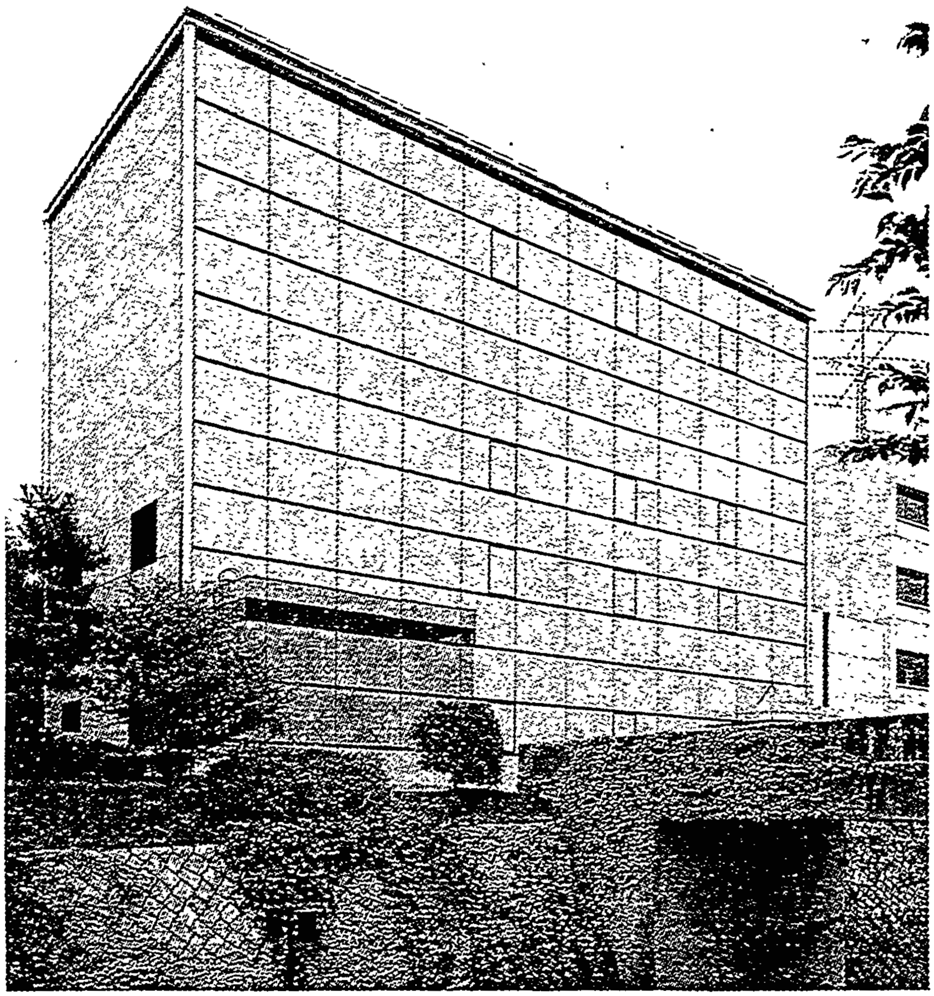


Figure 47 Taisei Corporation Demonstration Building on TASS System

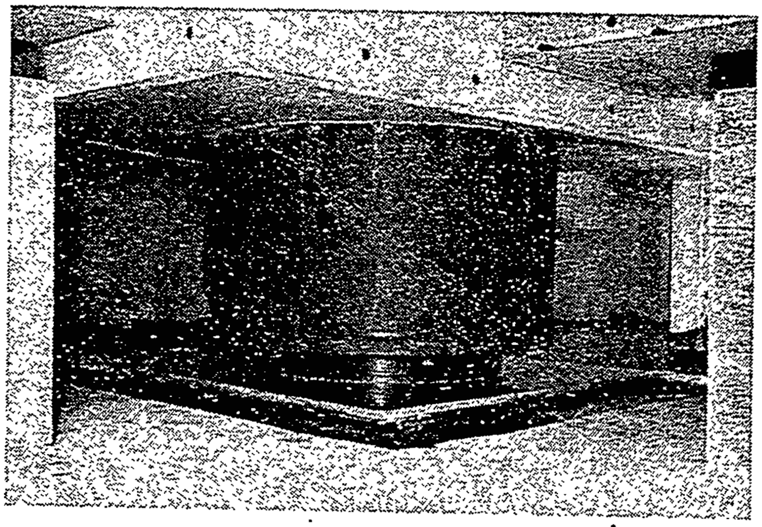


Figure 48 Vertical Load Support and Sliding Components

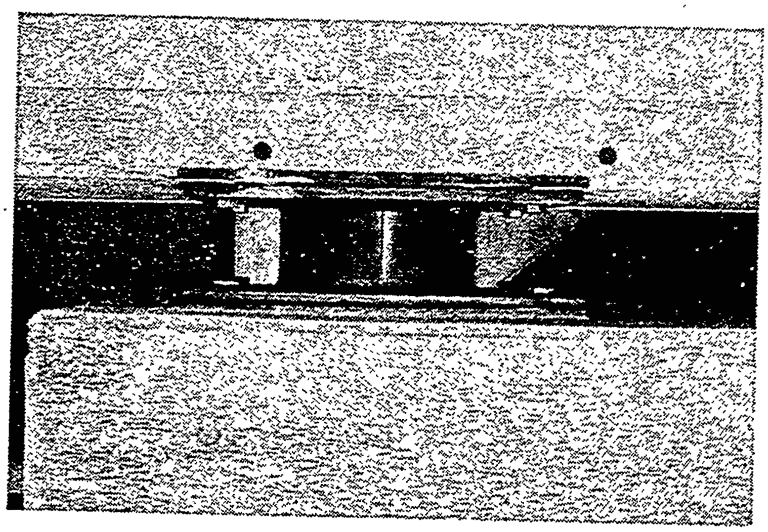


Figure 49 Horizontal Elastomeric Spring Used in TASS System

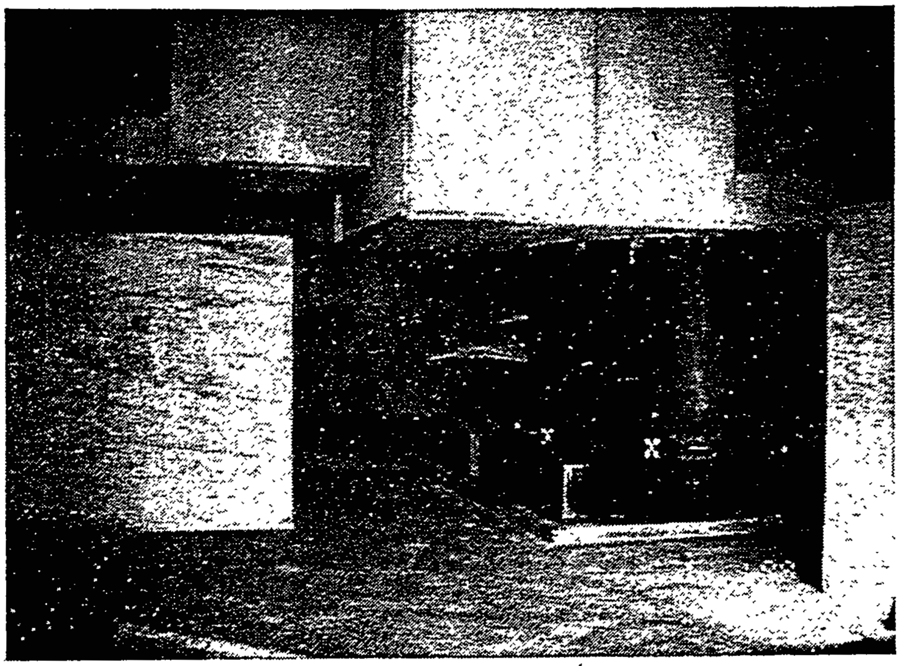


Figure 50 TASS System in Demonstration Building

(g) time history analyses must be performed using El Centro 1940 NS, Taft 1952 EW, and Hachinohe 1968 NS and EW (long-period motion);

(h) elastomeric bearings should be designed to carry an average pressure of less than 10 MPa; the allowable average shear strain (extreme lateral displacement divided by total rubber thickness) is a function of average pressure and is generally less than 200%;

(i) aging and fire-proofing of isolators should be checked; and

(j) provisions should be made for replacing the isolation mechanism if necessary.

Several construction companies have gained approval for commercial buildings that are now under construction and others are awaiting approval. Such details as are available on the isolation systems and the design of the buildings follow.

Ohbayashi-Gumi Co. has completed an office building in Shibuya for Shimizu Co. This is a five-story (Fig. 51) reinforced concrete building of 560 m^2 on twenty Bridgestone natural rubber isolators with 108 round steel bar dampers. It was granted Ministry of Construction approval in 1987 and was completed in 1988. Details of the building and the isolation system are shown in Fig. 52.

Ohbayashi-Gumi has also recently completed an electron microscope laboratory at the Tsukuba Mukizai Research Institute. This building is only one story (Fig. 53) and is 834 m^2 in area. It uses 32 Bridgestone bearings and 48 round steel bars as dampers. The need for isolation in this building is not dictated by seismic design requirements for the building but by the need to provide a high degree of seismic protection to the extremely sensitive instrumentation that it houses. The system provides a building which is both vibration isolated and seismically isolated. The design of the building was approved in 1987 and construction completed in 1988. In late 1987, Ohbayashi-Gumi was granted approval for an eight-story office building of 461 m^2 which is now under construction.

The Okumura Construction Co. in collaboration with Tokyo Kenchiku Structural Engineering completed a four-story apartment of 225 m^2 in 1987. It is carried on 12 rubber bearings and has 7 of the coil-type elasto-plastic dampers developed by Okumura. The same collaboration has built a 102 m^2 research laboratory for the Fujita Corporation in Kanagawa. It is a three-story reinforced concrete building on 4 rubber bearings with the coil-type dampers. Additionally, the Okumura Construction Company was granted permission in 1987 to build a three-story apartment house of 192 m^2 on 10 rubber bearings with 7 coil-type dampers.

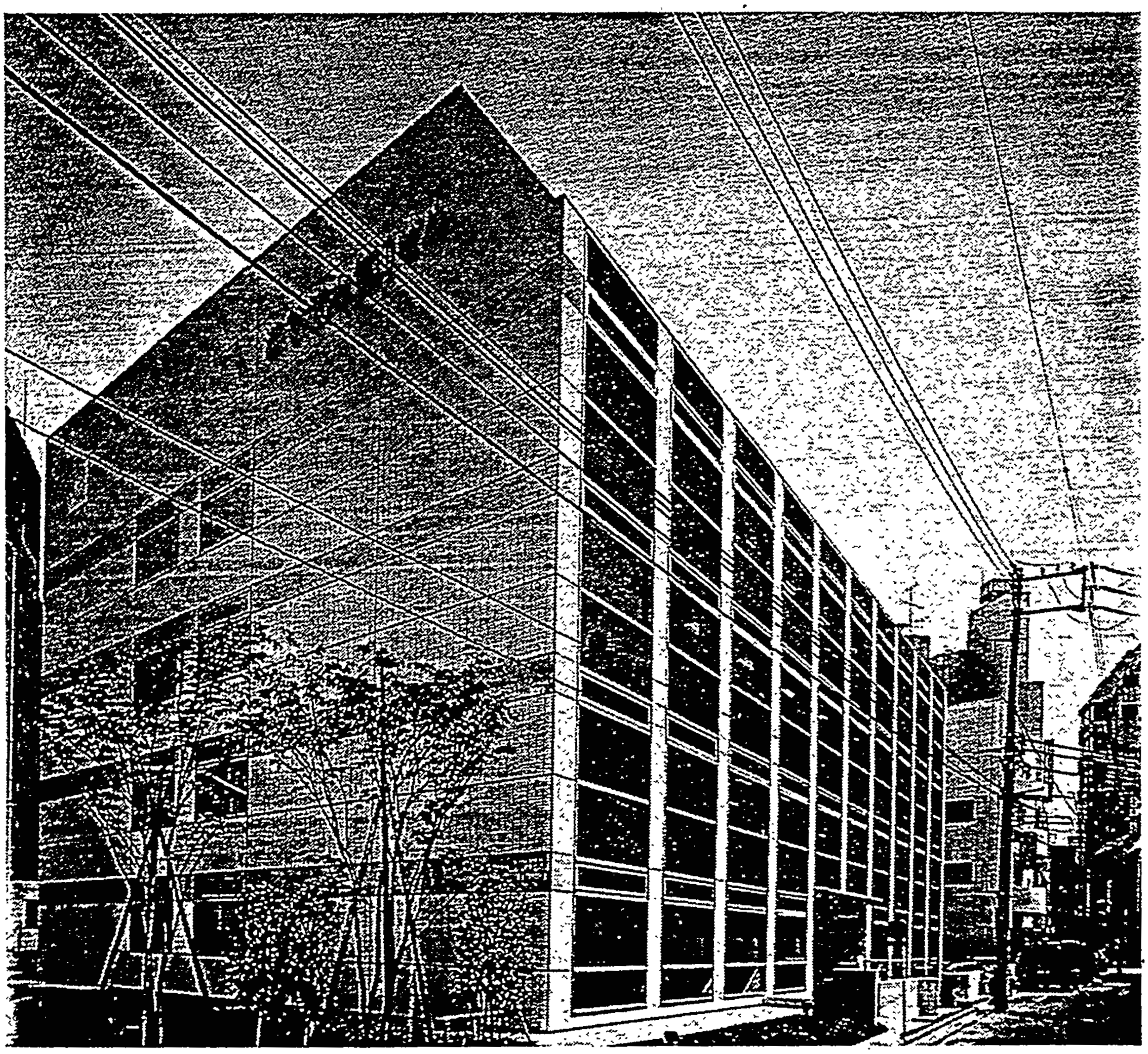


Figure 51 Shimizu Corporation Shibuya Office Building
Built by Ohbayashi-Gumi Ltd.

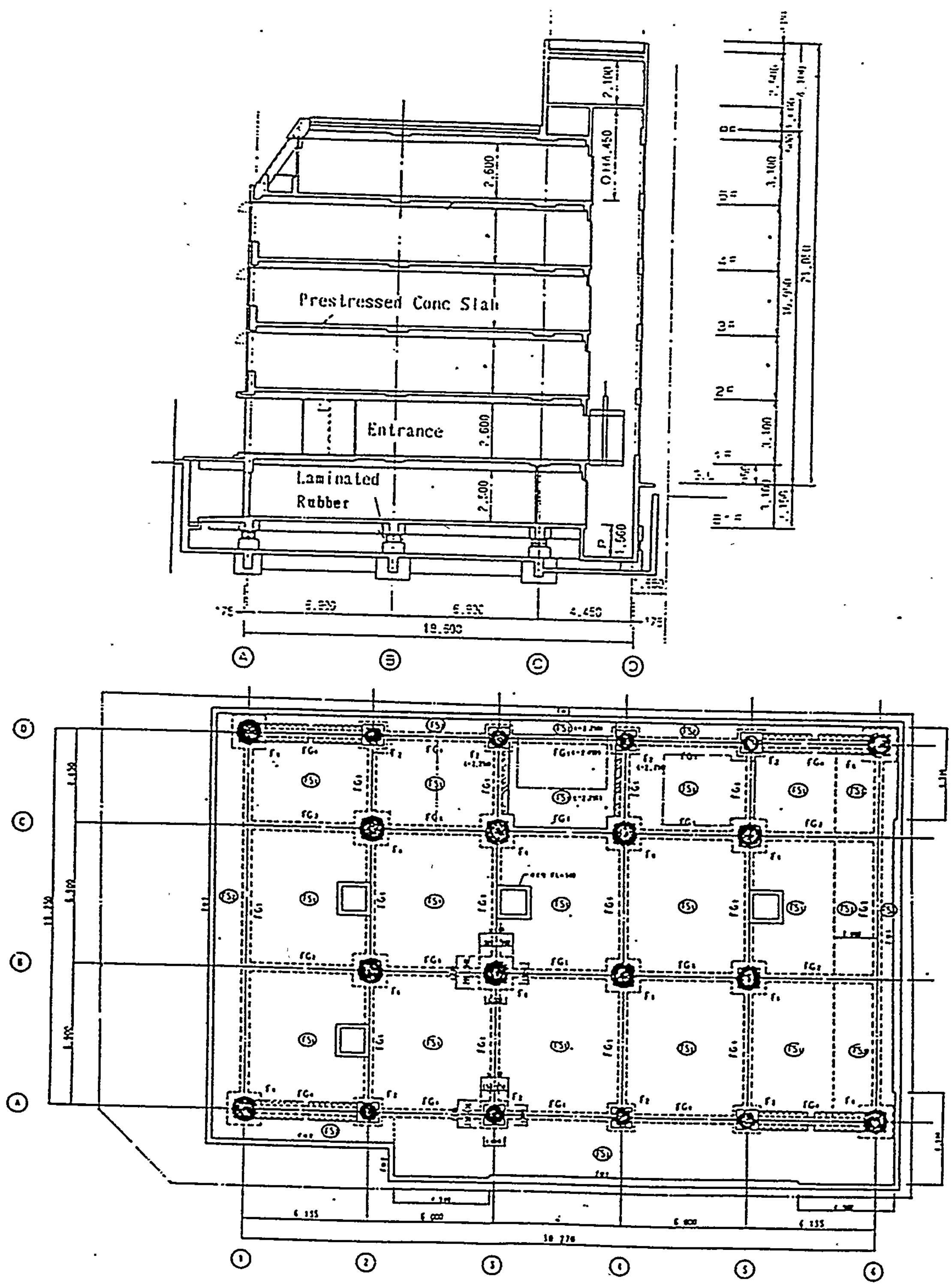


Figure 52 Plan and Section of Shimizu Corporation Base-Isolated Building in Shibuya

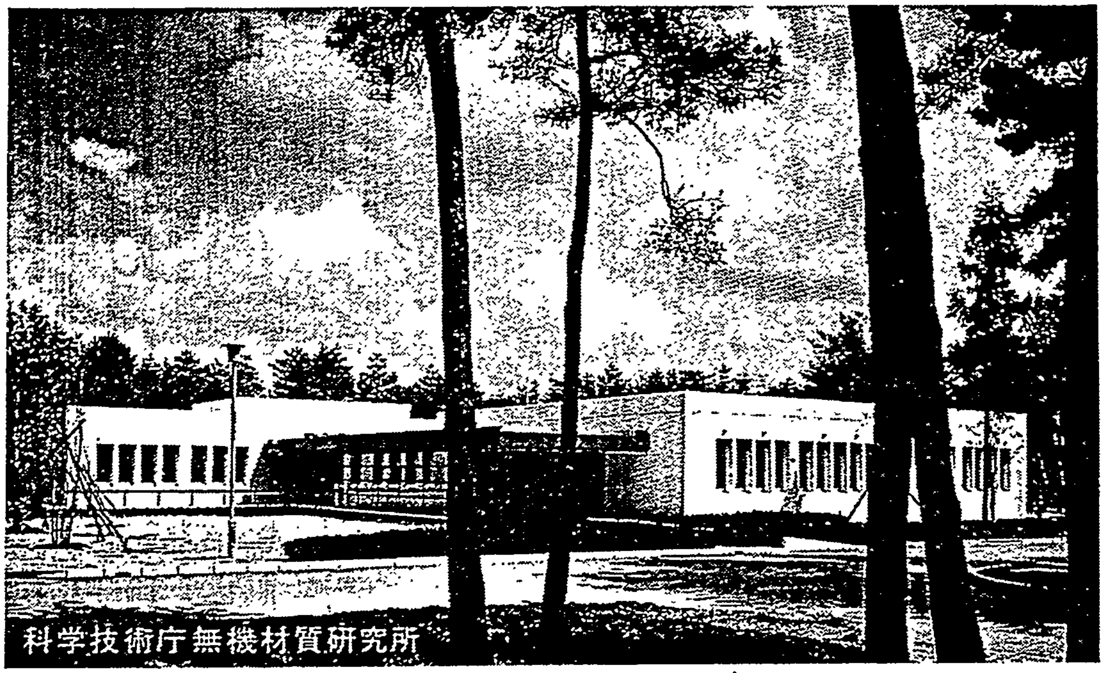


Figure 53 Electron Microscope Building by Ohbayashi-Gumi Ltd

Figure 54 Ibaraki Branch Office of Shimuzu Corporation

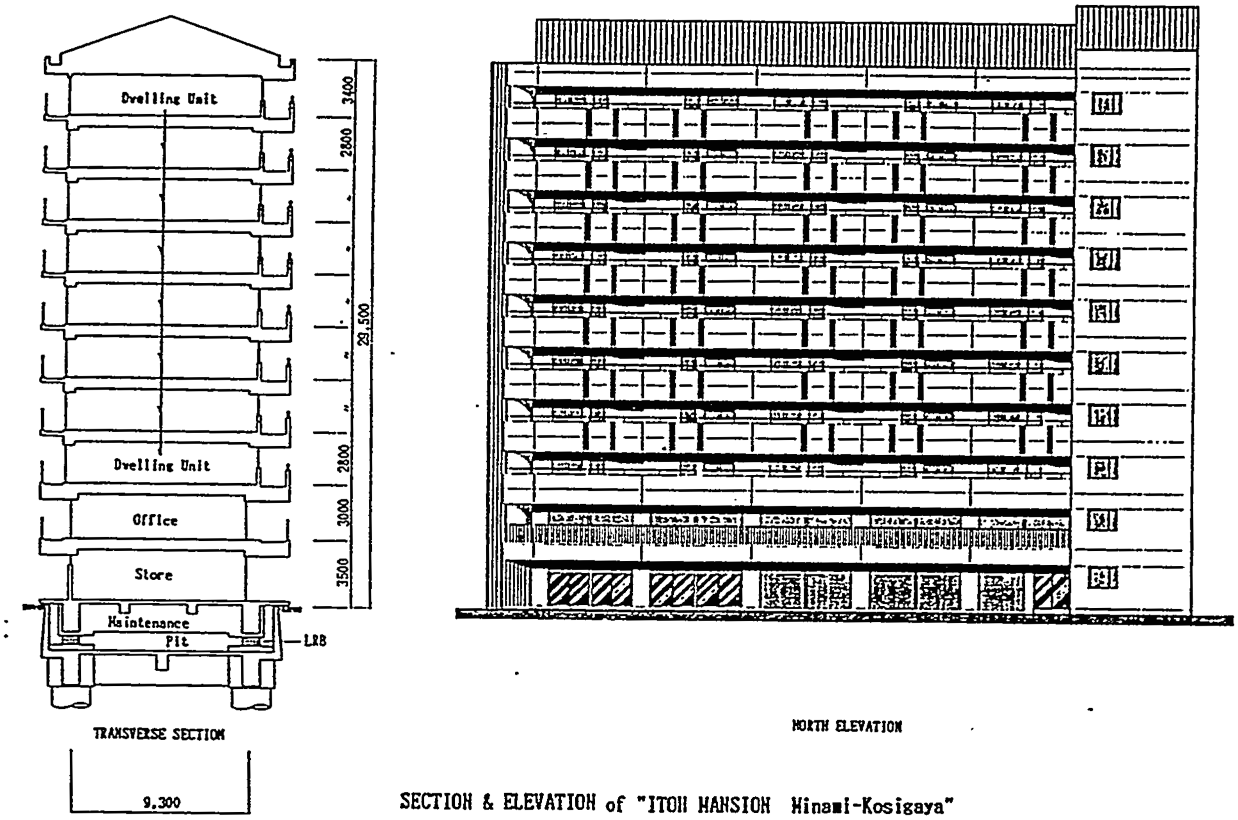
The Shimizu Construction Co. has built two base-isolated buildings in addition to its demonstration building. One is a four-story office building of 170 m^2 on 14 lead-rubber bearings provided by Oiles Industries (see Fig. 54). It is located in Ibaraki and is a branch office of the company. The second is an eight-story office building for the Bridgestone Corporation on 12 Bridgestone bearings. The dampers in this building are round steel cantilever bars.

In addition to the design and construction of the Oiles Technical Center, Sumitomo has recently completed the design of a ten-story apartment building and a seven-story office building both of which are to be isolated with lead-rubber bearings. The apartment building is known as the Itoh Mansion in Minami-Kosigaya near Tokyo (Fig. 55). Its total weight of 660 tonnes will be supported on 14 lead-rubber bearings, each 900 mm in diameter with various lead plugs ranging from 160 to 222 mm in diameter. Eight uplift restrainers are used to resist overturning of the building in the transverse (narrow) direction (Fig. 55). These devices are structural yokes which surround massive edge beams that run the length of the building. Each set of 4 uplift restrainers has a capacity of 500 tonnes. Two omnidirectional stoppers are used to control displacement (maximum limiting force is 400 tonnes). This building was approved in February 1988 and construction is underway.

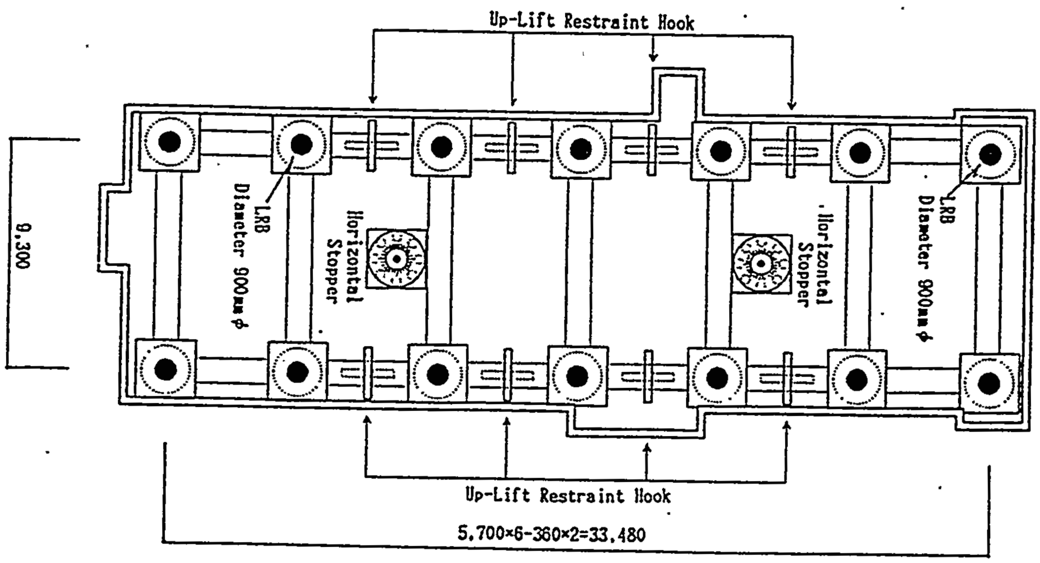
Located in Nagoya City and to be known as the Asano Building, the seven-story office building is unusual in that it is to be isolated just below the second floor (Fig. 56). The total weight to be supported above the isolators is 3,800 tonnes. Ten lead-rubber bearings of diameter 800 mm and 900 mm and two horizontal stoppers are to be installed on a mezzanine floor within the first story of the building. The motivation for isolating at this level is to eliminate the trenches around the building which are necessary if the isolators are located in the basement. Such a detail permits the building to extend to the lot line and optimizes the use of expensive commercial real estate.

A consequence of this configuration has been the need to quantify the fire resistance of elastomeric bearings. Sumitomo has conducted tests on various bearings using different fire protection materials wrapped around each. Performance has been observed in three-hour tests at 1,000 °C in accordance with the Japanese equivalent of the ASTM standard. Excellent resistance has been demonstrated and this work appears to confirm previous work done by the Malaysian Rubber Producers' Research Association of England on unprotected bearings. The design for the Asano Building has been submitted for approval and should be under construction before the end of 1988.

The Fujita Corporation has built a small three-story reinforced concrete building on isolation bearings provided by Oiles Industries Ltd. The bearings are 450 mm in

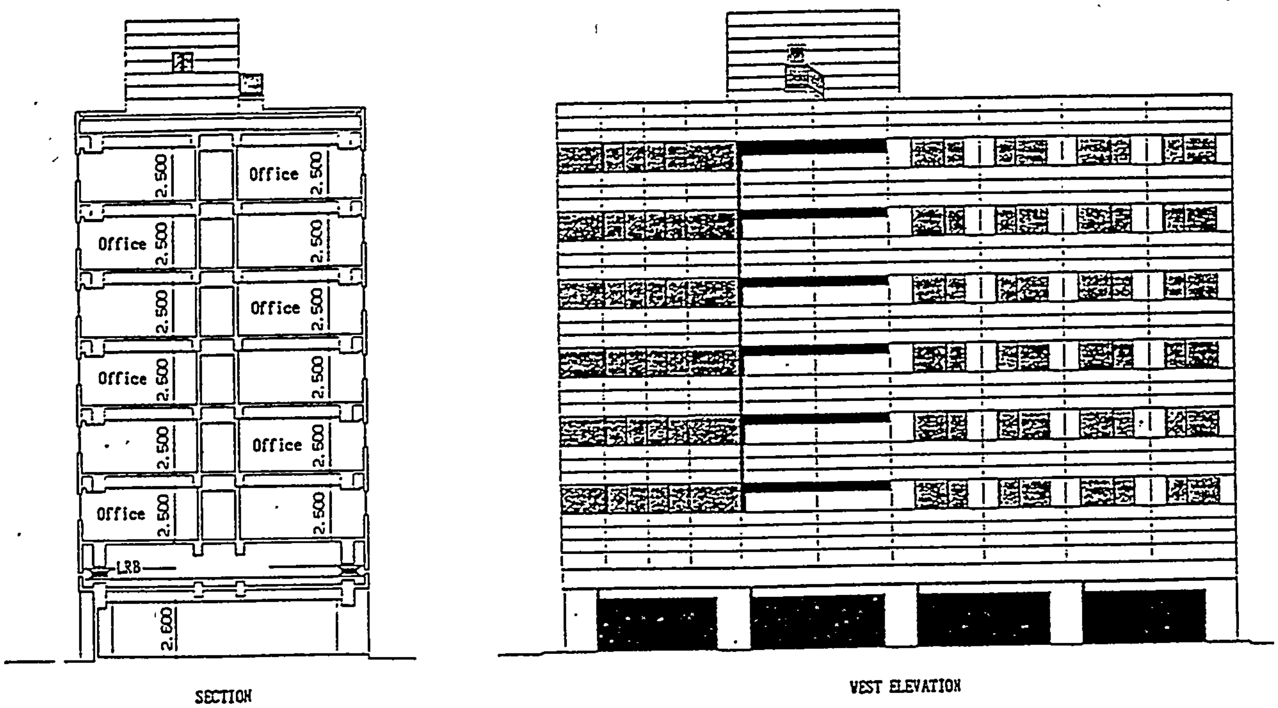


SECTION & ELEVATION of "ITOH MANSION Minami-Kosigaya"

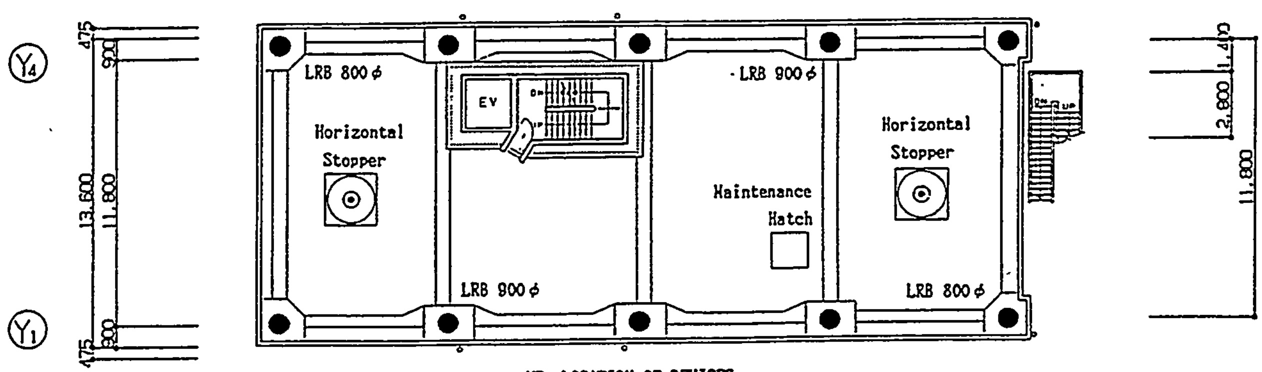


LOCATION OF DEVICES

Figure 55 Itoh Mansion under Construction by Sumitomo Construction Company



SECTION & ELEVATION OF "ASANO BUILDING"



MF LOCATION OF DEVICES

Figure 56 Asano Building under Construction by Sumitomo Construction Company diameter and are 350 mm high, there are four in the isolation system and each carries a load of 85 tonnes. The building was initially constructed as fixed at the base and remained so for six months after which the isolators were installed. Forced vibration tests and observations of its earthquake response were carried out for both isolated and non-isolated states. Small earthquakes that occurred when the building was both fixed and isolated and comparisons of the responses indicate that the amplification factor for ground acceleration was smaller for the isolated condition. The effectiveness of the system should of course increase as the earthquake intensity increases. The building on the four bearings is shown in Fig. 57 and a close-up view of two of the bearings is shown in Fig. 58.

Nikken Sekkei is designing a six-story, 3.5 m by 22.5 m computer center using 28 low-damping natural rubber bearings 700 mm high and 600 mm in diameter. The period of the structure on rubber alone is about 3.0 seconds. The system will incorporate a set of friction dampers designed by Sumitomo Metal Industries, Ltd. The system will include 12 of the friction dampers. The total weight of the building is 7,000 tonnes, and the floor area is 4500 m^2 and it is 24.8 m high. Approval is anticipated in May 1989 and construction will begin shortly after. An artist's rendering of the building is shown in Fig. 59.

There are several other projects undergoing the approval process at this time, both offices and apartment buildings, the tallest of which is fourteen stories. No details of the isolators or superstructures are available. It is to be expected that the boom in the construction of base-isolated buildings in Japan will continue and will be very influential in stimulating acceptance of the strategy in other countries.

Concluding Remarks

Why has base isolation taken such a hold in Japan and why so rapidly? Research expenditures play a major role. Each of the large construction companies has invested about $3 million per year on base isolation research and development which combined with the investment by the bearing manufacturers such as Bridgestone and Oiles Industries puts total investment in base isolation research and development at approximately $25 million per year. This should be compared with a total of around $14 million made available for all earthquake engineering research by the National Science Foundation in the United States.

All the companies market base isolation very aggressively using videotapes, high-quality brochures, presentations on national television, technical papers in national and international conferences and numerous other formats. Most companies offer a

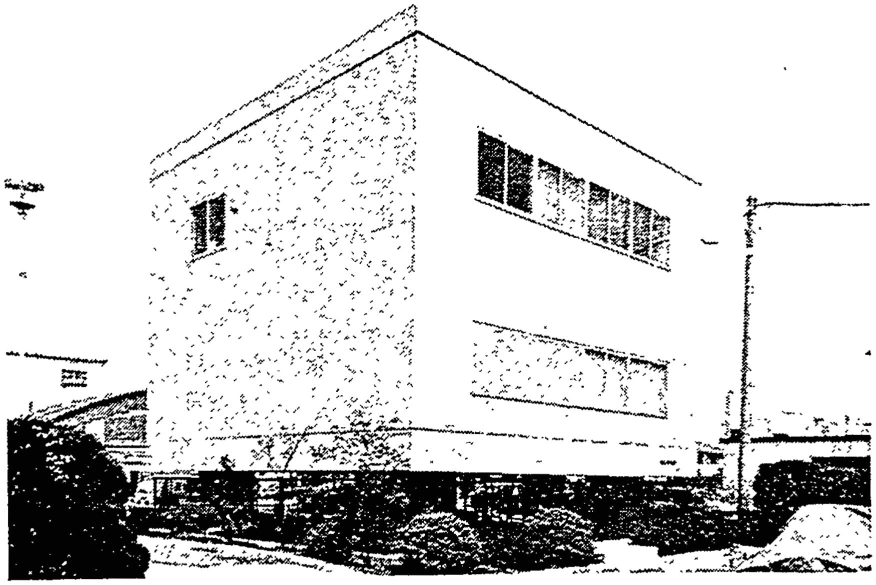


Figure 57 View of Fujita Corporation Base-Isolated Building

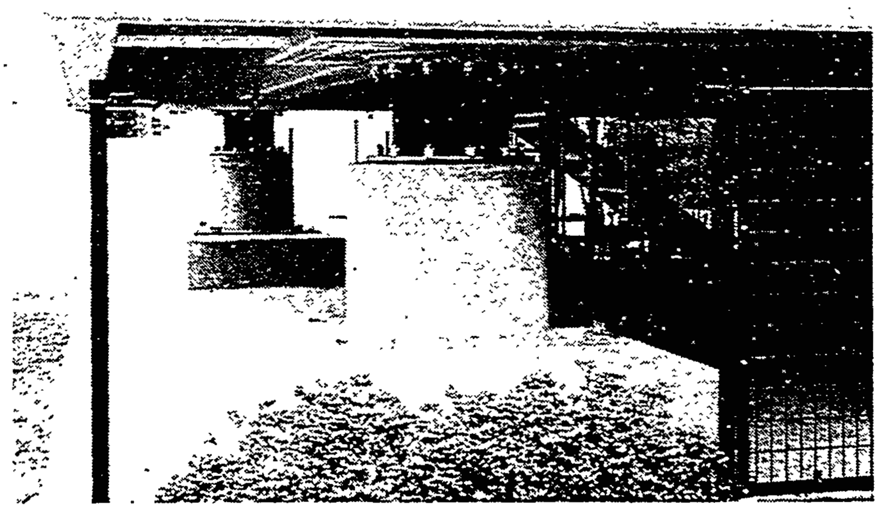


Figure 58 Isolators under Fujita Corporation Building

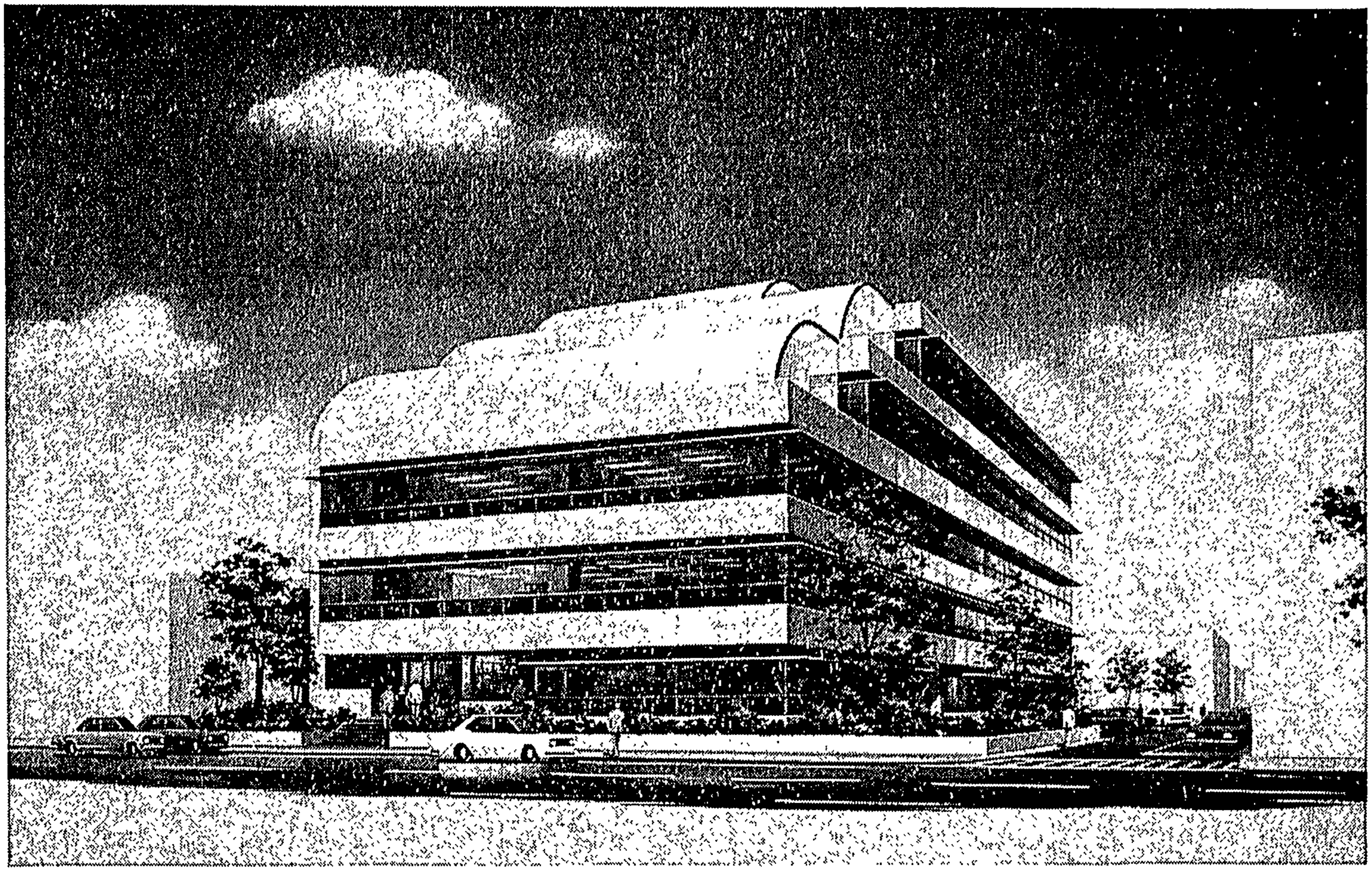


Figure 59 Rendering of Proposed Nikken Sekkei Base-Isolated Building

multifaceted approach to isolation including approaches to seismic isolation, vibration isolation (against traffic-induced vibration), floor isolation systems for computer centers and energy-absorbing devices for interior walls.

The approval process is conducted at a higher level than in the United States where a building permit would be issued by the office of the city engineer, who may not be comfortable with a new technology. It is not reduced cost that has generated the boom. Base-isolated buildings in Japan are generally between 5% and 10% more expensive than conventional construction. The increased cost is justified in the increased safety of the buildings and the potential reduction in damage to the contents of the building due to moderate and severe earthquakes over the life of the building.

In the final analysis, it is most probably the promise of increased safety and reduced damage that has led to the rapid acceptance of the new technology. Japan is a country where severe earthquakes are a certainty and the potential for catastrophic damage extremely high. Moderate earthquakes occur many times a year and are a constant public reminder of the danger. In contrast, earthquakes in the United States, even in California, are less frequent and the public is not as alert to the danger. In addition there is a long-term bias toward low-cost, low-bid construction. The Japanese view appears to be that life-cycle costs rather than first costs are more important in deciding on an earthquake-resistant design strategy and it is this philosophy that has led to the rapid acceptance of base isolation in Japan.

Recent Developments in Base Isolation in Japan

The previous chapters have covered base isolation in Japan up to the end of 1988. Since then there have been continued developments in all aspects of base isolation. The complete list of base-isolated buildings now includes fifty-three buildings, and much research is underway in universities and research institutes and a very large program of development for nuclear application has been funded. These recent developments are summarized in the following sections.

Civil Structures

All base isolation projects in Japan are approved by a standing committee of the Ministry of Construction. At the present time (6/30/91) a total of fifty-three buildings have been licensed and the list will no doubt continue to grow in the near future. A wide variety of isolators are being used in Japan and at the present time the most common are the high-damping rubber bearing and the lead rubber bearing. The current list of buildings is included in Table I. Many of the completed buildings have experienced

Table I							
List of Seismic Isolation Buildings in Japan							
License		Name of Building	Designer and/or Constructor	Type of Isolation Device	Building		
					Story		(m^2)
1	83/3	Yachiodai House	Tokyo Arch. Okumura	RB+PC Friction	RC	2F	114
2	85/11	Tsukuba Research Bldg.	Unichika,etc/ Okumura	RB+Loop Type Steel	RC	4F	1330
3	86/1	Tech. Research Bldg.	Ohbayashi	RB+Hysteretic Steel Bar	RC	5F1B	1624
		Tohoku Univ. Bldg.	Shimizu	RB+Oil Damper - High-Damping Rubber	RC	3F	417
4	86/3	Oiles Corp. TC Bldg.	Yasui Sekkei/ Sumitomo	Lead Rubber	RC	5F	4765
5	86/4	Taketomo Dormitory	Takenaka	RB+Viscous Damping	RC	3F	1530
6	86/5	Nishi Chofu Audio Bldg.	Kajima	RB+Hysteretic Steel Bar	RC	2F	655
7	86/7	Ohiso Christian Museum	Tokyo Arch./ Okumura	RB+Loop Type Steel	RC	2F	293
8	86/12	APT Fukamiya	Tokyo Arch./ Okumura	RB+Loop Type Steel	RC	4F	682
9	87/2	Shibuya Shimizu Daiichi	Ohbayashi	RB+Hysteretic Steel Bar	RC	5F1B	3385
10	87/2	Research Ins. Exp. Bldg.	Fujita	Lead Rubber	RC	3F	395
11	87/6	Tsuchiura Bldg.	Shimizu	Lead Rubber	RC	4F	634
12	87/6	Inorganic Material Res.	MOC, etc./ Ohbayashi	RB+Hysteretic Steel Bar	RC	1F	616
13	87/7	Taisei Tech. Ins. Bldg.	Taisei	Sliding Plate+Rubber	RC	4F1B	1029
14	87/7	Koushinzuka No. 3 Bldg.	Tokyo Arch./ Okumura	RB+Hysteretic Steel Bar	RC	3F	476
15	87/12	BS Toranomon Bldg.	Shimizu/ Shimizu, JV	RB+Hysteretic Steel Bar +Oil Damper	RC	8F	3360
16	88/2	Minami Koshigaya Bldg.	Sumitomo	Lead Rubber	RC	10F	470
17	88/2	Ichinoe Dormitory	Kumagai	RB+Hysteretic Steel Bar	RC	3F	771

RB = Rubber Bearing

Table I - Continued

License		Name of Building	Designer and/or Constructor	Type of Isolation Device	Building		
					Story		(m^2)
18	88/6	14F-PR Bldg.	Tokyo Arch./ Okumura	RB+Loop Type Steel	RC	14F	16394
19	88/6	Takenaka Clean Room Bldg.	Takenaka	Multistage RB+ +Viscous Damper	RC	2F	406
20	88/6	JAPCO Atagawa Bldg.	Taisei	Sliding Plate+Rubber	RC	1F	140
21	88/6	Ogawa Bldg.	Kumagai	High-Damping Rubber	RC	4F	1186
22	88/6	Asano Bldg.	Sumitomo	Lead Rubber	RC	7F	3256
23	88/8	Kusukida Bldg.	Hazama	High-Damping Rubber	RC	4F1B	1044
24	88/12	Ichikawa Co. House	Kounoike	RB+Loop Type Steel +Lead Damper	RC	2F	298
25	88/12	Tohoku Electric Dai-nai Computer Center	Higashi Nippon Shimizu	High-Damping Rubber	RC	6F	10032
26	88/12	Tokyu Machine Parts Center	Tokyu	High-Damping Rubber	RC	2F	255
27	88/12	Gerontology Res. Ins.	Kume Arch./ Ohbayashi	RB+Hysteretic Steel Bar	RC	2F1B	1112
28	89/2	M-300 Oiles Rest Home	Mitsui Home	Lead Rubber	W	2F	310
29	89/2	Harvest Hills House	Okumura	RB+Loop Type Steel	RC	6F	1115
30	89/2	Nishi Chofu Reform Plan	Kajima	RB+Hysteretic Steel Bar +Buffer	RC	2F	655
31	89/4	Toshin 24 Omori Bldg.	Kamjima	RB+Hysteretic Steel Bar	SRC	9F1B	7579
32	89/4	Haseko Experiment House	Haseko	RB+Loop Type Steel +Lead Damper	RC	3F	681
33	89/4	Minami Otsuka Bldg.	Sumitomo	Lead Rubber	RC	12F2B	6020
34	89/4	Tobishima Tech. Ins. Bldg.	Tobishima	High-Damping Rubber +Rubber Damper	RC	3F	478
35	89/6	C.P. Fukusumi Bldg.	Nikken Design	RB+Lead Damper +Friction Damper	RC	5F	4407
36	89/6	Maeda Co. House	Maeda	Lead Rubber	RC	4F	653
37	89/8	Toho Gas Control Bldg.	Taisei	RB (Friction Type) +Rubber Spring	RC	3F	1800
38	89/8	Toda Tsudanuma Dorm.	Toda	RB+Hysteretic Steel Bar	RC	2F	202
39	89/10	Mitsui Home (M-300)	Mitsui Home	Lead Rubber +Back-up Ring	W	2F	214

Table I - Continued

License		Name of Building	Designer and/or Constructor	Type of Isolation Device	Building Story		(m^2)
40	89/10	Koganei Co. House	Fujita	Lead Rubber	RC	3F	714
41	89/10	Operation Center	Fujita	Lead Rubber	RC	2F1B	8657
42	89/12	Urawa Kogyo Kuki Factory	Hazama	High-Damping Rubber	RC	5F	1525
43	90/2	Kokudo Kaihatsu Tech. Ins.	Kokudo Kaihastu	RB+Viscous Damper	RC	3F	955
44	90/2	Hiroshima Nokyo	Tokyo Arch./ Okumura	Lead Rubber	RC	3F	5423
45	90/2	C-1 Bldg.	Nikon Sekkei/ Shimizu	Lead Rubber	SRC	7F1B	37845
46	90/2	Hydrodynamics Res. Ins.	Takenaka	RB+Viscous Damper	RC	3F	628
47	90/4	Mitsui Kashiwa Factory	Mitsui	High-Damping Rubber	RC	4F	2187
48	90/4	Tsukuba Tech. Ins.	Hazama	RB+Lead Damper +Friction Damper	RC	2F	908
49	90/4	Nishimatsu Yamato Dorm.	Nishimatsu	RB+Hysteretic Steel Bar	RC	8F	1922
50	90/6	Kawaguishi Dorm.	Daimatsu	Lead Rubber	RC	4F	659
51	90/6	PNC Computer Center	Nikken Sekkei/ Shimizu	RB+Lead Damper	RC	3F	3309
52	90/6	Ando Tech. Ins.	Ando	Lead Rubber	RC	3F	1930
53	90/6	Toyo Rubber Co. House	Kumagi	RB+Hysteretic Steel Bar	RC	7F	3520
54	90/6	Chube Elec. Power Co. Admin. Center	Shimizu+Kajima	High-Damping Rubber +Lead Rubber	SRC	6F	6000

earthquakes and their response can, in some cases, be compared with adjacent conventional structures. In such cases, the response of the isolated building has been judged to be highly favorable, although none of the buildings has been struck by a nearby large earthquake.

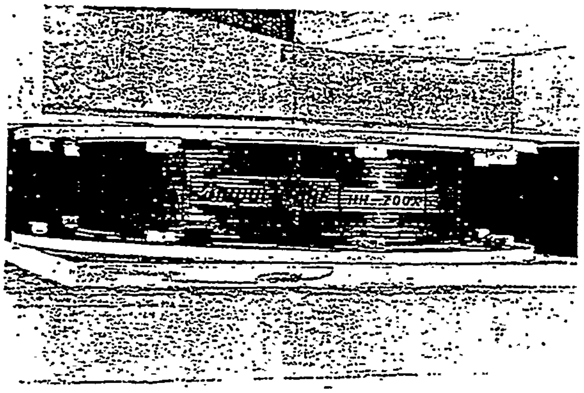
More than half of the buildings so far completed have used a combination of low-damping natural rubber bearings with some form of external damper. These external dampers have been hydraulic dampers, steel bars, and loop type steel coils. Another common type of isolator has been the lead plug isolator. Recently, in response to the concern that if the lead in the isolator fails it cannot be detected without removing the isolator, lead bars have been used as external dampers. Every type of external or internal damper requires mechanical attachments or other cost-increasing adaptations. In the past two years in Japan, there has been an increasing use of high-damping natural rubber isolators where the damping is included in the elastomer by appropriate compounding. There are now several large buildings that use this isolation approach and an outstanding example is the recently completed Dan-Ni computer center for Tohoku Electric Power Co. in Sendai, Miyako Province.

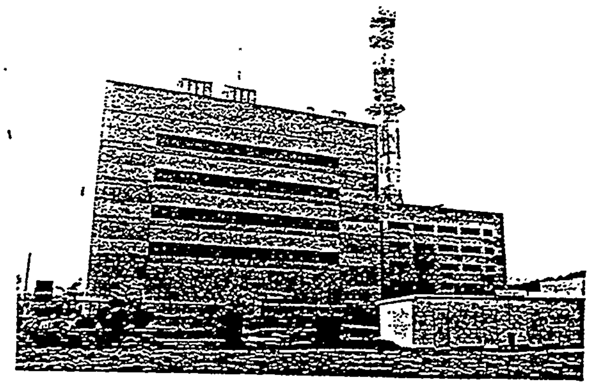
Dai-Ni Computer Center

The Dai-Ni Computer Center is a six-story reinforced concrete building in Sendai which is in a highly seismic area of northern Japan. The computer center houses the computers for the building and production records for the electric power utility. The building has a floor space of 10,000 square meters (108,000 sq.ft.), and is at the present time, the largest base-isolated building in Japan. It will accommodate a large number of main-frame computers and hard disk data storage equipment. It has large internal clear spans to facilitate location of the equipment. The standard bay is 9.5 m by 7.0 m. As a result of its six-stories, the column loads are very large due to the large column spacing and the type of equipment in the building. There are a total of 40 bearings of 3 different sizes. The isolators are 900 mm in diameter, 1,000 mm in diameter, and 1,200 mm in diameter, and the vertical loads range from 830 kips to 1,760 kips. The thickness of rubber in all of the isolators is 200 mm. The details of the structure and the isolators are given in Figs. 60-64.

The design of the structure was based on 4 standard earthquakes scaled to two levels; namely, Level 1 with a peak velocity of 35 cm/sec and Level 2 with a peak velocity of 50 cm/sec. The system was analyzed using nonlinear time history analysis, and it was determined that the superstructure would remain elastic at level 2 inputs, the isolation system would experience 200 mm displacement, and the base shear would

TOHOKU ELECTRIC COMPANY
NO.2 CONTROL AND DATA CENTER





LAYOUT OF MRB

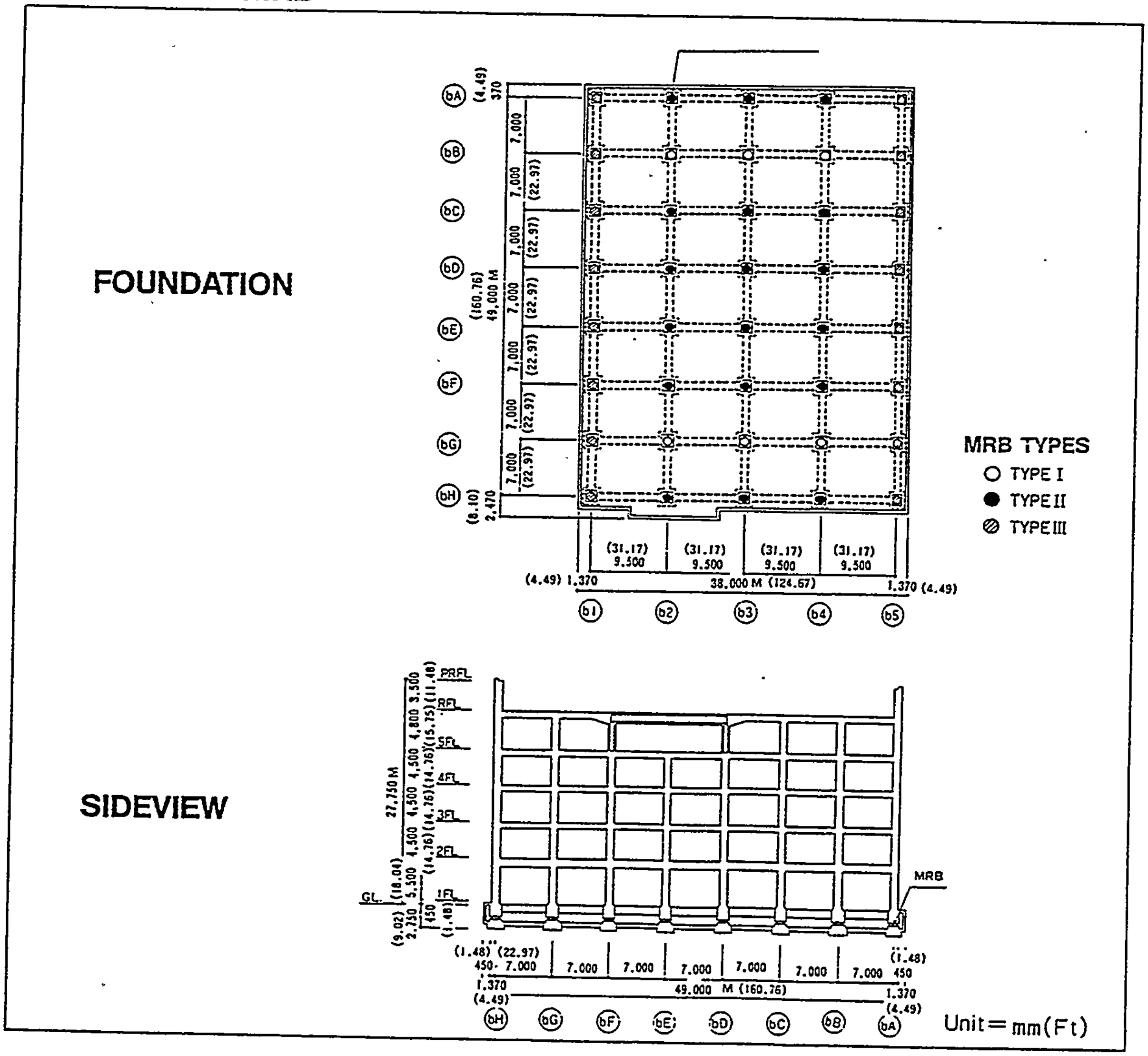
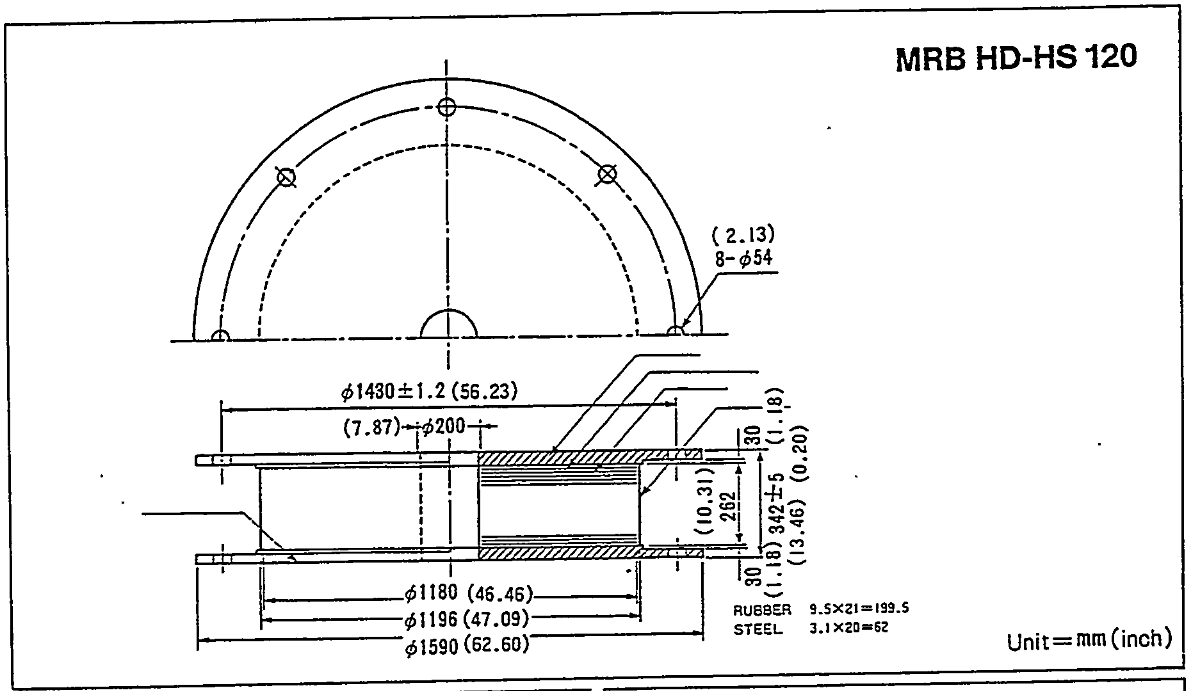
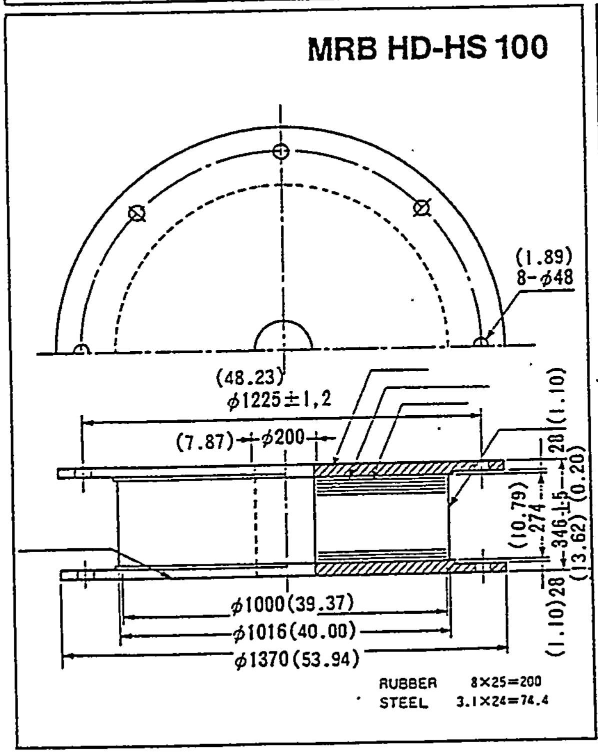


Fig. 60 Foundation and Side View of Data Center

Number of Stories: 6
Floor Space: 10,032 square meters (107,984 square feet)
Type of Structure: Reinforced concrete
Completion Date: 1989
Construction Company: Kumagai Construction
Location: Sendai, Japan





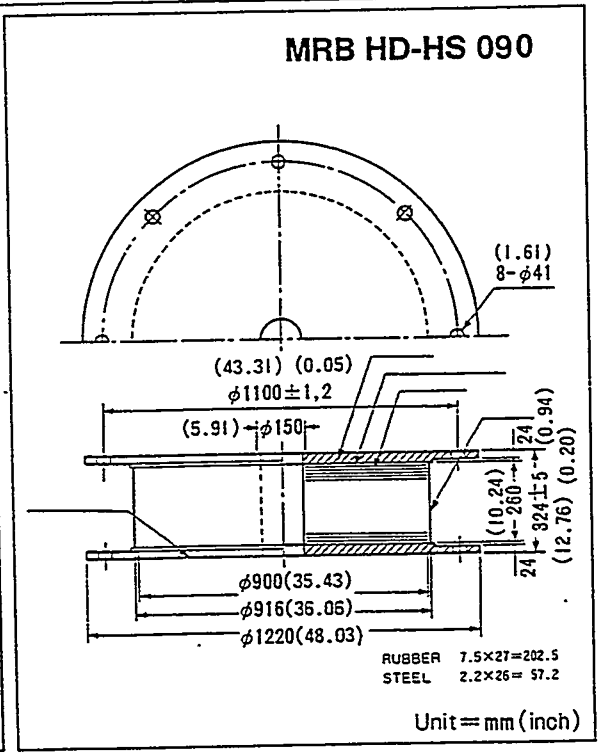


Fig. 61 Detail of Rubber Bearing

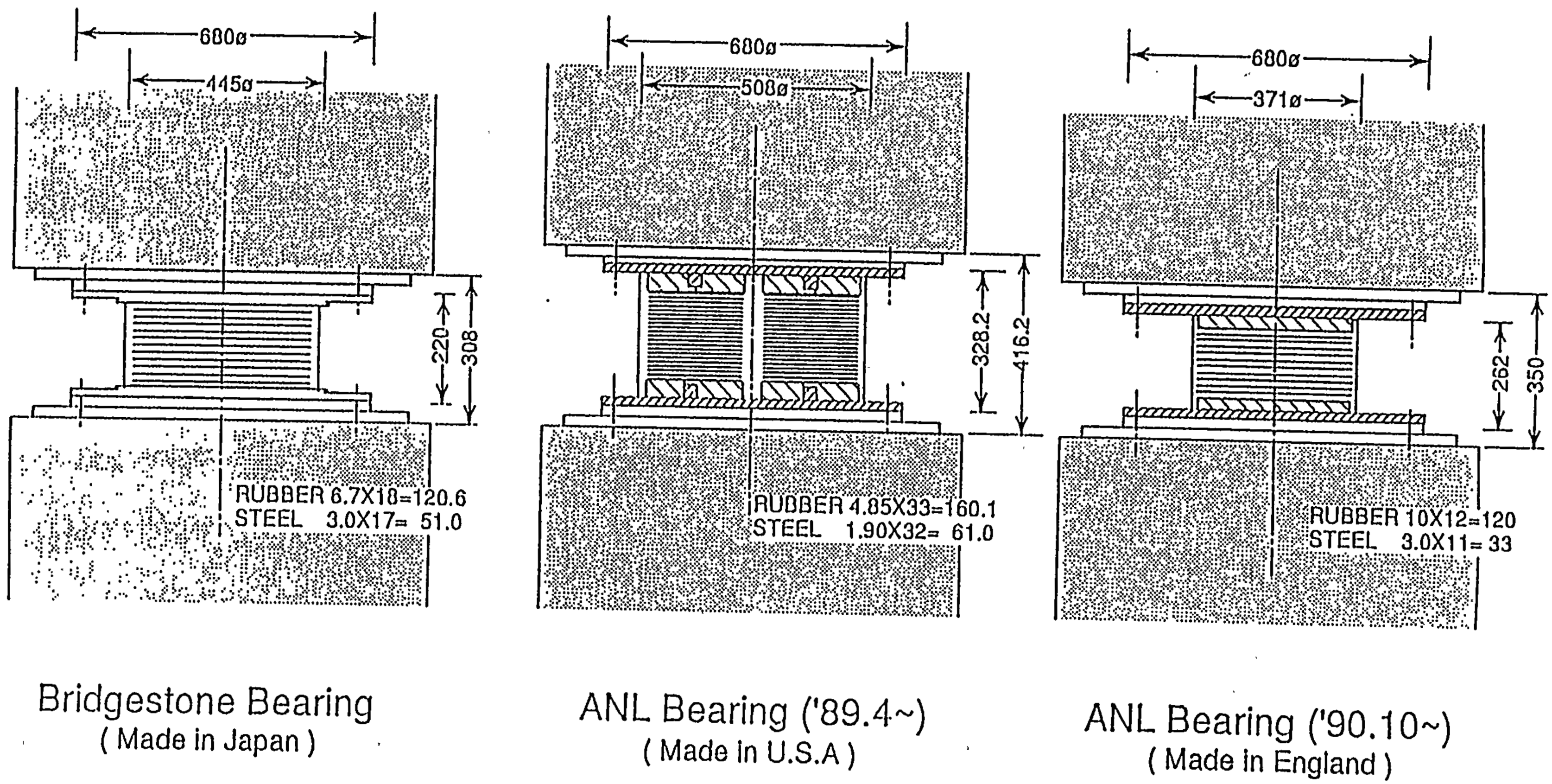


Fig. 62 High Damping Rubber Bearings Installed in Demonstration Building

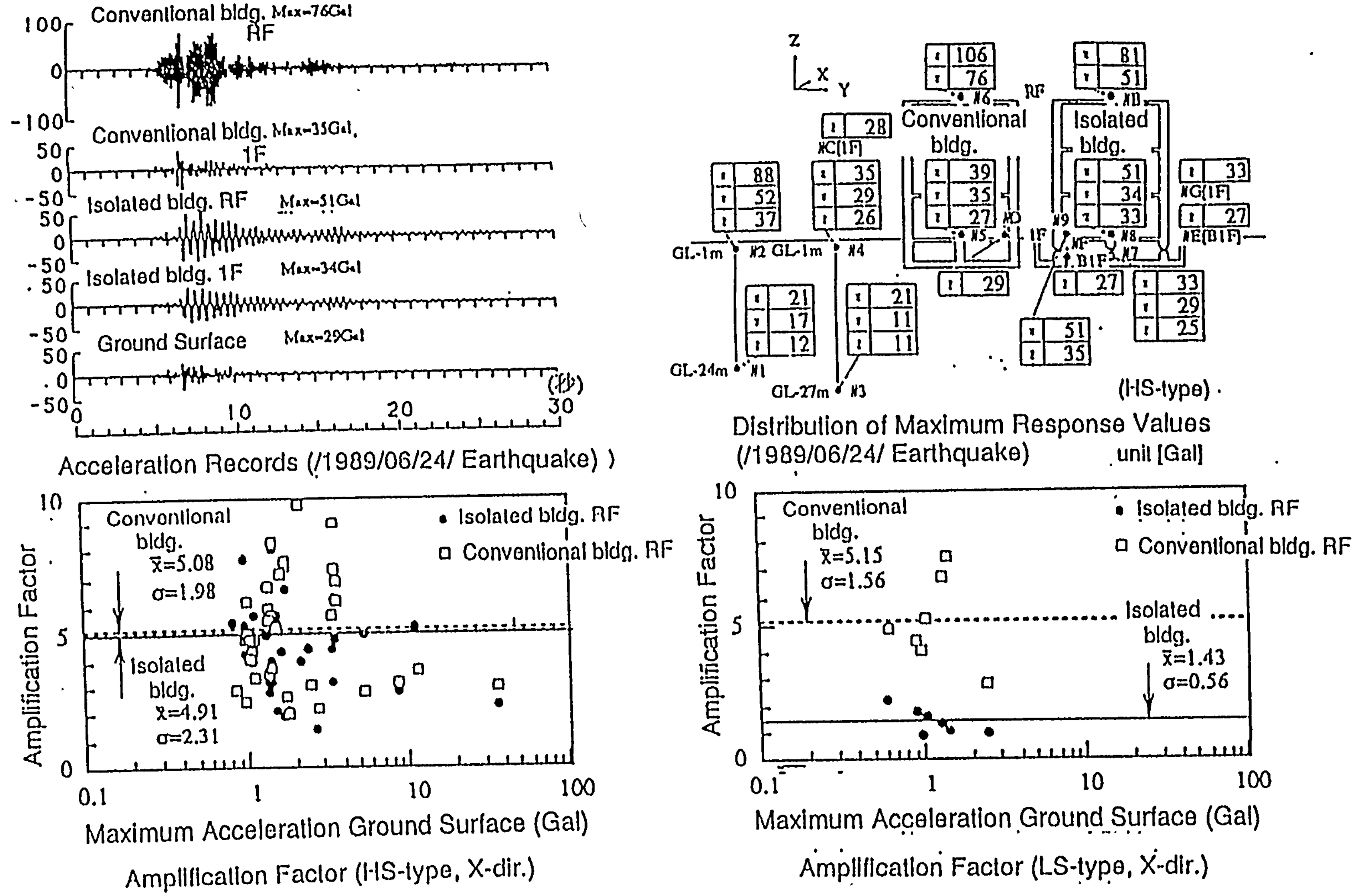
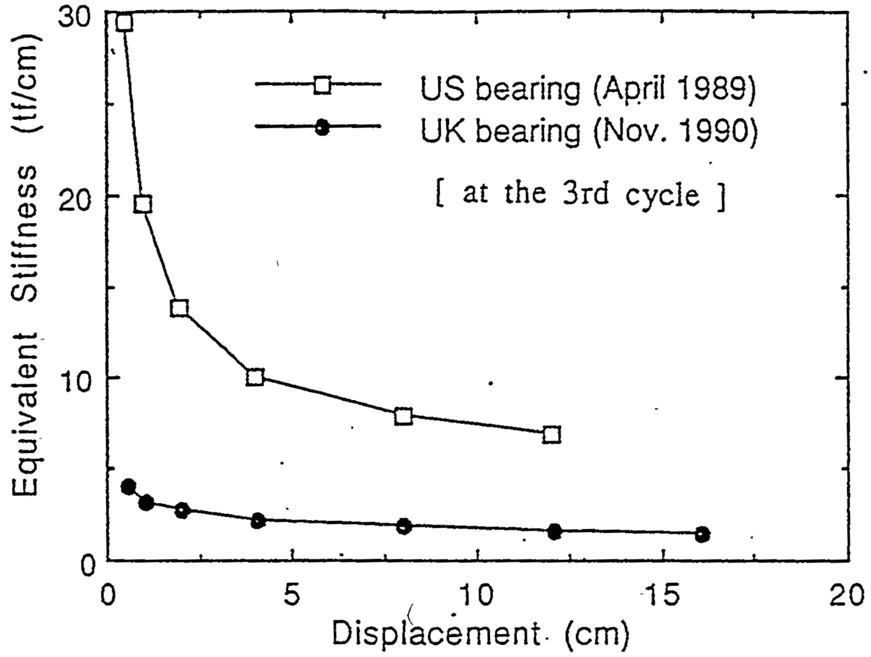
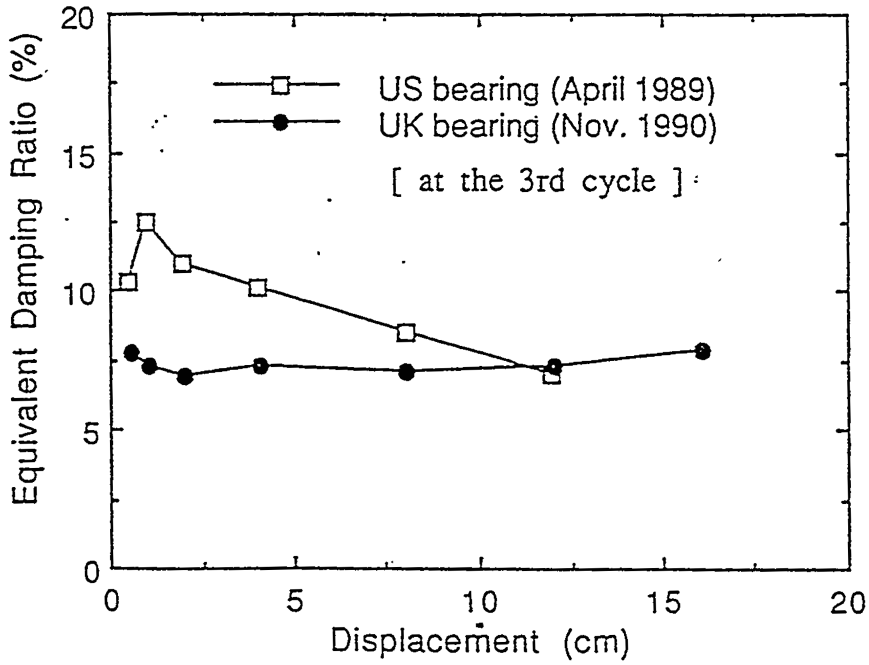


Fig. 63 Earthquake Responses of Demonstration Building



(a) Equivalent Stiffness



(b) Equivalent Damping Ratio

Fig. 64 Mechanical Characteristics of Both Bearings be below 0.15 g. The response is much less than would be expected from a fixed base structure, and at this level the computer equipment is expected to continue to function during and after a large earthquake.

There is a large conventionally designed office building next to this building. The buildings are connected by expansion joints designed for 35 cm of movement and the seismic gap between the two buildings is 40 cm. If the displacement should exceed 40 cm (which is twice the design displacement), bumpers made of hollow rubber tubes would be engaged. Both the isolated and the adjacent conventional structure are instrumented with strong motion seismographs.

The construction of the building began in March 1989 and was completed in March 1990. The isolation system proved simple to install. The bearings were all placed in three days and their base plates grouted after 6 days. The total construction cost, not including the internal equipment, was $20 million. The cost of the 40 isolators was $1 million. The building represents a significant new cost-effective design approach for buildings housing expensive and critical equipment and it is certain that many more such structures will be built in Japan in coming years.

Nuclear Structures

Seismic isolation is also gaining support in the Japanese nuclear industry. This is especially true among companies and utilities participating in the development of the advanced demonstration fast breeder reactor plant (DFBR) whose construction is expected to start in the late 1990's. At the present, an important factor inhibiting the commercialization of FBRs in Japan is the construction cost. The cost of Monju, a demonstration plant with a capacity of 280 MWe, exceeds that of commercial light water plants of the 1,000 MWe class [13]. Seismic isolation is expected to significantly reduce the cost of seismic design of FRBs and facilitate the standardization of FBR designs for a wide range of siting conditions. For example, Shiojiri et al. [14], demonstrated that the weight of a pool-type FBR reactor vessel can be reduced by half if horizontal isolation is used.

Different concepts of isolation have been evaluated [14]. These include total base isolation of the nuclear island building (NIB) or isolation of the reactor cavity in the horizontal direction (2-D) or in the vertical plus horizontal directions (3-D), or hybrid isolation where the NIB is isolated in the horizontal direction and the reactor cavity in the vertical direction. It is recognized, however, that vertical isolation is more difficult to achieve and would require more R&D. Furthermore, since all the buildings isolated in Japan use horizontal isolation only, it would be more difficult to collect performance data on vertically isolated buildings in the near future. Thus, it is expected that if seismic isolation is selected for the first DFBR, horizontal isolation only will be adopted, while keeping vertical isolation an option for future applications.

It has been recognized that extensive research and development would be required to produce isolation systems with the required reliability for nuclear applications. It is also recognized that performance experience of isolated structures must be gained in less critical structures as a precondition for accepting seismic isolation for nuclear plants. Thus, research in this field is receiving large support from the government and utilities. CRIEPI under a contract from the Ministry of International Trade and Industry (MITI), is managing a seven year research program in seismic isolation [15]. The program started in 1987 and has several objectives, including: the establishment of a seismic isolation design for the DFBR, selection of appropriate seismic isolation devices, performance of large-scale element tests of selected devices, performance of large-scale system tests on the Tadotsu shake table, construction of and observation of the earthquake response of a large-scale isolated nuclear building model, observation and cataloging of the response of all isolated buildings in Japan during earthquakes, identification of a set of design earthquakes with long-period components, and the development of design guidelines for isolated nuclear systems.

To date, a large number of reduced-scale and full-scale steel-laminated elastomeric bearings composed of high-damping and unfilled rubbers as well as lead rubber bearings have been tested to various load levels up to failure by CRIEPI [16] and other program participants [13] [17]. These tests have demonstrated the effectiveness of elastomeric bearings for isolating nuclear structures and the high reliability with which they can be manufactured in Japan. Another major achievement of this program includes the development of a tentative design response spectrum for seismically isolated FBRs [18].

Another six-year program was started in 1985 to establish a seismic isolation design for light water reactor plants (LWR). The program participants include 10 electric power companies, 3 reactor manufacturers, and 5 construction companies [19], and consists of three phases with a final objective of preparing a technical design guide for seismically isolated LWR plants.

The accumulation of response data recorded during earthquakes in isolated buildings and comparisons with the response of conventional buildings would be essential for qualifying and licensing isolation systems for nuclear facilities. Several of the isolated buildings in Japan have experienced moderate earthquakes. In all cases, the isolation systems performed as expected in reducing the ground accelerations. The highest acceleration recorded to date was in the Inorganic Material Laboratory building in Tsukuba City which is isolated using a system designed by Ohbayashi Corp. consisting of elastomeric bearings and steel dampers. The maximum acceleration recorded in the basement of 0.5 g, and in the building the acceleration was reduced of 0.05 g.

Research - Tohoku Test Facility

The demonstration building at Tohoku University, described in detail in the earlier section, has been used to study two further types of high-damping isolators. The design, manufacture, and procurement of these isolators was funded by National Science Foundation (NSF) and carried out as a joint program by Argonne National Laboratory, EERC, and the Shimizu Corporation.

A set of high-damping isolators using the compound used in both the Foothills Community Law and Justice Center (FCLJC) building and the Los Angeles Fire Command and Control Center (FCCC) building, and modelled on the isolators developed for the PRISM reactor of General Electric Corp., were installed from the period April 1989 to November 1990. These isolators have a high-shape factor and the compound is very nonlinear being very stiff at small strains. The isolators were designed to give a frequency of 0.75 Hz at a shear strain of 50%. Over the period of observation, 37 earthquakes were recorded, but all provided very small levels of input generating extremely small displacements in the isolators and resulting in high stiffness in the isolators. The observed frequency of the isolated building at these levels of input is around 2.5 Hz which when compared with the frequency of the unisolated buildings, around 3.5 Hz, shows that the these levels of input, no isolation effect is achieved. This is also shown by the fact that the amplification factors for both the isolated and unisolated buildings is about the same.

During November 1990, this system was replaced by a new set of isolators which use a new, soft high-damping compound that was developed for the isolation of light structures or for systems that need a very long period. The new compound and isolators using it were tested in a shake table study at EERC. These isolators will remain in place at the demonstration building until October 1991. Since their installation, there have been 7 earthquakes but again, all of low level. However, the new compound is less nonlinear than the stiffer compound the stiffness is not so high at small strains, and some isolation effect has been observed. The amplification factor of the isolated building under small earthquakes is about one fourth that of the unisolated building.

References

[1] M. Izumi, *State of the Art - Passive and Active Control of Building Vibrations in Japan (June 1988)*, Tohoku University, Sendai, Japan (June 1988).

[2] H. Tada, "The Research Study of Aseismic Isolation System by the Enforcement Construction: Parts 1-13," *Proceedings*, Annual Meeting of the Architectural Institute of Japan, 1983-1988.

[3] I. Abe, S. Otsuka, and O. Harada, "A Study of the Dynamic Behavior of a Base Isolated Structure (Parts 1, 2)," *Proceedings*, Annual Meeting of the Architectural Institute of Japan (1987).

[4] S. Aoyagi and O. Harada, "Experimental Study on Reliability of Base Isolated Building," *Proceedings*, 7th Japanese Earthquake Engineering Symposium, pp. 1591-1596 (1986).

[5] T. Takeda, H. Okada, T. Tsunoda, T. Nakamura, M. Seki, H. Uchida, and A. Teramura, "Study on Earthquake Base Isolation System for Structures: Parts 1-12," *Proceedings*, Annual Meeting of the Architectural Institute of Japan, Technical Research Institute, Ohbayashi Corp. (1985-1988).

[6] I. Shimoda, S. Nakano, Y. Kitagawa, and M. Miyazaki, "Experimental Study on Base Isolated Building Using Lead Rubber Bearing Through Vibration Tests, *Technical Report*, Sumitomo Construction Company.

[7] H. Koshida, A. Yasaka, M. Izuka, T. Yano, N. Fujimoto, and K. Andow, "Vibration Tests of the Base Isolated Building," *Proceedings*, 7th Japanese Earthquake Engineering Symposium, pp. 1633-1638, Tokyo, Japan (1986).

[8] M. Higashino, S. Aizawa, and Y. Hayamizu, "An Experimental Study of A Base-Isolation System Using Laminated Rubber Bearings and Viscous Dampers," *Takenaka Technical Research Report*, Takenaka Technical Research Laboratory, pp. 155-163, Japan (November 1987).

[9] *The Base Isolation System in Funabashi Chikuyuryo Dormitory and Earthquake Records, April 1987*, Takenaka Technical Research Laboratory, Japan (May 1988).

[10] H. Tada, M. Takayama, and Y. Mae, *Earthquake Shock Absorbers*, Mitsubishi Metal Corp., Tokyo, Japan (December 1987).

[11] H. Yamahara and M. Izumi, *Actual Proof Tests of the Base-Isolated Building Using Full-Sized Model*, Shimizu Construction Corp., Tohoku University, Japan

(August 1987).

[12] I. Nagashima, S. Kawamura, K. Kitazawa, and M. Hisano, "Study on a Base Isolation System," *Proceedings*, 3rd International Conference on Soil Dynamics and Earthquake Engineering, Princeton University, Princeton, NJ (June 1987).

[13] T. Fujita, "Seismic Isolation Rubber Bearings for Nuclear Facilities," *Proceedings*, 1st Int. Seminar on Seismic Base Isolation of Nuclear Power Facilities, San Francisco (1989).

[14] H. Shiojiri, "CRIEPI Test Program for Seismic Isolation of FBR." *Proceedings*, 1st Int. Seminar on Seismic Base Isolation of Nuclear Power Facilities, San Francisco (1989).

[15] Y. Sawada et al., "Seismic Isolation Test Program," *Proceedings*, SMiRT-10, Vol. K2, Anaheim (August 1989).

[16] T. Mazda et al, "Test on Large-Scale Seismic Isolation Elements," *Proceedings*, SMiRT-10, Vol. K2, Anaheim (August 1989).

[17] T. Fujita et al, "Research, Development and Implementation of Rubber Bearings for Seismic Isolation," *Seismic, Shock, and Vibration Isolation*, ASME PVP, Vol. 181 (1989).

[18] K. Ishida et al, "Tentative Design Response Spectrum for Seismically Isolated FBR," *Proceedings*, SMiRT-10, Vol. K2, Anaheim (1989).

[19] M. Kato, S. Sato, and I. Shimomura, "Utilities/Industries Joint Study on Seismic Isolation Systems for LWR (Part I) Experimental and Analytical Studies on Seismic Isolation Systems", *Proceedings*, 1st Int. Seminar on Seismic Base Isolation of Nuclear Power Facilities, San Francisco (1989).

Appendix

Additional buildings that have been completed or have obtained licenses for construction.

No.	Name of Building	Site of Const.	Year *1	Structure *2	Uses	Isol. (φ) *3	Damper *4	N.Per. (sec)	Remarks *5
56	Dainihon-doboku Ichigao Dorm	Kanaga-wa	1990	RC +4	Dormitory	NI 500-600	LD	2.6 (0.9)	(own)
57	ENICOM Computer Center	Tokyo	1990	S +6	Office Cp.Room	NI 800	HD+LD	2.8 (1.2)	
58	Chubu E.P.Corp. East Bldg.	Aichi	1991	SRC +6	Office	LI 800 -1100	--	2.9 (2.2)	
59	DOMANI Musashino	Tokyo	1991	RC +3	Apartment	NI 530	HD	1.3	
60	Oiles Industry Araijuku Dorm	Tokyo	1991	W +2	Dormitory	LI 260	--	2.5 (2.0)	(own)
61	Sato Corp. Urawa Dorm	Saitama	1991	RC +5	Dormitory	NI 760,800	HD	2.0 (1.2)	(own)
62	Shinetsu Chemistry Guard House	Gunma	1991	RC +2	Guard House	Silicone. 600	--	2.6 (1.9)	
63	Mitsui Corp. Kashiwa Dorm	Chiba	1991	RC +3	Dormitory	HI	HD	2.1 (1.1)	(own)
65	Fujita Corp. Tech.Lab No6 Red	Kanaga-wa	1991	RC +3	Laboratory	NI	LD	1.8 (1.3)	(own)

Note *1 Year of Design : The year the technical evaluation of the BCJ was obtained.
*2 RC : Reinforced Concrete Struc. SRC : Composite Struc. S : Steel Struc. W : Wooden Str.
*3 NI : Natural Rubber Iso. LI : Lead Rubber Iso. HI : High-Damping Iso. φ : Rubber Dia.
*4 HD : Steel Hysteresis Damper VD : Viscous Damper LD : Lead Damper FD : Friction Damper
*5 (own) : Building for company's own use

LECTURE 3

Linear Theory of Base Isolation

by

James M. Kelly
Professor of Civil Engineering
Earthquake Engineering Research Center
University of California
Berkeley, California 94720

Short Course on Seismic Base Isolation
Department of Energy

August 10 - August 14, 1992

Department of Energy **J.M. Kelly**
Short Course on Seismic Base Isolation **August 10-14, 1992**

LECTURE 3

Linear Theory of Base Isolation

Vibration isolation is a very highly developed technical area that has many elements in common with seismic isolation. However, seismic isolation differs from vibration isolation several ways. Base flexibility of the isolated structure must be taken into account and there will be a base slab of some sort on which the building is mounted, the mass of which must be included in the model. In vibration isolation, we are concerned with protection against a specified range of frequencies and displacements in the mounting system are generally negligible. In seismic isolation, the input against which the protection is sought is much less well-prescribed and displacements of the isolation system can be very large.

An elementary analysis for the purpose of gaining insight into the behavior of isolated buildings can be developed using a simple linear two-degree-of-freedom model with linear springs and linear viscous damping. Since most isolation systems are intrinsically nonlinear, this analysis will be only approximate for such systems, and the effective stiffness and damping will have to be estimated by some equivalent linearization process. The parameters of the model are shown in Fig. 3-1. The absolute equations of motion are:

$$m \ddot{u} = -c (\dot{u} - \dot{u}_b) - k (u - u_b) \tag{3.1}$$

and

$$m \ddot{u} + m_b \ddot{u}_b = -c_b (\dot{u}_b - \dot{u}_g) - k (u_b - u_g) \tag{3.2}$$

It is convenient to work with relative displacements:

$$v_s = u - u_b \tag{3.3}$$

and

$$v_b = u_b - u_g \tag{3.4}$$

in terms of which the equations of motion become:

$$m\,\ddot{v}_b + m\,\ddot{v}_s + c\,\dot{v}_s + k\,v_s = -m\,\ddot{u}_g \tag{3.5}$$

and

$$(m + m_b)\,\ddot{v}_b + m\,\ddot{v}_s + c_b\,\dot{v}_b + k_b\,v_b = -(m + m_b)\,\ddot{u}_g \tag{3.6}$$

It is easy to see that if the relative motion between the structure and the base, expressed by v_s, is suppressed, the second equation becomes the single-degree-of-freedom equation:

$$M\,\ddot{v}_b + c_b\,\dot{v}_b + k_b\,v_b = -M\,\ddot{u}_g \tag{3.7}$$

where M is the total weight of the building plus that of the slab. If v_b is suppressed, the first equation becomes the usual equation for a fixed-base single-degree-of-freedom system.

This two-degree-of-freedom system of equations can be solved directly or through modal decomposition. A modal analysis provides insight into the response of isolated systems and the results will be applicable to more elaborate models. To develop the modes, frequencies, and participation factors of the system, we write the equations in the matrix form:

$$\mathbf{M}^*\,\ddot{\mathbf{v}}^* + \mathbf{C}^*\,\dot{\mathbf{v}}^* + \mathbf{K}\mathbf{v}^* = -\mathbf{M}^*\,\mathbf{r}^*\,\ddot{u}_g \tag{3.8}$$

where

$$\mathbf{M}^* = \begin{bmatrix} M & m \\ m & m \end{bmatrix} \quad \mathbf{C}^* = \begin{bmatrix} c_b & 0 \\ 0 & c \end{bmatrix} \quad \mathbf{K}^* = \begin{bmatrix} k_b & 0 \\ 0 & k \end{bmatrix}$$

$$\mathbf{v}^* = \begin{Bmatrix} v_b \\ v_s \end{Bmatrix} \quad \mathbf{r}^* = \begin{Bmatrix} 1 \\ 0 \end{Bmatrix}$$

and make the following order of magnitude estimates. We assume that:

(i) $m_b < m$, but are of the same order of magnitude;

(ii) $\omega_s = \left(\dfrac{k}{m}\right)^{1/2} >> \omega_b = \left(\dfrac{k_b}{M}\right)^{1/2}$ and define $\varepsilon = \left(\dfrac{\omega_b}{\omega_s}\right)^2$ and take ε to be of order of magnitude 10^{-2};

(iii) the damping factors for the structure and isolation system, β_s and β_s, respectively, where $\beta_s = \dfrac{c}{2m\,\omega_s}$ and $\beta_b = \dfrac{c_b}{2M\,\omega_b}$, are of the same order of magnitude as ε.

The undamped natural modes of the system

$$\phi^n = \{\phi_b^n, \phi_s^n\}^T \quad ; \quad n = 1,2$$

are given by the two equations:

$$(-\omega_n^2 + \omega_b^2)\,\phi_b^n + (-\gamma\,\omega_n^2)\,\phi_s^n = 0 \tag{3.9a}$$

and

$$(-\omega_n^2)\,\phi_b^n + (-\omega_n^2 + \omega_s^2)\,\phi_s^n = 0 \tag{3.9b}$$

where ω_n is the frequency of the mode and $\gamma = m/M$ is a mass ratio, less than one.

The characteristic equation for ω_n is

$$(1-\gamma)\,\omega_n^4 - (\omega_b^2 + \omega_s^2)\,\omega_n^2 + \omega_b^2\,\omega_s^2 = 0 \tag{3.10}$$

The lower of the two roots, ω_1 and ω_2, of this equation will be denoted by ω_b^* which represents the shifted isolation frequency, and the higher root by ω_s^*, which represents the structural frequency modified by the presence of the isolation system. The exact roots are given by

$$\begin{matrix}\omega_2^2 \\ \omega_1^2\end{matrix} = \frac{1}{2(1-\gamma)}\left\{(\omega_s^2+\omega_b^2) \pm \left[\,(\omega_s^2+\omega_b^2)^2 - 4\,(1-\gamma)\,\omega_s^2\,\omega_b^2\right]^{1/2}\right\} \tag{3.11}$$

When we account for the fact that $\omega_b << \omega_s$ and rewrite the radical in the form

$$(\omega_s^2 - \omega_b^2)^2\left[1 + 4\gamma\,\frac{\omega_b^2\omega_s^2}{(\omega_s^2-\omega_b^2)^2}\right] \tag{3.12}$$

and expand this by binomial series, we obtain, to the same order of ε, the results:

$$\omega_1^2 = \omega_b^{*2} = \omega_b^2 \left[1 - \gamma \frac{\omega_b^2}{\omega_s^2} \right] \tag{3.13}$$

and

$$\omega_2^2 = \omega_s^{*2} = \frac{\omega_s^2}{1-\gamma} \left[1 + \gamma \frac{\omega_b^2}{\omega_s^2} \right] \tag{3.14}$$

In many cases it may be sufficiently accurate to take as approximations for ω_b^* , ω_s^* the first terms:

$$\omega_b^* = \omega_b \tag{3.15}$$

$$\omega_s^* = \frac{\omega_s}{(1-\gamma)^{1/2}} \tag{3.16}$$

This indicates that the isolation frequency is only slightly changed by flexibility in the structure (the change is of order ε), while the structural frequency is significantly increased by the presence of the base mass. The separation between the isolation frequency and the fixed-base structural frequency is increased by combining the two elements.

The mode shape, ϕ^1, is given by:

$$(-\omega_b^{*2} + \omega_b^2)\, \phi_b^1 + (-\gamma\, \omega_b^{*2})\, \phi_s^1 = 0 \tag{3.17a}$$

or

$$-\omega_b^{*2} \phi_b^1 + (\omega_s^2 - \omega_b^{*2}) \phi_s^1 = 0 \tag{3.17b}$$

and, retaining terms of order ε and setting $\phi_b^1 = 1$, we get:

$$\phi^1 = \begin{Bmatrix} 1 \\ \omega_b^2/\omega_s^2 \end{Bmatrix} \tag{3.18}$$

To the same order of ε, we find that:

$$\phi^2 = \begin{Bmatrix} 1 \\ -\dfrac{1}{\gamma} \left[-(1-\gamma) \dfrac{\omega_b^2}{\omega_s^2} \right] \end{Bmatrix} \tag{3.19}$$

These are sketched in Fig. 3-2 and show that ϕ^1 is approximately a rigid structure mode, whereas ϕ^2 involves both structural deformation and isolation system deformation. The displacement of the top of the structure is of the same order as the base displacement, but opposite in direction.

With these two modes, ϕ^1 and ϕ^2, we can write the relative displacements, v_b and v_s, in the form:

$$v_b = q_1\phi_b^1 + q_2\phi_b^2 \tag{3.20}$$

and

$$v_s = q_1\phi_s^1 + q_2\phi_s^2 \tag{3.21}$$

and reduce the basic matrix equation to the two equations:

$$\ddot{q}_1 + 2\omega_b^*\beta_b^*\dot{q}_1 + \omega_b^{*2}q_1 = -L_1\ddot{u}_g \tag{3.22}$$

and

$$\ddot{q}_2 + 2\omega_s^*\beta_s^*\dot{q}_2 + \omega_s^{*2}q_2 = -L_2\ddot{u}_g \tag{3.23}$$

where we have implicitly assumed that the damping in the system is light enough to allow us to retain the orthogonality of the modes. In these equations, L_1 and L_2 are the participation factors for the two modes. These are given by

$$L_n = \frac{\phi^{n^T}\mathbf{M}^*\,\mathbf{r}^*}{\phi^{n^T}\mathbf{M}^*\,\phi^n} \tag{3.24}$$

The computation of L_1 involves the following matrix multiplications

$$L_1M_1 = (1,\varepsilon)\begin{bmatrix} M & m \\ m & m \end{bmatrix}\begin{bmatrix} 1 \\ 0 \end{bmatrix} = M + m\,\varepsilon \tag{3.25}$$

where

$$M_1 = (1,\varepsilon)\begin{bmatrix} M & m \\ m & m \end{bmatrix}\begin{bmatrix} 1 \\ \varepsilon \end{bmatrix} = M + 2m\,\varepsilon + m\,\varepsilon^2$$

Retaining only terms to order ε, we have:

$$L_1 = 1 - \gamma\varepsilon \tag{3.26}$$

For L_2, the same computations give:

$$L_2 M_2 = M + m\,a \tag{3.27}$$

where

$$M_2 = M + 2m\,a + a^2 m$$

and

$$a = -\frac{1}{\gamma}\left[1 - (1-\gamma)\,\varepsilon\right]$$

Since

$$\gamma = \frac{m}{M},\ L_2 M_2 = M\,(1-\gamma)\,\varepsilon$$

and

$$M_2 = M\ \frac{(1-\gamma)\left[1-2(1-\gamma)\,\varepsilon\right]}{\gamma}$$

then

$$L_2 = \gamma\,\varepsilon \tag{3.28}$$

This result and that for the shifted frequencies reveal the underlying reasons for the effectiveness of a seismic isolation system. The participation factor for the second mode, which is the mode that involves structural deformation, is of order ε and if the original frequencies, ω_b, ω_s, are well separated, this is very small. In addition, the frequency of this mode is shifted to a higher value than the original fixed-base frequency and if the earthquake input has large spectral accelerations at the original structural frequency, shifting it higher could shift it out of the range of strong earthquake motion. Moreover, since the participation factor for this mode is very small, this mode is orthogonal to the earthquake input characterized by $\mathbf{M}^* \mathbf{r}^* \ddot{u}_g$. This means that even if the earthquake does have energy at this frequency, the ground motion will not be transmitted into the structure. A seismic isolation system works not by absorbing energy; it deflects energy through this property of orthogonality.

Energy absorption is of course an important part of the behavior of an isolation system and in this simple model we have represented it by linear viscous damping. Additionally, we have assumed that the uncoupled equations still hold. The question now arises as to how to select the modal damping factors, β_b^*, β_s^*. In this case, we are in the unusual position of being able to make very good estimates of the damping in each element when treated separately. An isolation system based on a laminated rubber will provide a degree of damping that is in the range of 10-20% of critical damping. The structure will have significantly less, probably of the order of 2%. In conventional structural analysis, it is generally assumed that the damping in a structure will be 5% of critical damping, but this reflects the fact that some degree of structural and non-structural damage is presumed to occur when a conventional structure experiences strong ground motion. In a base-isolated building, the aim is to reduce the forces experienced by the structure to such a level that no damage to the structure or to nonstructural elements such as partitions will occur and thus a lower value for the damping is appropriate.

In any discretized structural dynamics problem, the equations of motion can be written in the form:

$$\mathbf{M}\ddot{\mathbf{x}} + \mathbf{C}\dot{\mathbf{x}} + \mathbf{K}\mathbf{x} = \mathbf{P} \tag{3.29}$$

where $\mathbf{P}$ is the load vector. Multiplying by $\dot{\mathbf{x}}$ and summing over all components gives

$$\dot{\mathbf{x}}^T \mathbf{M}\ddot{\mathbf{x}} + \dot{\mathbf{x}}^T \mathbf{C}\dot{\mathbf{x}} + \dot{\mathbf{x}}^T \mathbf{K}\mathbf{x} = \dot{\mathbf{x}}^T \mathbf{P} \tag{3.30}$$

which can be interpreted as:

$$\frac{d}{dt}(K.E. + P.E.) = \dot{W} - D \tag{3.31}$$

where

$K.E. = \frac{1}{2}\dot{\mathbf{x}}^T \mathbf{M}\dot{\mathbf{x}}$	is the kinetic energy;
$P.E. = \frac{1}{2}\mathbf{x}\mathbf{K}\mathbf{x}$	the potential energy;
$\dot{W} = \dot{\mathbf{x}}^T \mathbf{P}\&$	the external work rate; and
$D = \dot{\mathbf{x}}^T \mathbf{C}\dot{\mathbf{x}}$	is the dissipation of energy.

The modal representation of this equation using $\mathbf{x} = \sum_{i=1}^{N} q_i \, \phi^i$, where N is the order of the system, is:

$$\ddot{q}_n + 2\omega_n \beta_n \dot{q}_n + \omega_n^2 q_n = p_n \tag{3.32}$$

where the ϕ^n are given by:

$$\omega_n^2 \mathbf{M} \phi^n = \mathbf{K} \phi^n$$

The global quantities take the form:

$$K.E. = \frac{1}{2} \sum_{i=1}^{N} \dot{q}_n^2 \tag{3.33}$$

and:

$$P.E. = \frac{1}{2} \sum_{i=1}^{N} \omega_n^2 q_n^2 \tag{3.34}$$

and

$$\dot{W} = \sum_{i=1}^{N} p_n \, \dot{q}_n \tag{3.35}$$

where it has been assumed that the ϕ^n are normalized such that:

$$\phi^{n^T} \mathbf{M} \phi^m = \delta_{mn} \tag{3.36}$$

with

$$\delta_{mn} = \begin{cases} 1 & m = n \\ 0 & m \neq n \end{cases}$$

On the other hand, the quantity $\dot{\mathbf{x}}^T \mathbf{C} \dot{\mathbf{x}}$ is, in terms of ϕ^n,

$$\sum_{m=1}^{N} \sum_{n=1}^{N} \dot{q}_m \, \dot{q}_n \, A_{mn} \tag{3.37}$$

where $A_{mn} = \phi^{m^T} \mathbf{C} \phi^n$.

To preserve the orthogonality properties of the modal representation we need to replace $\dot{\mathbf{x}}^T \mathbf{C} \dot{\mathbf{x}}$ above by the single summation term $\sum_{n=1}^{N} 2\omega_n \beta_n \dot{q}_n^2$, and the problem is then to select β_n knowing ω_n, ϕ_n to minimize the difference between:

$$D = \sum_m \sum_n \dot{q}_m \dot{q}_n A_{mn} \tag{3.38}$$

and

$$D = \sum_n 2\omega_n \beta_n \dot{q}_n^2 \tag{3.39}$$

It can be shown that the optimum choice of β_n is given by

$$2\omega_n \beta_n = \phi^{n^T} \mathbf{C} \phi^n \tag{3.40}$$

The corresponding terms in the simple isolation model are:

$$2\omega_n^* \beta_n^* = \frac{\phi^{n^T} \mathbf{C}^* \phi^n}{\phi^{n^T} \mathbf{M}^* \phi^n} \tag{3.41}$$

which is equivalent to neglecting the off-diagonal terms of $\phi^{n^T} \mathbf{C} \phi^m$, which would couple the equations of motion.

When we utilize the previous results for M_1 and M_2, we find that

$$2\omega_b^* \beta_b^* = 2\omega_b \beta_b (1 - 2\gamma\varepsilon) \tag{3.42}$$

and

$$2\omega_s^* \beta_s^* = \frac{2\omega_s \beta_s + \gamma 2\omega_b \beta_b}{1 - \gamma} \tag{3.43}$$

Using the result that

$$\omega_b^* = \omega_b (1 - \gamma\varepsilon)^{1/2} \tag{3.44}$$

we have

$$\beta_b^* = \frac{\beta_b (1 - 2\gamma\varepsilon)}{(1 - \gamma\varepsilon)^{1/2}} = \beta_b (1 - \frac{3}{2}\gamma\varepsilon) \tag{3.45}$$

Since

$$\omega_s^* = \omega_1 \frac{(1+\gamma\varepsilon)^{1/2}}{(1-\gamma)^{1/2}} \tag{3.46}$$

we have

$$\beta_s^* = \left[\frac{\beta_s}{(1-\gamma)^{1/2}} + \frac{\gamma\beta_b\varepsilon^{1/2}}{(1-\gamma)^{1/2}}\right](1 - \frac{1}{2}\gamma\varepsilon) \tag{3.47}$$

which shows that the structural damping is increased by the damping in the bearings to the order of $\varepsilon^{1/2}$; the product of β_b and $\varepsilon^{1/2}$ may be a significant addition to the term β_s and could be important if β_s is very small. This shows that high damping in the rubber bearings can contribute significant damping to the structural mode.

With these results for L_1, L_2, β_1, β_2, we are able to estimate the response of the system to specific earthquake inputs. First, let us consider the response to sinusoidal input and look at the amplification factor defined by the magnitude of the ratio of relative displacement to ground displacement. We denote

$$A_s = \left|\frac{v_s}{u_g}\right| \quad , \quad A_b = \left|\frac{v_b}{u_g}\right| .$$

and set

$$\ddot{u}_g = \omega^2 \bar{u}_g \, e^{i\omega t}$$

in the modal equations. We then have

$$A_s = \left|\frac{v_s}{u_g}\right| = \left|\phi_s^1 \frac{L_1\omega^2}{(\omega_b^{*2}-\omega^2)+i2\omega_b^*\omega\beta_b^*} + \phi_s^2 \frac{L_2\omega^2}{(\omega_s^{*2}-\omega^2)+i2\omega_s^*\omega\beta_s^*}\right| \tag{3.48}$$

and

$$A_b = \left|\frac{v_b}{u_g}\right| = \left|\phi_b^1 \frac{L_1\omega^2}{(\omega_b^{*2}-\omega^2)+i2\omega_b^*\omega\beta_b^*} + \phi_b^2 \frac{L_2\omega^2}{(\omega_s^{*2}-\omega^2)+i2\omega_s^*\omega\beta_s^*}\right| \tag{3.49}$$

Although these expressions can be written out as algebraic functions of ω without imaginary parts, it is more instructive to look at the form that they take for specific values of ω. We will be interested in three frequencies, namely the fixed-base structure frequency, ω_s, and the two isolated frequencies, ω_b^*, ω_s^*. When $\omega = \omega_b^*$, we have

$$A_s = \left| \varepsilon \frac{(1-\gamma\varepsilon)\,\omega_b^{*2}}{i\,2\omega_b^{*2}\beta_b^*} - \frac{1}{\gamma} \frac{[1-(1-\gamma)\varepsilon]\gamma\varepsilon\omega_b^{*2}}{(\omega_s^{*2}-\omega_b^{*2})+i\,2\omega_s^*\omega_b^*\beta_s^*} \right| \tag{3.50}$$

Retaining terms to the first order in ε, this gives:

$$A_s = \frac{\varepsilon}{2\beta_b^*} \tag{3.51}$$

For the amplification of base motion we have:

$$A_b = \left| \frac{1-\gamma\varepsilon}{2\beta_b^*} + \frac{\gamma\varepsilon^2}{(1-\varepsilon)+i\,\varepsilon} \right| \tag{3.52}$$

which to the same order in ε is:

$$A_b = \frac{1-\gamma\varepsilon}{2\beta_b\,(1-\frac{3}{2}\gamma\varepsilon)} = \frac{1}{2\beta_b}\,(1+\frac{1}{2}\gamma\varepsilon) \tag{3.53}$$

When $\omega = \omega_s^*$ we find by the same manipulations that:

$$A_s = (1-(1-\gamma)\,\varepsilon)\frac{\varepsilon}{2\beta_s^*} \tag{3.54}$$

and

$$A_b = 1 + \frac{1}{2}\frac{\gamma^2\varepsilon^2}{(2\beta_s^*)^2} \tag{3.55}$$

At the fixed-base frequency, the amplification factor, A_s, is actually of order ε and A_b is of order 1. These three sets of results are indicated in Table 3-1.

	Table 3-1		
ω	A	A_s	A_b
	Fixed-Base Structure	Isolated Structure	Isolated Base
ω_b^*	--	$0(1)$	$\frac{1}{2\beta_0} = 0(\frac{1}{\varepsilon})$
ω_s	$\frac{1}{2\beta_1} = 0(\frac{1}{\varepsilon})$	$0(\varepsilon)$	$0(1)$
ω_s^*	$0(\varepsilon)$	$0(1)$	$0(1)$

The exact expressions for all values of ω are:

$$A_s = \omega^2\varepsilon\left[\frac{(\omega_s^{*2}-\omega_b^{*2})^2+\omega^2(2\omega_s^*\beta_s^*-2\omega_b^*\beta_b^*)^2}{[(\omega_b^{*2}-\omega^2)^2+4\omega_b^{*2}\omega^2\beta_b^{*2}][(\omega_s^{*2}-\omega^2)^2+4\omega_s^{*2}\omega^2\beta_s^{*2}]}\right]^{1/2} \tag{3.56}$$

and

$$A_b = \omega^2\left[\frac{(\omega_s^{*2}-\omega_b^{*2})^2+\gamma\varepsilon\omega^2(2\omega_s^*\beta_s^*-2\omega_b^*\beta_b^*)^2}{[(\omega_b^{*2}-\omega^2)^2+4\omega_b^{*2}\omega^2\beta_b^{*2}][(\omega_s^{*2}-\omega^2)^2+4\omega_s^{*2}\omega^2\beta_s^{*2}]}\right]^{1/2} \tag{3.57}$$

These expressions have been evaluated for a few choices of the parameters $\gamma, \varepsilon, \beta_b^*, \beta_s^*$ and the results are shown in the Fig. 3-3.

If the time history of the ground motion, $\ddot{u}_g(t)$, is known, then the modal components $q_1(t), q_2(t)$ can be computed from:

$$q_1 = -\frac{L_1}{\omega_b^*}\int_0^t \ddot{u}_g(t-\tau)\, e^{-\omega_b^*\beta_b^*\tau}\sin\omega_b^*\tau\, d\tau \tag{3.58}$$

and

$$q_2 = -\frac{L_2}{\omega_s^*}\int_0^t \ddot{u}_g(t-\tau)\, e^{-\omega_s^*\beta_s^*\tau}\sin\omega_s^*\tau\, d\tau \tag{3.59}$$

and estimates of the maximum values of q_1 and q_2 are given by:

$$|q_1|_{\max} = L_1 S_D\,(\omega_b^*, \beta_b^*) \tag{3.60}$$

and

$$|q_2|_{\max} = L_2 S_D\,(\omega_s^*, \beta_s^*) \tag{3.61}$$

where $S_D\,(\omega, \beta)$ is the displacement response spectrum for the ground motion, $\ddot{u}_g(t)$, at frequency, ω and damping factor, β.

If for purposes of design we are given a design spectrum which is generally a damped acceleration response spectrum (e.g., Nuclear Regulatory Commission Regulatory Guide 16), the predicted relative displacement maxima will be

$$|v_s|_{\max} = \left\{ (\phi_s^1 q_{1\max})^2 + (\phi_s^2 q_{2\max})^2 \right\}^{1/2} \tag{3.62}$$

and

$$|v_b|_{\max} = \left\{ (\phi_b^1 q_{1\max})^2 + (\phi_b^2 q_{2\max})^2 \right\}^{1/2} \tag{3.63}$$

where now

$$q_{1\max} = L_1 \frac{1}{\omega_b^{*2}} S_A\,(\omega_b^*, \beta_b^*)$$

and

$$q_{2\max} = L_2 \frac{1}{\omega_s^{*2}} S_A\,(\omega_s^*, \beta_s^*)$$

with $S_A\,(\omega, \beta)$ the acceleration design spectrum at ω and β. Thus

$$|v_s|_{\max} = \left\{ \varepsilon^2(1-\gamma\varepsilon)^2 \frac{S_A^2(\omega_b^*, \beta_b^*)}{\omega_b^{*4}} + \frac{1}{\gamma^2}(1-(1-\gamma)\varepsilon)^2\gamma^2\varepsilon^2\,(1-2\gamma\varepsilon)^2 \frac{S_A^2(\omega_s^*, \beta_s^*)}{\omega_s^{*4}} \right\}^{1/2} \tag{3.64}$$

and

$$|v_b|_{\max} = \left\{ \frac{(1-\gamma\varepsilon)^2 S_A^2(\omega_b^*, \beta_b^*)}{\omega_b^{*4}} + \frac{\gamma^2\varepsilon^2(1-2\gamma\varepsilon)^2 S_A^2(\omega_s^*, \beta_s^*)}{\omega_s^{*4}} \right\}^{1/2} \tag{3.65}$$

Many design spectra are approximately constant velocity spectra and in such cases, the values of S_A for different frequencies, neglecting variations due to damping, are related by:

$$S_A\,(\omega\,,\beta) = \omega\, S_V \tag{3.66}$$

where S_V is a constant. For such design spectra we have:

$$|v_s|_{\max} = \varepsilon \left\{ (1-\gamma\varepsilon)^2 \frac{\omega_b^{*2}}{\omega_b^{*4}} + (1-(1-\gamma)\,\varepsilon)^2 \frac{\omega_s^{*2}}{\omega_s^{*4}} \right\}^{1/2} S_V = \varepsilon\, \frac{S_V}{\omega_b} = \varepsilon\, S_D\,(\omega_b\,,\beta_b) \tag{3.67}$$

and

$$|v_b|_{\max} = \left\{ \frac{(1-\gamma\varepsilon)^2\,\omega_b^{*2}}{\omega_b^{*4}} + \frac{\gamma\varepsilon^2\omega_s^{*2}}{\omega_s^{*4}} \right\}^{1/2} S_V = \frac{S_V}{\omega_b} = S_D(\omega_b\,,\beta_b) \tag{3.68}$$

This shows that for a constant velocity spectrum the relative displacement in the structure, i.e., the interstory drift, is of order ε compared to the isolation system displacement. The design base shear coefficient, C_s, is defined by:

$$C_s = \frac{k_s\, v_s}{m} = \omega_s^2\, v_s \tag{3.69}$$

and when unisolated this is

$$C_s = \omega_s^2 S_D\,(\omega_s\,,\beta_s) = S_A\,(\omega_s\,,\beta_s) \tag{3.70}$$

When isolated this becomes:

$$C_s = \omega_s^2 \left\{ \frac{\varepsilon^2(1-\gamma\varepsilon)^2 S_A^2\,(\omega_b^*\,,\beta_b^*)}{\omega_b^{*4}} + \frac{\varepsilon^2\,[1-(1-\gamma)\varepsilon]^2 S_A^2\, 6\omega_s^*\,,\beta_s^*)}{\omega_s^{*4}} \right\}^{1/2} \tag{3.71}$$

Recalling that $\omega_b^{*2} = \omega_b^2(1-\gamma\varepsilon)$ and $\omega_s^{*2} = \dfrac{\omega_s^2(1+\gamma\varepsilon)}{1-\gamma}$, we obtain:

$$C_s = \varepsilon \left\{ \frac{(1-\gamma\varepsilon)^2 S_A^2\,(\omega_b^*\,,\beta_b^*)}{(1-\gamma\varepsilon)^2\,\varepsilon^2} + \frac{[1-(1-\gamma)\varepsilon]^2 S_A^2\,(\omega_s^*\,,\beta_s^*)}{\dfrac{(1+\gamma\varepsilon)^2}{(1-\gamma)^2}} \right\}^{1/2}$$

$$= \left\{ S_A^2(\omega_b^*, \beta_b^*) + \varepsilon^2 (1-\gamma)^2 (1-2\varepsilon) S_A^2(\omega_s^*, \beta_s^*) \right\}^{1/2} \qquad (3.72)$$

Although the second term is multiplied by ε^2, it could be of the same order as the first term. This will be the case if the spectrum is a constant displacement spectrum. If the spectrum is constant acceleration, the second term is negligible; for constant velocity we get

$$C_s = \omega_b S_V \left(1 + \varepsilon^2 (1-\gamma)^2 \frac{\omega_s^{*2}}{\omega_b^2}\right)^{1/2} = S_A(\omega_b, \beta_b)\,(1 + (1-\gamma)\,\varepsilon)^{1/2}$$

so that again the second term is negligible. These results indicate that for small ε and a typical design spectrum, the isolation system can be designed at least in the initial phase for a relative base displacement, $S_D(\omega_b, \beta_b)$, and the building for a base-shear coefficient of $S_A(\omega_b, \beta_b)$. The reduction in base shear as compared with a fixed-base structure where $C_s = S_A(\omega_s, \beta_s)$ is given by

$$\frac{S_A(\omega_b, \beta_b)}{S_A(\omega_s, \beta_s)}$$

which for a constant velocity spectrum is ω_b/ω_s or roughly of order $\varepsilon^{1/2}$, and this underestimates the reduction since β_b will in general be larger than β_s.

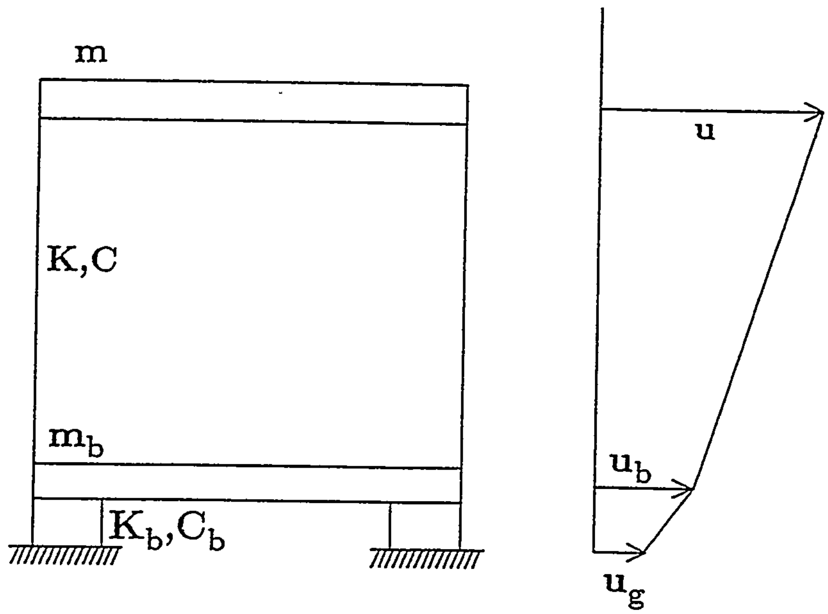


Fig. 3-1: Parameters of Two Degree-of-Freedom Isolated System

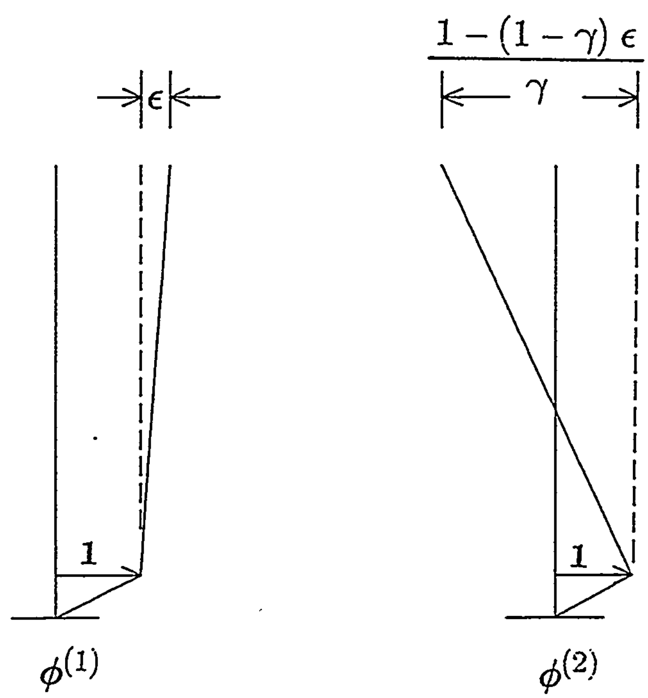


Fig. 3-2: Mode Shapes of Two Degree-of-Freedom Isolated System

BASE AND FLOOR DISPLACEMENT

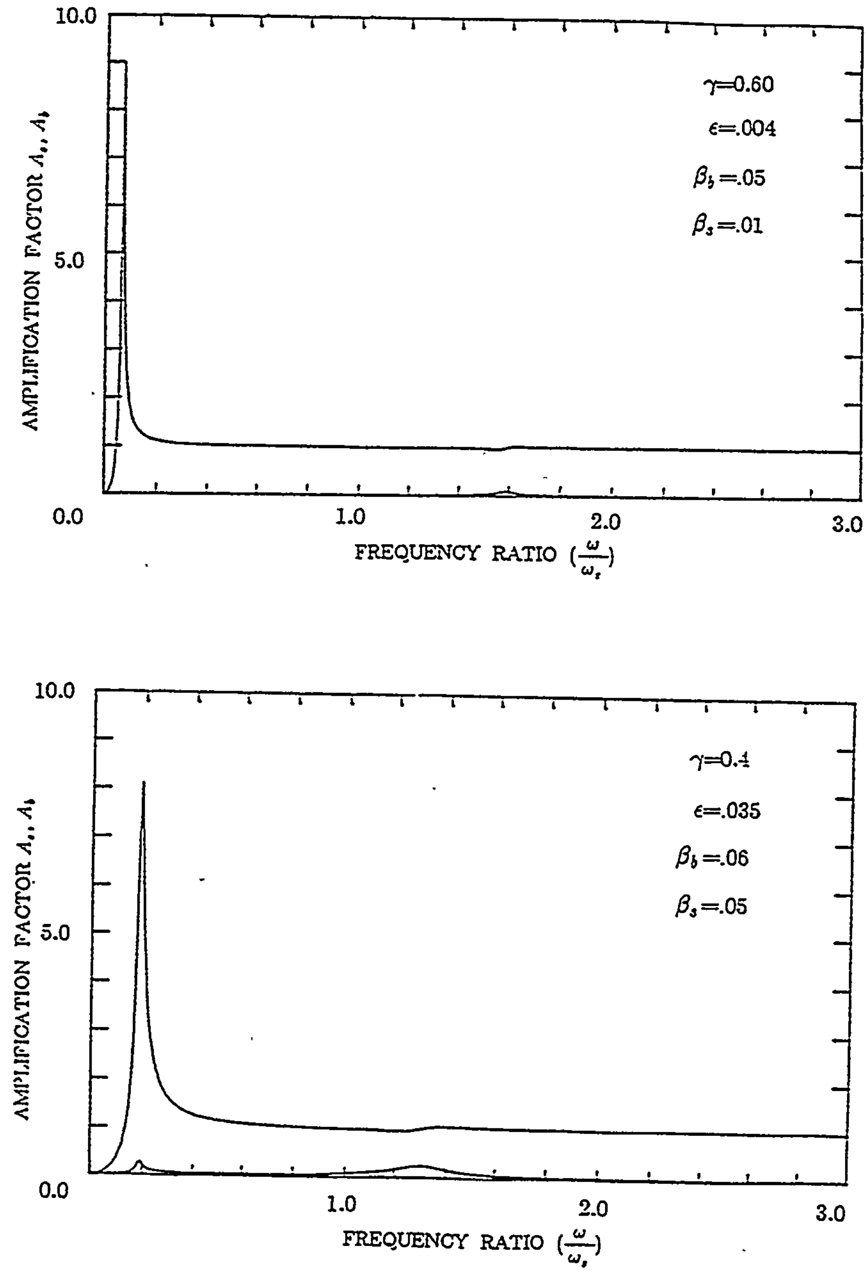


Fig. 3-3: Amplification Factors of Two Degree-of-Freedom Isolation System Subjected to Sinusoidal Excitation

$$m\,\ddot{u} = -c\,(\dot{u} - \dot{u}_b) - k\,(u - u_b) \tag{3.1}$$

$$m\,\ddot{u} + m_b\,\ddot{u}_b = -c_b\,(\dot{u}_b - \dot{u}_g) - k\,(u_b - u_g) \tag{3.2}$$

$$v_s = u - u_b \tag{3.3}$$

$$v_b = u_b - u_g \tag{3.4}$$

$$m\,\ddot{v}_b + m\,\ddot{v}_s + c\,\dot{v}_s + k\,v_s = -m\,\ddot{u}_g \tag{3.5}$$

$$(m + m_b)\ddot{v}_b + m\,\ddot{v}_s + c_b\,\dot{v}_b + k_b\,v_b = -(m + m_b)\,\ddot{u}_g \tag{3.6}$$

(i) $m_b < m$, but are of the same order of magnitude;

(ii) $\omega_s = \left[\dfrac{k}{m}\right]^{1/2} >> \omega_b = \left[\dfrac{k_b}{M}\right]^{1/2}$ and define $\varepsilon = \left[\dfrac{\omega_b}{\omega_s}\right]^2$ and take ε to be of order of magnitude 10^{-2};

(iii) the damping factors for the structure and isolation system, β_s and β_s, respectively, where $\beta_s = \dfrac{c}{2m\,\omega_s}$ and $\beta_b = \dfrac{c_b}{2M\,\omega_b}$, are of the same order of magnitude as ε.

$$\phi^n = \{\phi_b^n, \phi_s^n\}^T \quad ; \quad n = 1,2$$

$$(-\omega_n^2 + \omega_b^2)\,\phi_b^n + (-\gamma\,\omega_n^2)\,\phi_s^n = 0 \tag{3.9a}$$

$$(-\omega_n^2)\,\phi_b^n + (-\omega_n^2 + \omega_s^2)\,\phi_s^n = 0 \tag{3.9b}$$

$$(1-\gamma)\,\omega_n^4 - (\omega_b^2 + \omega_s^2)\,\omega_n^2 + \omega_b^2\,\omega_s^2 = 0 \tag{3.10}$$

$$\begin{matrix}\omega_2^2 \\ \omega_1^2\end{matrix} = \frac{1}{2(1-\gamma)} \left\{ (\omega_s^2 + \omega_b^2) \pm \left[(\omega_s^2 + \omega_b^2)^2 - 4\,(1-\gamma)\,\omega_s^2\,\omega_b^2 \right]^{1/2} \right\} \tag{3.11}$$

$$(\omega_s^2 - \omega_b^2)^2 \left[1 + 4\gamma \frac{\omega_b^2 \omega_s^2}{(\omega_s^2 - \omega_b^2)^2} \right] \tag{3.12}$$

$$\omega_1^2 = \omega_b^{*2} = \omega_b^2 \left[1 - \gamma \frac{\omega_b^2}{\omega_s^2} \right] \tag{3.13}$$

$$\omega_2^2 = \omega_s^{*2} = \frac{\omega_s^2}{1-\gamma} \left[1 + \gamma \frac{\omega_b^2}{\omega_s^2} \right] \tag{3.14}$$

$$\omega_b^* = \omega_b \tag{3.15}$$

$$\omega_s^* = \frac{\omega_s}{(1-\gamma)^{1/2}} \tag{3.16}$$

$$(-\omega_b^{*2} + \omega_b^2)\,\phi_b^1 + (-\gamma\,\omega_b^{*2})\,\phi_s^1 = 0 \tag{3.17a}$$

$$-\omega_b^{*2}\phi_b^1 + (\omega_s^2 - \omega_b^{*2})\,\phi_s^1 = 0 \tag{3.17b}$$

$$\phi^1 = \begin{Bmatrix} 1 \\ \omega_b^2/\omega_s^2 \end{Bmatrix} \tag{3.18}$$

$$\phi^2 = \begin{Bmatrix} 1 \\ -\dfrac{1}{\gamma}\left[1-(1-\gamma)\dfrac{\omega_b^2}{\omega_s^2}\right] \end{Bmatrix} \tag{3.19}$$

$$v_b = q_1\phi_b^1 + q_2\phi_b^2 \tag{3.20}$$

$$v_s = q_1 \phi_s^1 + q_2 \phi_s^2 \tag{3.21}$$

$$\ddot{q}_1 + 2\omega_b^* \beta_b^* \dot{q}_1 + \omega_b^{*2} q_1 = -L_1 \ddot{u}_g \tag{3.22}$$

$$\ddot{q}_2 + 2\omega_s^* \beta_s^* \dot{q}_2 + \omega_s^{*2} q_2 = -L_2 \ddot{u}_g \tag{3.23}$$

$$L_n = \frac{\phi^{n^T} \mathbf{M}^* \mathbf{r}^*}{\phi^{n^T} \mathbf{M}^* \phi^n} \tag{3.24}$$

$$L_1 M_1 = (1, \varepsilon) \begin{bmatrix} M & m \\ m & m \end{bmatrix} \begin{bmatrix} 1 \\ 0 \end{bmatrix} = M + m\,\varepsilon \tag{3.25}$$

$$M_1 = (1, \varepsilon) \begin{bmatrix} M & m \\ m & m \end{bmatrix} \begin{bmatrix} 1 \\ \varepsilon \end{bmatrix} = M + 2m\,\varepsilon + m\,\varepsilon^2$$

$$L_1 = 1 - \gamma\,\varepsilon \tag{3.26}$$

$$L_2 M_2 = M + m\,a \tag{3.27}$$

where:

$$M_2 = M + 2m\,a + a^2 m$$

and:

$$a = -\frac{1}{\gamma}\left[1 - (1-\gamma)\,\varepsilon\right]$$

Since

$$\gamma = \frac{m}{M},\ L_2 M_2 = M\,(1-\gamma)\,\varepsilon$$

and

$$M_2 = M\ \frac{(1-\gamma)\left[1-2(1-\gamma)\,\varepsilon\right]}{\gamma}$$

then

$$L_2 = \gamma\,\varepsilon \tag{3.28}$$

$$2\,\omega_n^*\,\beta_n^* = \frac{\phi^{n^T}\,\mathbf{C}^*\,\phi^n}{\phi^{n^T}\,\mathbf{M}^*\,\phi^n} \tag{3.41}$$

$$2\,\omega_b^*\,\beta_b^* = 2\,\omega_b\,\beta_b\,(1 - 2\gamma\,\varepsilon) \tag{3.42}$$

$$2\,\omega_s^*\,\beta_s^* = \frac{2\,\omega_s\,\beta_s + \gamma\,2\,\omega_b\,\beta_b}{1 - \gamma} \tag{3.43}$$

$$\omega_b^* = \omega_b\,(1 - \gamma\,\varepsilon)^{1/2} \tag{3.44}$$

$$\beta_b^* = \frac{\beta_b\,(1 - 2\gamma\,\varepsilon)}{(1 - \gamma\,\varepsilon)^{1/2}} = \beta_b\,(1 - \frac{3}{2}\,\gamma\,\varepsilon) \tag{3.45}$$

$$\omega_s^* = \omega_1\,\frac{(1 + \gamma\,\varepsilon)^{1/2}}{(1 - \gamma)^{1/2}} \tag{3.46}$$

$$\beta_s^* = \left[\frac{\beta_s}{(1-\gamma)^{1/2}} + \frac{\gamma\beta_b\,\varepsilon^{1/2}}{(1-\gamma)^{1/2}}\right](1 - \frac{1}{2}\gamma\,\varepsilon) \tag{3.47}$$

Table 3-1			
ω	A Fixed-Base Structure	A_s Isolated Structure	A_b Isolated Base
ω_b^*	--	$0(1)$	$\frac{1}{2\beta_0} = 0(\frac{1}{\varepsilon})$
ω_s	$\frac{1}{2\beta_1} = 0(\frac{1}{\varepsilon})$	$0(\varepsilon)$	$0(1)$
ω_s^*	$0(\varepsilon)$	$0(1)$	$0(1)$

$$A_s = \left|\frac{v_s}{u_g}\right| = \left|\phi_s^1 \frac{L_1\omega^2}{(\omega_b^{*2}-\omega^2)+i\,2\omega_b^*\omega\beta_b^*} + \frac{L_2\omega^2}{(\omega_s^{*2}-\omega^2)+i\,2\omega_s^*\omega\beta_s^*}\right| \tag{3.48}$$

$$A_b = \left|\frac{v_b}{u_g}\right| = \left|\phi_b^1 \frac{L_1\omega^2}{(\omega_b^{*2}-\omega^2)+i\,2\omega_b^*\omega\beta_b^*} + \phi_b^2 \frac{L_2\omega^2}{(\omega_s^{*2}-\omega^2)+i\,2\omega_s^*\omega\beta_s^*}\right| \tag{3.49}$$

$$A_s = \left| \varepsilon \frac{(1-\gamma\varepsilon)\,\omega_b^{*2}}{i\,2\,\omega_b^{*2}\beta_b^{*}} - \frac{1}{\gamma}\,\frac{[1-(1-\gamma)\,\varepsilon]\,\gamma\,\varepsilon\,\omega_b^{*2}}{(\omega_s^{*2}-\omega_b^{*2})+i\,2\omega_s^{*}\omega_b^{*}\beta_s^{*}} \right| \tag{3.50}$$

$$A_s = \frac{\varepsilon}{2\,\beta_b^{*}} \tag{3.51}$$

$$A_b = \left| \frac{1-\gamma\,\varepsilon}{2\,\beta_b^{*}} + \frac{\gamma\,\varepsilon^2}{(1-\varepsilon)+i\,\varepsilon} \right| \tag{3.52}$$

$$A_b = \frac{1-\gamma\,\varepsilon}{2\,\beta_b\,(1-\frac{3}{2}\gamma\,\varepsilon)} = \frac{1}{2\,\beta_b}\,(1+\frac{1}{2}\gamma\,\varepsilon) \tag{3.53}$$

$$A_s = (1-(1-\gamma)\,\varepsilon)\frac{\varepsilon}{2\,\beta_s^{*}} \tag{3.54}$$

$$A_b = 1 + \frac{1}{2}\frac{\gamma^2 \varepsilon^2}{(2\beta_s^*)^2} \tag{3.55}$$

$$A_s = \omega^2 \varepsilon \left[\frac{(\omega_s^{*2} - \omega_b^{*2})^2 + \omega^2 (2\omega_s^* \beta_s^* - 2\omega_b^* \beta_b^*)^2}{[(\omega_b^{*2} - \omega^2)^2 + 4\omega_b^{*2}\omega^2\beta_b^{*2}][(\omega_s^{*2} - \omega^2)^2 + 4\omega_s^{*2}\omega^2\beta_s^{*2}]} \right]^{1/2} \tag{3.56}$$

$$A_b = \omega^2 \left[\frac{(\omega_s^{*2} - \omega_b^{*2})^2 + \gamma\varepsilon\omega^2 (2\omega_s^* \beta_s^* - 2\omega_b^* \beta_b^*)^2}{[(\omega_b^{*2} - \omega^2)^2 + 4\omega_b^{*2}\omega^2\beta_b^{*2}][(\omega_s^{*2} - \omega^2)^2 + 4\omega_s^{*2}\omega^2\beta_s^{*2}]} \right]^{1/2} \tag{3.57}$$

$$q_1 = -\frac{L_1}{\omega_b^*} \int_0^t \ddot{u}_g (t - \tau) e^{-\omega_b^* \beta_b^* \tau} \sin \omega_b^* \tau \, d\tau \tag{3.58}$$

$$q_2 = -\frac{L_2}{\omega_s^*} \int_0^t \ddot{u}_g (t - \tau) e^{-\omega_s^* \beta_s^* \tau} \sin \omega_s^* \tau \, d\tau \tag{3.59}$$

$$|q_1|_{max} = L_1 S_D\,(\omega_b^*, \beta_b^*) \tag{3.60}$$

$$|q_2|_{max} = L_2 S_D\,(\omega_s^*, \beta_s^*) \tag{3.61}$$

$$|v_s|_{max} = \left\{(\phi_s^1 q_{1\,max})^2 + (\phi_s^2 q_{2\,max})^2\right\}^{½} \tag{3.62}$$

$$|v_b|_{max} = \left\{(\phi_b^1 q_{1\,max})^2 + (\phi_b^2 q_{2\,max})^2\right\}^{½} \tag{3.63}$$

where now:

$$q_{1\,max} = L_1 \frac{1}{\omega_b^{*2}} S_A\,(\omega_b^*, \beta_b^*)$$

and

$$q_{2\,max} = L_2 \frac{1}{\omega_s^{*2}} S_A\,(\omega_s^*, \beta_s^*)$$

$$|v_s|_{max} = \left\{ \varepsilon^2 (1-\gamma\varepsilon)^2 \frac{S_A^2(\omega_b^*, \beta_b^*)}{\omega_b^{*4}} + \frac{1}{\gamma^2} (1-(1-\gamma)\varepsilon)^2 \gamma^2 \varepsilon^2 \, (1-2\gamma\varepsilon)^2 \frac{S_A^2(\omega_s^*, \beta_s^*)}{\omega_s^{*4}} \right\}^{1/2} \tag{3.64}$$

$$|v_b|_{max} = \left\{ \frac{(1-\gamma\varepsilon)^2 S_A^2(\omega_b^*, \beta_b^*)}{\omega_b^{*4}} + \frac{\gamma^2 \varepsilon^2 (1-2\gamma\varepsilon)^2 S_A^2(\omega_s^*, \beta_s^*)}{\omega_s^{*4}} \right\}^{1/2} \tag{3.65}$$

$$S_A(\omega, \beta) = \omega S_V \tag{3.66}$$

$$|v_s|_{max} = \varepsilon \left\{ (1-\gamma\varepsilon)^2 \frac{\omega_b^{*2}}{\omega_b^{*4}} + (1-(1-\gamma)\varepsilon)^2 \frac{\omega_s^{*2}}{\omega_s^{*4}} \right\}^{1/2} S_V = \varepsilon \frac{S_V}{\omega_b} = \varepsilon S_D(\omega_b, \beta_b) \tag{3.67}$$

$$|v_b|_{\max} = \left\{ \frac{(1-\gamma\varepsilon)^2 \omega_b^{*2}}{\omega_b^{*4}} + \frac{\gamma\varepsilon^2 \omega_s^{*2}}{\omega_s^{*4}} \right\}^{½} S_V = \frac{S_V}{\omega_b} = S_D(\omega_b, \beta_b)$$

(3.68)

$$C_s = \frac{k_s\, v_s}{m} = \omega_s^2 v_s \qquad (3.69)$$

$$C_s = \omega_s^2 S_D(\omega_s, \beta_s) = S_A(\omega_s, \beta_s) \qquad (3.70)$$

$$C_s = \omega_s^2 \left\{ \frac{\varepsilon^2 (1-\gamma\varepsilon)^2 S_A^2(\omega_b^*, \beta_b^*)}{\omega_b^{*4}} + \frac{\varepsilon^2 [1-(1-\gamma)\varepsilon]^2 S_A^2(\omega_s^*, \beta_s^*)}{\omega_s^{*4}} \right\}^{½}$$

(3.71)

Recalling that $\omega_b^{*2} = \omega_b^2(1-\gamma\varepsilon)$ and $\omega_s^{*2} = \dfrac{\omega_s^2(1+\gamma\varepsilon)}{1-\gamma}$, we obtain:

$$C_s = \varepsilon\left\{\frac{(1-\gamma\varepsilon)^2 S_A^2(\omega_b^*, \beta_b^*)}{(1-\gamma\varepsilon)^2\varepsilon^2} + \frac{[1-(1-\gamma)\varepsilon]^2 S_A^2(\omega_s^*, \beta_s^*)}{\frac{(1+\gamma\varepsilon)^2}{(1-\gamma)^2}}\right\}^{1/2}$$

$$= \left\{S_A^2(\omega_b^*, \beta_b^*) + \varepsilon^2(1-\gamma)^2(1-2\varepsilon)S_A^2(\omega_s^*, \beta_s^*)\right\}^{1/2} \quad (3.72)$$

$$C_s = \omega_b S_V (1 + \varepsilon^2 (1-\gamma)^2 \frac{\omega_s^{*2}}{\omega_b^2})^{1/2}$$

$$= S_A(\omega_b, \beta_b)(1 + (1-\gamma)\varepsilon)^{1/2}$$

$$\frac{S_A(\omega_b, \beta_b)}{S_A(\omega_s, \beta_s)}$$

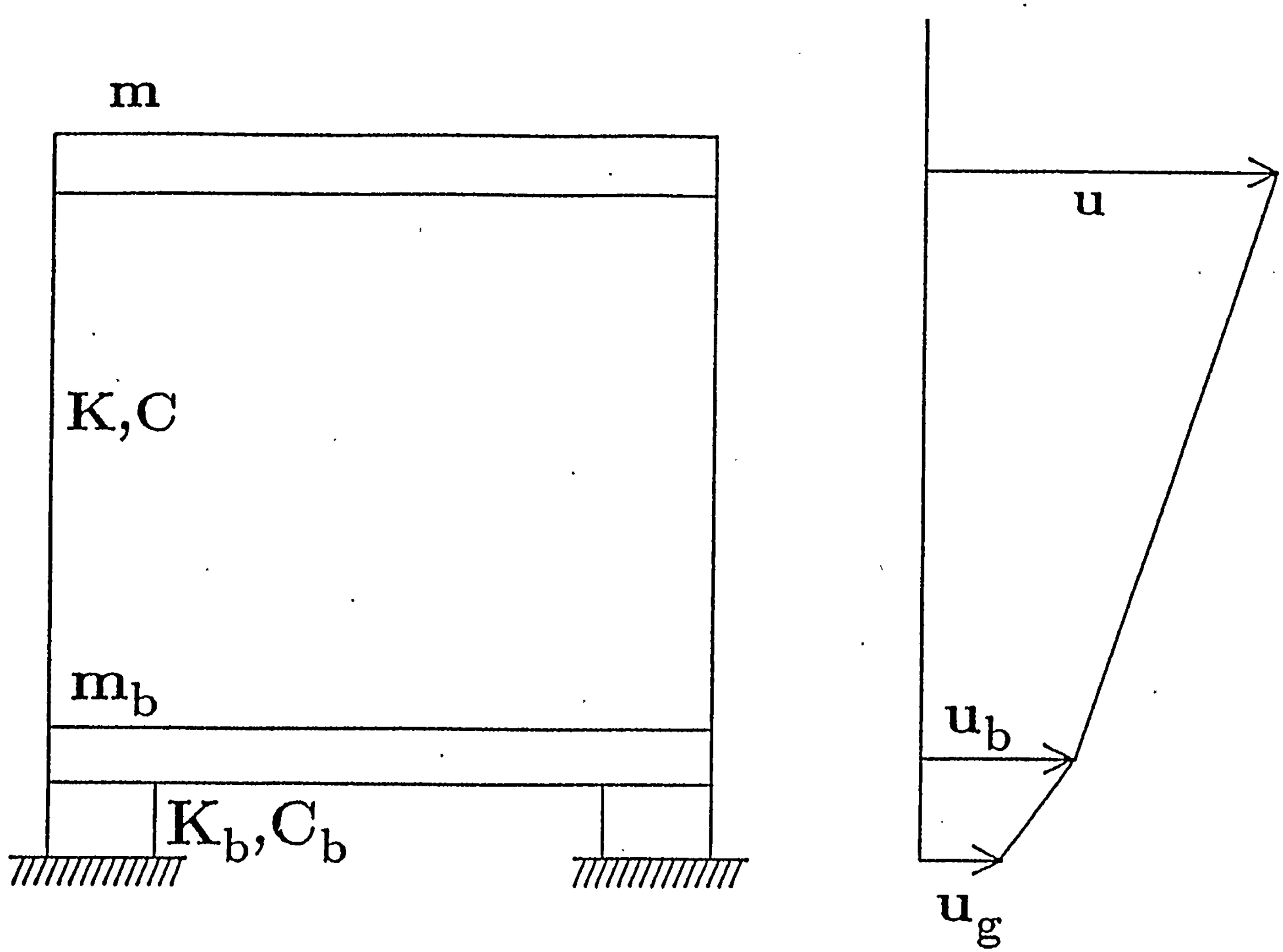


Fig. 3-1: Parameters of Two Degree-of-Freedom Isolated System

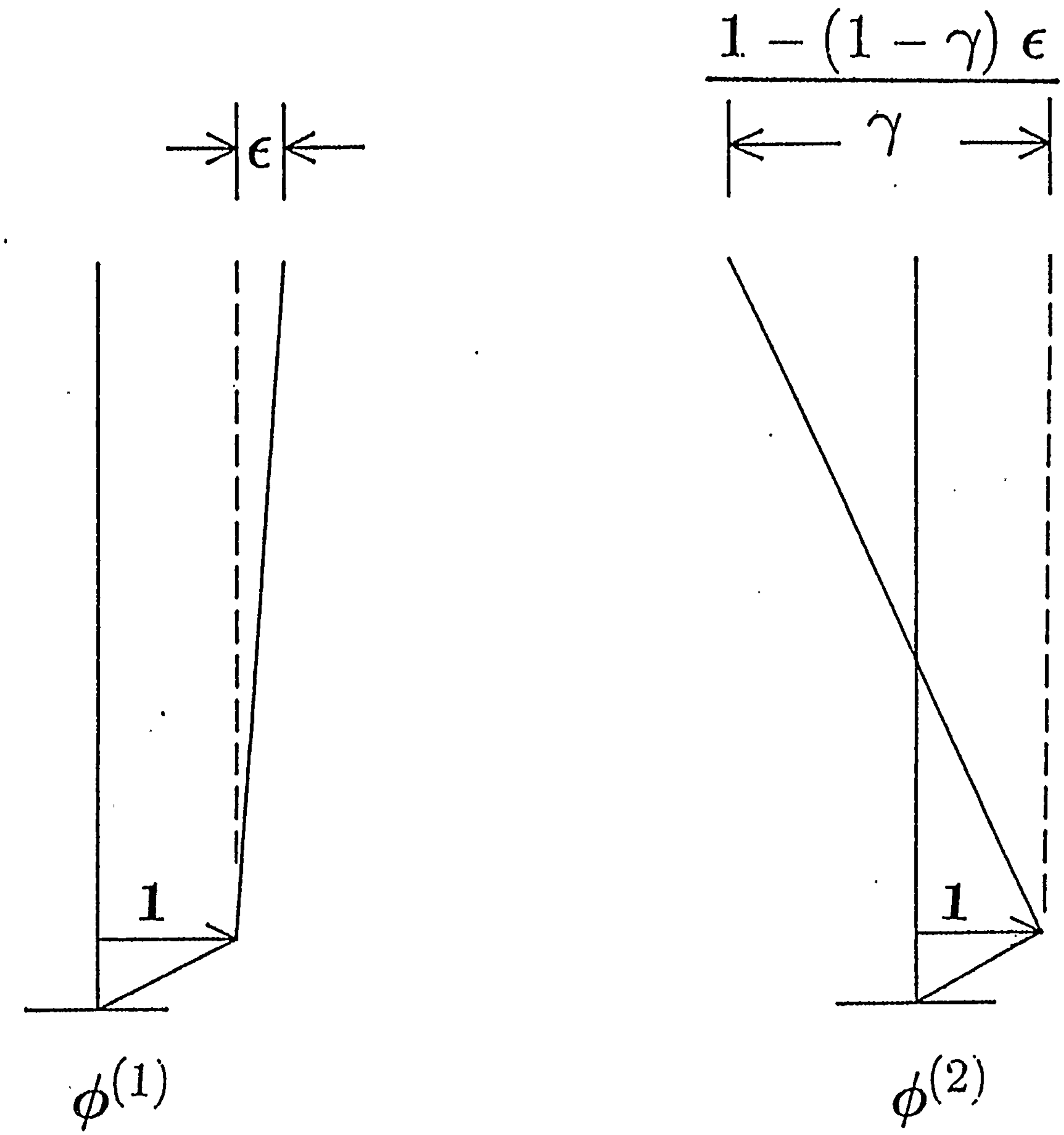


Fig. 3-2: Mode Shapes of Two Degree-of-Freedom Isolated System

LECTURE 4

Extension of Theory to Buildings

by

James M. Kelly
Professor of Civil Engineering
Earthquake Engineering Research Center
University of California
Berkeley, California 94720

Short Course on Seismic Base Isolation
Department of Energy

August 10 - August 14, 1992

LECTURE 4

Extension of Theory to Buildings

Design Technique

The foregoing two-degree-of-freedom analysis of the simple linear model can be applied to the case of a building with several stories. Let us represent the structural system of this building by a mass matrix, **M**, a damping matrix, **C**, and stiffness matrix, **K**. If the structure were conventionally based, the relative displacement **u** of each degree of freedom with respect to the ground would be given by

$$\mathbf{M}\ddot{\mathbf{u}} + \mathbf{C}\dot{\mathbf{u}} + \mathbf{K}\mathbf{u} = -\mathbf{M}\mathbf{r}\ddot{u}_g. \tag{4.1}$$

where **r** is a vector that couples each degree of freedom to the ground motion. When this structural model is superposed on a base isolation system with base mass, m_b, stiffness, k_b, and damping, c_b, this equation is replaced by

$$\mathbf{M}\ddot{\mathbf{v}} + \mathbf{C}\dot{\mathbf{v}} + \mathbf{K}\mathbf{v} = -\mathbf{M}\mathbf{r}(\ddot{u}_g + \ddot{v}_b) \tag{4.2}$$

where **v** is the displacement relative to the base slab and v_b is the relative displacement of the base slab to the ground. The overall equation of motion for the combined building and base slab is

$$\mathbf{r}^T\mathbf{M}(\ddot{\mathbf{v}} + \mathbf{r}\ddot{v}_b + \mathbf{r}\ddot{u}_g) + m_b(\ddot{v}_b + \ddot{u}_g) + c_b\dot{v}_b + k_b v_b = 0 \tag{4.3}$$

which can be written in the form

$$\mathbf{r}^T\mathbf{M}\ddot{\mathbf{v}} + (m + m_b)\ddot{v}_b + c_b\dot{v}_b + k_b v_b = -(m + m_b)\ddot{u}_g \tag{4.4}$$

In this equation we have identified $\mathbf{r}^T\mathbf{M}\mathbf{r}$ as the total mass of the building so that $m + m_b$ is the total mass carried on the isolation system. The matrix form of these equations is

$$\mathbf{M}^*\ddot{\mathbf{v}}^* + \mathbf{C}^*\dot{\mathbf{v}}^* + \mathbf{K}^*\mathbf{v}^* = -\mathbf{M}^*\mathbf{r}^*\ddot{u}_g \tag{4.5}$$

where

$$\mathbf{M}^* = \begin{bmatrix} m + m_b & \mathbf{r}^T \mathbf{M} \\ \mathbf{M}\mathbf{r} & \mathbf{M} \end{bmatrix} \quad \mathbf{C}^* = \begin{bmatrix} c_b & 0 \\ 0 & \mathbf{C} \end{bmatrix}$$

and

$$\mathbf{K}^* = \begin{bmatrix} k_b & 0 \\ 0 & \mathbf{K} \end{bmatrix} \quad \mathbf{r}^* = \begin{bmatrix} 1 \\ 0 \end{bmatrix}$$

with

$$\mathbf{v}^* = \begin{bmatrix} v_b \\ \mathbf{v} \end{bmatrix}$$

The natural modes of the fixed-base structure are assumed known and denoted by ϕ^i where $i = 1$ to N. In terms of these, the displacement of each degree of freedom of the structure can be represented as

$$\mathbf{v} = \sum_{i=1}^{N} q_i \, \phi^i \tag{4.6}$$

The natural frequencies ω_n^2 are given by

$$+\mathbf{M}\,\phi^n\,\omega_n^2 = \mathbf{K}\,\phi^n \tag{4.7}$$

and we assume that $\phi^n\,\mathbf{C}\,\phi^m = 0$ if $n \neq m$.

The matrix equations of motion reduce to the $N+1$ equations

$$\sum_{i=1}^{N} \mathbf{r}^T \mathbf{M}\,\phi^i\,\ddot{q}_i + (m + m_b)\,\ddot{v}_b + c_b\,\dot{v}_b + k_b\,v_b = -(m + m_b)\,\ddot{u}_g \tag{4.8}$$

and

$$\ddot{q}_i + 2\,\omega_i\,\beta_i\,\dot{q}_i + \omega_i^2\,q_i = -L_i\,(\ddot{v}_b + \ddot{u}_g) \qquad i = 1 \text{ to } N \tag{4.9}$$

where L_i are the participation factors of the fixed-base modes, i.e.,

$$L_i = \frac{\phi^{i^T}\mathbf{M}\,\mathbf{r}}{\phi^{i^T}\mathbf{M}\,\phi^i}$$

The fixed-base modal masses are

$$M_i = \phi^{i^T}\mathbf{M}\,\phi^i \tag{4.10}$$

and we can write these equations in the form

$$\sum_{i=1}^{N} \frac{L_i M_i}{(m + m_b)} \ddot{q}_i + \ddot{v}_b + 2\omega_b \beta_b \dot{v}_b + \omega_b^2 v_b = -\ddot{u}_g \tag{4.11}$$

and

$$L_i \ddot{v}_b + \ddot{q}_i + 2\omega_i \beta_i \dot{q}_i + \omega_i^2 q_i = -L_i \ddot{u}_g \quad i = 1 \text{ to } N \tag{4.12}$$

The natural modes and frequencies of the composite system (in terms of the fixed-base modes, ϕ^i and v_b) are given by the equations

$$\ddot{q}_i + \omega_i^2 q_i + L_i \ddot{v}_b = 0 \quad i = 1 \text{ to } N \tag{4.13}$$

and

$$\sum_{i=1}^{N} \frac{L_i M_i}{(m + m_b)} \ddot{q}_i + \ddot{v}_b + \omega_b^2 v_b = 0 \tag{4.14}$$

If we denote the new eigenvector in v_b and q_i by ψ^j, where

$$\psi^{j^T} = \left\{ \bar{v}_b, \bar{q}_1, \bar{q}_2, \ldots, \bar{q}_N \right\} \tag{4.15}$$

with $j = 0$ to N, (i.e., ψ_o^j is the value of $\bar{v}_b$ of the j^{th} mode) and take this in every case equal to unity, we have

$$(-\omega^2 + \omega_i^2)\bar{q}_i - \omega^2 L_i \bar{v}_b = 0 \quad i = 1 \text{ to } N \tag{4.16}$$

$$-\omega^2 \sum_{i=1}^{N} \frac{L_i M_i}{m + m_b} \bar{q}_i + (-\omega^2 + \omega_b^2)\bar{v}_b = 0 \tag{4.17}$$

Substitution of $\bar{q}_i$ in terms of $\bar{v}_b$ from each of the first N equations into the remaining equation gives

$$-\omega^2 \sum_{i=1}^{N} \frac{L_i^2 M_i}{m + m_b} \frac{\omega^2}{(-\omega^2 + \omega_i^2)} \bar{v}_b + (-\omega^2 + \omega_b^2)\bar{v}_b = 0 \tag{4.18}$$

This leads to a single characteristic equation for the frequencies of the composite system, namely

$$\sum_{i=1}^{N} \bar{\gamma}_i \frac{1}{1 - \dfrac{\omega_i^2}{\omega^2}} = (1 - \frac{\omega_b^2}{\omega^2}) \tag{4.19}$$

where

$$\bar{\gamma}_i = \frac{L_i^2 M_i}{m + m_b}$$

Each of the terms on the left hand side become singular at $\omega^2 = \omega_i^2$ and goes from $-\infty$ to $+\infty$ as ω^2 goes from $\omega_i^2 - 0$ to $\omega_i^2 + 0$. The equation can be solved by an iterative process for each root. For example, the lowest root when ω^2 is close to ω_b^2 (i.e., ω_b^{*2}) is given approximately by

$$1 - \frac{\omega_b^2}{\omega^2} = \sum_{i=1}^{N} \bar{\gamma}_i \frac{1}{1 - \dfrac{\omega_i^2}{\omega_b^2}} \tag{4.20}$$

or when we recognize that $\omega_i^2/\omega_b^2 >> 1$

(4.21)

The next root near ω_1 we first approximately determine from

$$1 - \frac{\omega_b^2}{\omega^2} = \bar{\gamma}_1 \left[\frac{1}{1 - \dfrac{\omega_1^2}{\omega^2}} \right] \tag{4.22}$$

and set $\omega_b^2/\omega^2 \approx 0$, getting

$$\omega^2 = \omega_1^2 \left(\frac{1}{1 - \bar{\gamma}_1} \right) \tag{4.23}$$

A more refined estimate is then obtained from

$$\bar{\gamma}_1 \left[\frac{1}{1 - \dfrac{\omega_1^2}{\omega^2}} \right] = 1 - (1 - \bar{\gamma}_1) \frac{\omega_b^2}{\omega_1^2} - \sum_{i=2}^{N} \bar{\gamma}_i \left[\frac{1}{1 - \dfrac{\omega_i^2}{\omega_1^2}(1 - \bar{\gamma}_1)} \right] \tag{4.24}$$

leading to

$$\omega_1^{*2} = \frac{\omega_1^2}{1 - \bar{\gamma}_1 \left[\dfrac{1}{1 - (1 - \bar{\gamma}_1) \dfrac{\omega_b^2}{\omega_1^2} + \displaystyle\sum_{i=2}^{N} \dfrac{\bar{\gamma}_i}{1 - \dfrac{\omega_i^2}{\omega_1^2}(1 - \bar{\gamma}_1)}} \right]} \tag{4.25}$$

The general result for the k^{th} mode is given by

$$\bar{\gamma}_k \left[\frac{1}{1 - \dfrac{\omega_k^2}{\omega^2}} \right] = 1 - (1 - \bar{\gamma}_k) \frac{\omega_b^2}{\omega_k^2} - \sum_{\substack{i=1 \\ i \neq k}}^{N} \bar{\gamma}_i \left[\frac{1}{1 - \dfrac{\omega_i^2}{\omega_k^2} (1 - \bar{\gamma}_k)} \right] \tag{4.26}$$

or

$$\omega_k^{*2} = \frac{\omega_k^2}{1 - \bar{\gamma}_k \left[\dfrac{1}{1 - (1 - \bar{\gamma}_k) \dfrac{\omega_b^2}{\omega_k^2} - \displaystyle\sum_{\substack{i=1 \\ i \neq k}}^{N} \dfrac{\bar{\gamma}_i}{1 - \dfrac{\omega_i^2}{\omega_1^2} (1 - \bar{\gamma}_k)}} \right]} \tag{4.27}$$

The corresponding eigenvectors in $\bar{q}_i^j$ to the ω_i^* are computed by substitution of these values for ω_j^* back into the expression

$$\bar{q}_i^j = - \frac{\omega_j^{*2}}{\omega_j^{*2} - \omega_i^2} L_i \tag{4.28}$$

where the value of $\bar{v}_b$ for each mode, ω, is taken as unity, and the resulting mode shapes for the combined structure are given by

$$\psi^{j^T} = \left\{ 1, \sum_{i=1}^{N} \bar{q}_i^j \phi_1^i , \sum_{i=1}^{N} \bar{q}_i^j \phi_2^i , \sum_{i=1}^{N} \bar{q}_i^j \phi_3^i , \cdots \right\} \tag{4.29}$$

As an example of the use of this technique, let us consider a five-story shear building modelled as a five degree-of-freedom system. The fixed-base model is shown in Fig. 4-1. We assume that all floor masses are the same, that the first fixed-base period is 0.40 seconds, and the first mode is a straight line. The corresponding isolated model has a base mass equal to each of the other five and the isolated period is 2.0 seconds.

The interstory stiffness, as denoted by k_{12}, k_{23} in Fig. 4-1, can be computed directly from the equations of undamped motion if they are written in the form

$$m_1 \ddot{v}_1 = -k_{12}(v_1 - v_2)$$

$$m_1 \ddot{v}_1 + m_2 \ddot{v}_2 = -k_{23}(v_2 - v_3)$$

$$\sum_{i=1}^{j} m_i \ddot{v}_i = -k_{j,j+1}(v_{j+1} - v_j)$$

where j runs from 1 to 5 in this case.

Setting all $m_i = m_0$, $\mathrm{v} = e^{i\omega_1 t}\phi^1$ with $\phi^{1^T} = (1,2,3,4,5)$, we find

$$k_{12} = 5\,\omega_1^2 m_0$$

$$k_{23} = 9\,\omega_1^2 m_0$$

$$k_{34} = 12\,\omega_1^2 m_0$$

$$k_{45} = 14\,\omega_1^2 m_0$$

$$k_{56} = 15\,\omega_1^2 m_0$$

and the stiffness matrix, **K**, becomes

$$\mathbf{K} = \omega_1^2 m_0 \begin{bmatrix} 5 & 5 & 0 & 0 & 0 \\ -5 & 14 & -9 & 0 & 0 \\ 0 & -9 & 21 & -12 & 0 \\ 0 & 0 & -12 & 26 & -14 \\ 0 & 0 & 0 & -14 & 29 \end{bmatrix}$$

The complete set of mode shapes normalized in such a way that the bottom displacement is unity is given as

$$\Phi = \left[\phi^1, \phi^2, \phi^3, \phi^4, \phi^5\right] = \begin{bmatrix} 5 & -2.143 & 0.75 & -0.179 & 0.024 \\ 4 & 0.429 & -1.50 & 0.821 & -0.091 \\ 3 & 1.571 & -0.25 & -1.179 & 0.643 \\ 2 & 1.643 & 1.00 & 0.071 & -1.143 \\ 1 & 1.000 & 1.00 & 1.000 & 1.000 \end{bmatrix}$$

with the frequencies given by

$$\omega_2^2 = 6\,\omega_1^2, \quad \omega_3^2 = 15\,\omega_1^2, \quad \omega_4^2 = 28\,\omega_1^2, \quad \omega_5^2 = 45\,\omega_1^2$$

The participation factors defined by

$$L_i = \frac{\phi^{i^T} \mathbf{M} \mathbf{r}}{\phi^{i^T} \mathbf{M} \phi^i}$$

are $L_i = \{ 0.273, 0.229, 0.205, 0.172, 0.121 \}$. When this model is isolated on a system with an isolation period of 2 seconds (based on a rigid structure), and with a base mass, m_0, the important parameters are the modified mass ratios, $\bar{\gamma}_i$, given by

$$\bar{\gamma}_i = \frac{L_i^2 m_i}{m + m_b} = \frac{(\phi^{i^T} \mathbf{M} \mathbf{r})^2}{(\phi^{i^T} \mathbf{M} \phi^{i^T})(m + m_b)}$$

which in this case become

$$\bar{\gamma}_i = \frac{(\phi^{i^T} \mathbf{r})^2}{6(\phi^{i^T} \phi^i)}$$

We find that $\bar{\gamma}_i = \{ 0.682, 0.095, 0.034, 0.015, 0.007 \}$.

The first isolated frequency of the model is given by

$$\omega_b^{*2} = \omega_b^2 \left\{ 1 - \sum_{i=1}^{5} \gamma_i \frac{\omega_b^2}{\omega_i^2} \right\}$$

and with $\omega_b^2 = \left(\frac{2\pi}{2} \right)^2 = 9.8696 \text{ sec}^{-2}$, becomes $\omega_b^{*2} = 0.5930 \text{ sec}^{-2}$.

The others, using the approximate formulae (Eqs. 4.25 and 4.27) are found to be

$$\omega_1^{*2} = 3.428\, \omega_1^2$$

$$\omega_2^{*2} = 1.050\, \omega_2^2$$

$$\omega_3^{*2} = 1.018\, \omega_3^2$$

$$\omega_4^{*2} = 1.00788\, \omega_4^2$$

$$\omega_5^{*2} = 1.0034\, \omega_5^2$$

It is clear that the presence of the isolation system has a large effect on the frequencies of the first two modes, but almost no effect on those of the higher modes. What is happening is that the base mass at these high frequencies is acting as a fixed base.

It is useful for the purpose of this demonstration to normalize the mode shapes in the same way as that used for the fixed-base modes where the first-story displacement was set equal to unity. Here, we set the displacement at the same level equal to unity, recalling that now this is the second level above the ground and we compute this displacement relative to the ground rather than as before relative to the base mass.

The frequencies can, of course, be computed directly for the composite six degree-of-freedom system using any standard eigenvalue package. The results for this example when m_0 is taken as 4 448 kN (1 000 kips) and ω_1 as $2\pi/0.4$ rads/sec are shown in the Table 4-1.

Table 4-1

Frequencies

(m_o) = 1 000 kips, $\omega_1 = \dfrac{2\pi}{0.4}$ rads/sec

	Fixed Base	Isolated (Approx.)	Isolated (Exact)
0	-	3.0973	3.0980
1	15.71	25.42	25.977
2	38.48	50.96	47.031
3	60.84	70.48	67.913
4	83.12	89.67	88.585
5	105.37	109.14	108.83

The approximate method gives results that are very close to the direct computation. It is not easy to say which is the more exact result since the direct computation involves a stiffness matrix with one diagonal element that is two orders of magnitude smaller than the others.

Returning to the full equations of motion for the combined system and comparing them with the previous set for the isolated one-degree-of-freedom structure, the corresponding equations for the model on the base slab are

$$\gamma \ddot{v}_s + \ddot{v}_b + 2\omega_b \beta_b \dot{v}_b + \omega_b^2 v_b = -\ddot{u}_g \tag{4.30}$$

and

$$\ddot{v}_s + \ddot{v}_b + 2\omega_s \beta_s \dot{v}_s + \omega_s^2 v_s = -\ddot{u}_g \tag{4.31}$$

If we retain only one mode of the fixed-base structure, we can make the equations correspond by replacing v_b in the elementary analysis by $L_1 v_b$, $\ddot{u}_g$ by $L_1 \ddot{u}_g$ and

$$\gamma = \frac{m}{m + m_b} = \frac{m}{M}$$

by

$$\gamma = \frac{L_1^2 M_1}{m + m_b} = \bar{\gamma}_1$$

When this is done, q_1 will be given by the solution for v_s.

The basic results for the single-degree-of-freedom structure, namely that

$$|v_b|_{\max} = \frac{1}{\omega_b^2} S_A(\omega_b, \beta_b) \tag{4.32}$$

and

$$C = \left\{ S_A^2(\omega_b^*, \beta_b^*) + \varepsilon^2 (1-\gamma)^2 S_A^2(\omega_s^*, \beta_s^*) \right\}^{1/2} \tag{4.33}$$

are replaced as follows. The maximum relative base displacement is given by

$$|L_1 v_b|_{\max} = \frac{1}{\omega_b^2} L_1 S_A(\omega_b, \beta_b) \tag{4.34}$$

and, since L_1 appears on both sides, we have the same result as before.

To obtain the base shear we have

$$|q_1|_{\max} = \left\{ \frac{\varepsilon^2 L_1^2 S_A^2(\omega_b^*, \beta_b^*)}{\omega_b^{*4}} + \frac{\varepsilon^2 L_1^2 S_A^2(\omega_s^*, \beta_s^*)}{\omega_s^{*4}} \right\}^{1/2} \tag{4.35}$$

with ω_s^*, β_s^* are calculated as before.

The relative displacement vector, v, is given by

$$\mathbf{v} = q_1 \phi^1 \tag{4.36}$$

and the inertial force on each element, neglecting damping contributions, is

$$\mathbf{F} = \mathbf{K}\mathbf{v} = q_1 \mathbf{K}\phi^1 = q_1 \mathbf{M}\phi^1 \omega_1^2 \tag{4.37}$$

The total horizontal force on the superstructure is

$$\mathbf{r}^T \mathbf{F} = q_1 \omega_1^2 L_1 M_1 \tag{4.38}$$

and this is expressed in terms of the base shear coefficient, C_s, through

$$C_s\, m = \mathbf{r}\mathbf{F} \tag{4.39}$$

Thus

$$C_s = \frac{L_1 M_1}{m} \left\{ L_1^2 S_A^2(\omega_b, \beta_b) + (1-\bar{\gamma}_1)^2 \varepsilon^2 L_1^2 S_A^2(\omega_s^*, \beta_s^*) \right\}^{1/2}$$

$$= \frac{L_1^2 M_1}{m} \left\{ S_A^2(\omega_b, \beta_b) + (1-\bar{\gamma}_1)^2 \varepsilon^2 S_A^2(\omega_s^*, \beta_s^*) \right\}^{1/2} \tag{4.40}$$

with $\varepsilon = \omega_b^2/\omega_1^2$ as previously.

In the standard equivalent lateral force procedure, the building considered as a fixed-base structure is designed to resist the lateral seismic base shear, V, given by $V = C_s W$ where W is the total weight. The value of C_s is derived from a code formula.

In the fixed-base building, the lateral seismic shear force, F_x, at any level denoted by x is given by

$$F_x = C_{V_x} V \tag{4.41}$$

where

$$C_{V_x} = \frac{W_x h_x^j}{\sum_i W_i h_i^j}$$

In this formula the terms W_i, W_x are the weights of levels i and x, h_i, h_x the heights of levels i and x, and j an exponent which is taken to be 1 if the period is 0.5 seconds or less and 2 if it is 2.50 seconds or more. This reflects the fact that a stiff, low building will respond predominantly in shear with a roughly linear first mode and a tall, flexible building in bending with a quadratic first mode. The exponent j may be selected by linear interpolation between 1 and 2.

The seismic shear force at any level x is calculated from

$$V_x = \sum_{i=x}^{N} F_i \tag{4.42}$$

and the structure designed using these values of shear. If the code forces are elastic forces, then they are reduced by factors denoted by R_W that are referred to as ductility factors and these are specified by the code for different types of structural systems. They represent the intuitive feeling that many structural forms have substantial reserves of strength beyond their elastic capacity. In some codes, ductility factors for moment-resisting steel frames may be as high as 12.

For an isolated building the distribution of shear force should be given by the shape of the first isolated mode, namely $\{1, \varepsilon L_1 \phi^1\}^T$, or, if we neglect the ε terms, the distribution should be uniform. Then

$$F_x = C_{V_x} V \tag{4.43}$$

and

$$C_{V_x} = \frac{W_x}{W} \tag{4.44}$$

and

$$V_x = \sum_{i=x}^{N} C_{V_i} V = V \sum_{i=x}^{N} \frac{W_i}{W} = C_s \sum_{i=x}^{N} W_i \tag{4.45}$$

Thus, to design a base-isolated structure, we estimate C_s from the design spectrum and design the superstructure at each level for a shear force equivalent C_s times the weight above. If appropriate, the ductility factors, R_W, can then be taken into account, but the reductions in seismic load achieved by isolation will be so substantial that it may not be necessary to include them. If, for example, in a large earthquake the ductile capacity of a moment-resisting steel frame is called upon, there will be substantial interstory drifts and consequently considerable damage to nonstructural elements, windows, and ceilings. Moreover, development of large ductility will produce a lengthening of the fixed-base period of the superstructure reducing the benefit of the isolation. Large R_W factors also translate into expensive structural forms and it is probably more cost effective to use a structural system with a low R_W factor, e.g. nonductile concrete, with an isolation system and design for elastic response in the superstructure.

$$\mathbf{M}\ddot{\mathbf{u}} + \mathbf{C}\dot{\mathbf{u}} + \mathbf{K}\mathbf{u} = -\mathbf{M}\mathbf{r}\ddot{u}_g \tag{4.1}$$

$$\mathbf{M}\ddot{\mathbf{v}} + \mathbf{C}\dot{\mathbf{v}} + \mathbf{K}\mathbf{v} = -\mathbf{M}\mathbf{r}(\ddot{u}_g + \ddot{v}_b) \tag{4.2}$$

$$\mathbf{r}^T\mathbf{M}(\ddot{\mathbf{v}} + \mathbf{r}\ddot{v}_b + \mathbf{r}\ddot{u}_g) + m_b(\ddot{v}_b + \ddot{u}_g) + c_b\dot{v}_b + k_b v_b = 0 \tag{4.3}$$

$$\mathbf{r}^T\mathbf{M}\ddot{\mathbf{v}} + (m + m_b)\ddot{v}_b + c_b\dot{v}_b + k_b v_b = -(m + m_b)\ddot{u}_g \tag{4.4}$$

$$\mathbf{M}^*\ddot{\mathbf{v}}^* + \mathbf{C}^*\dot{\mathbf{v}}^* + \mathbf{K}^*\mathbf{v}^* = -\mathbf{M}^*\mathbf{r}^*\ddot{u}_g \tag{4.5}$$

$$\mathbf{M}^*\ddot{\mathbf{v}}^* + \mathbf{C}^*\dot{\mathbf{v}}^* + \mathbf{K}^*\mathbf{v}^* = -\mathbf{M}^*\mathbf{r}^*\ddot{u}_g \quad (4.5)$$

where

$$\mathbf{M}^* = \begin{bmatrix} m + m_b & \mathbf{r}^T \mathbf{M} \\ \mathbf{M}\,\mathbf{r} & \mathbf{M} \end{bmatrix} \quad \mathbf{C}^* = \begin{bmatrix} c_b & 0 \\ 0 & \mathbf{C} \end{bmatrix}$$

and

$$\mathbf{K}^* = \begin{bmatrix} k_b & 0 \\ 0 & \mathbf{K} \end{bmatrix} \quad \mathbf{r}^* = \begin{bmatrix} 1 \\ 0 \end{bmatrix}$$

with

$$\mathbf{v}^* = \begin{bmatrix} v_b \\ \mathbf{v} \end{bmatrix}$$

$$\mathbf{v} = \sum_{i=1}^{N} q_i \, \phi^i \tag{4.6}$$

$$+\mathbf{M}\,\phi^n \,\omega_n^2 = \mathbf{K}\,\phi^n \tag{4.7}$$

$$\sum_{i=1}^{N} \mathbf{r}^T \mathbf{M} \phi^i \ddot{q}_i + (m + m_b) \ddot{v}_b + c_b \dot{v}_b + k_b v_b = -(m + m_b) \ddot{u}_g \tag{4.8}$$

$$\ddot{q}_i + 2 \omega_i \beta_i \dot{q}_i + \omega_i^2 q_i = -L_i (\ddot{v}_b + \ddot{u}_g) \qquad i = 1 \text{ to } N \tag{4.9}$$

$$L_i = \frac{\phi^{i^T} \mathbf{M} \mathbf{r}}{\phi^{i^T} \mathbf{M} \phi^i}$$

$$M_i = \phi^{i^T} \mathbf{M} \phi^i \tag{4.10}$$

$$\sum_{i=1}^{N} \frac{L_i M_i}{(m + m_b)} \ddot{q}_i + \ddot{v}_b + 2 \omega_b \beta_b \dot{v}_b + \omega_b^2 v_b = -\ddot{u}_g \tag{4.11}$$

$$L_i \ddot{v}_b + \ddot{q}_i + 2\omega_i \beta_i \dot{q}_i + \omega_i^2 q_i = -L_i \ddot{u}_g \quad i = 1 \text{ to } N \tag{4.12}$$

$$\ddot{q}_i + \omega_i^2 q_i + L_i \ddot{v}_b = 0 \quad i = 1 \text{ to } N \tag{4.13}$$

$$\sum_{i=1}^{N} \frac{L_i M_i}{(m + m_b)} \ddot{q}_i + \ddot{v}_b + \omega_b^2 v_b = 0 \tag{4.14}$$

$$\psi^{j^T} = \left\{ \overline{v}_b, \overline{q}_1, \overline{q}_2, \ldots, \overline{q}_N \right\} \tag{4.15}$$

$$(-\omega^2 + \omega_i^2)\overline{q}_i - \omega^2 L_i \overline{v}_b = 0 \quad i = 1 \text{ to } N \tag{4.16}$$

$$-\omega^2 \sum_{i=1}^{N} \frac{L_i M_i}{m + m_b} \bar{q}_i + (-\omega^2 + \omega_b^2)\bar{v}_b = 0 \qquad (4.17)$$

$$-\omega^2 \sum_{i=1}^{N} \frac{L_i^2 M_i}{m + m_b} \frac{\omega^2}{(-\omega^2 + \omega_i^2)} \bar{v}_b + (-\omega^2 + \omega_b^2)\bar{v}_b = 0 \quad (4.18)$$

$$\sum_{i=1}^{N} \bar{\gamma}_i \frac{1}{1 - \dfrac{\omega_i^2}{\omega^2}} = (1 - \frac{\omega_b^2}{\omega^2}) \qquad (4.19)$$

where

$$\bar{\gamma}_i = \frac{L_i^2 M_i}{m + m_b}$$

$$1 - \frac{\omega_b^2}{\omega^2} = \sum_{i=1}^{N} \bar{\gamma}_i \frac{1}{1 - \dfrac{\omega_i^2}{\omega_b^2}} \qquad (4.20)$$

$$\omega_b^{*2} = \omega_b^2 \left[1 - \sum_{i=1}^{N} \bar{\gamma}_i \frac{\omega_b^2}{\omega_i^2} \right] \tag{4.21}$$

$$1 - \frac{\omega_b^2}{\omega^2} = \bar{\gamma}_1 \left[\frac{1}{1 - \dfrac{\omega_1^2}{\omega^2}} \right] \tag{4.22}$$

and set $\omega_b^2/\omega^2 \approx 0$, obtaining

$$\omega^2 = \omega_1^2 \left(\frac{1}{1 - \bar{\gamma}_1} \right) \tag{4.23}$$

$$\bar{\gamma}_1 \left[\frac{1}{1 - \dfrac{\omega_1^2}{\omega^2}} \right] = 1 - (1 - \bar{\gamma}_1) \frac{\omega_b^2}{\omega_1^2} - \sum_{i=2}^{N} \bar{\gamma}_i \left[\frac{1}{1 - \dfrac{\omega_i^2}{\omega_1^2} (1 - \bar{\gamma}_1)} \right] \tag{4.24}$$

$$\omega_1^{*2} = \frac{\omega_1^2}{1 - \bar{\gamma}_1 \left[\dfrac{1}{1 - (1 - \bar{\gamma}_1)\dfrac{\omega_b^2}{\omega_1^2} + \displaystyle\sum_{i=2}^{N} \dfrac{\bar{\gamma}_i}{1 - \dfrac{\omega_i^2}{\omega_1^2}(1 - \bar{\gamma}_1)}} \right]} \tag{4.25}$$

$$\bar{\gamma}_k \left[\frac{1}{1 - \dfrac{\omega_k^2}{\omega^2}} \right] = 1 - (1 - \bar{\gamma}_k)\frac{\omega_b^2}{\omega_k^2} - \sum_{\substack{i=1 \\ i \neq k}}^{N} \bar{\gamma}_i \left[\frac{1}{1 - \dfrac{\omega_i^2}{\omega_k^2}(1 - \bar{\gamma}_k)} \right] \tag{4.26}$$

$$\omega_k^{*2} = \frac{\omega_k^2}{1 - \bar{\gamma}_k \left[\dfrac{1}{1 - (1 - \bar{\gamma}_k)\dfrac{\omega_b^2}{\omega_k^2} - \displaystyle\sum_{\substack{i=1 \\ i \neq k}}^{N} \dfrac{\bar{\gamma}_i}{1 - \dfrac{\omega_i^2}{\omega_1^2}(1 - \bar{\gamma}_k)}} \right]} \tag{4.27}$$

$$\bar{q}_i^j = -\frac{\omega_j^{*2}}{\omega_j^{*2} - \omega_i^2} L_i \tag{4.28}$$

$$\psi^{j^T} = \left\{ 1,\ \sum_{i=1}^{N} \bar{q}_i^j \phi_1^i\ ,\ \sum_{i=1}^{N} \bar{q}_i^j \phi_2^i\ ,\ \sum_{i=1}^{N} \bar{q}_i^j \phi_3^i\ , \cdots \right\} \tag{4.29}$$

$$m_1 \ddot{v}_1 = -k_{12}(v_1 - v_2)$$

$$m_1 \ddot{v}_1 + m_2 \ddot{v}_2 = -k_{23}(v_2 - v_3)$$

$$\sum_{i=1}^{j} m_i \ddot{v}_i = -k_{j,j+1}(v_{j+1} - v_j)$$

Setting all $m_i = m_0$, $\mathbf{v} = e^{i\omega_1 t}\,\phi^1$ with $\phi^{1^T} = (1,2,3,4,5)$, we find

$$k_{12} = 5\,\omega_1^2 m_0$$

$$k_{23} = 9\,\omega_1^2 m_0$$

$$k_{34} = 12\,\omega_1^2 m_0$$

$$k_{45} = 14\,\omega_1^2 m_0$$

$$k_{56} = 15\,\omega_1^2 m_0$$

$$\mathbf{K} = \omega_1^2 m_0 \begin{bmatrix} 5 & 5 & 0 & 0 & 0 \\ -5 & 14 & -9 & 0 & 0 \\ 0 & -9 & 21 & -12 & 0 \\ 0 & 0 & -12 & 26 & -14 \\ 0 & 0 & 0 & -14 & 29 \end{bmatrix}$$

$$\Phi = \left[\phi^{1,}\phi^{2,}\phi^{3,}\phi^{4,}\phi^5\right] = \begin{bmatrix} 5 & -2.143 & 0.75 & -0.179 & 0.024 \\ 4 & 0.429 & -1.50 & 0.821 & -0.091 \\ 3 & 1.571 & -0.25 & -1.179 & 0.643 \\ 2 & 1.643 & 1.00 & 0.071 & -1.143 \\ 1 & 1.000 & 1.00 & 1.000 & 1.000 \end{bmatrix}$$

$$\omega_2^2 = 6\,\omega_1^2, \quad \omega_3^2 = 15\,\omega_1^2, \quad \omega_4^2 = 28\,\omega_1^2, \quad \omega_5^2 = 45\,\omega_1^2$$

$$L_i = \frac{\phi^{i^T}\,\mathbf{M}\,\mathbf{r}}{\phi^{i^T}\,\mathbf{M}\,\phi^i}$$

$$\bar{\gamma}_i = \frac{L_i^2\, m_i}{m + m_b} = \frac{(\phi^{i^T}\,\mathbf{M}\,\mathbf{r})^2}{(\phi^{i^T}\,\mathbf{M}\,\phi^{i^T})(m + m_b)}$$

$$\bar{\gamma}_i = \frac{(\phi^{i^T}\,\mathbf{r})^2}{6\,(\phi^{i^T}\,\phi^i)}$$

$$\omega_b^{*2} = \omega_b^2 \left\{ 1 - \sum_{i=1}^{5} \gamma_i \frac{\omega_b^2}{\omega_i^2} \right\}$$

$$\omega_1^{*2} = 3.428\,\omega_1^2$$

$$\omega_2^{*2} = 1.050\,\omega_2^2$$

$$\omega_3^{*2} = 1.018\,\omega_3^2$$

$$\omega_4^{*2} = 1.00788\,\omega_4^2$$

$$\omega_5^{*2} = 1.0034\,\omega_5^2$$

Table 4-1

Frequencies

$(m_o) = 1\ 000$ kips, $\omega_1 = \dfrac{2\pi}{0.4}$ rads/sec

	Fixed Base	Isolated (Approx.)	Isolated (Exact)
0	-	3.0973	3.0980
1	15.71	25.42	25.977
2	38.48	50.96	47.031
3	60.84	70.48	67.913
4	83.12	89.67	88.585
5	105.37	109.14	108.83

$$\gamma\,\ddot{v}_s + \ddot{v}_b + 2\,\omega_b\,\beta_b\,\dot{v}_b + \omega_b^2\,v_b = -\ddot{u}_g \tag{4.30}$$

$$\ddot{v}_s + \ddot{v}_b + 2\omega_s \beta_s \dot{v}_s + \omega_s^2 v_s = -\ddot{u}_g \tag{4.31}$$

$$\gamma = \frac{m}{m + m_b} = \frac{m}{M}$$

$$\gamma = \frac{L_1^2 M_1}{m + m_b} = \bar{\gamma}_1$$

$$|v_b|_{max} = \frac{1}{\omega_b^2} S_A(\omega_b, \beta_b) \tag{4.32}$$

$$C = \left\{ S_A^2(\omega_b^*, \beta_b^*) + \varepsilon^2 (1-\gamma)^2 S_A^2(\omega_s^*, \beta_s^*) \right\}^{1/2} \tag{4.33}$$

$$|L_1 v_b|_{\max} = \frac{1}{\omega_b^2} L_1 S_A\,(\omega_b\,,\beta_b) \tag{4.34}$$

$$|q_1|_{\max} = \left\{ \frac{\varepsilon^2 L_1^2\, S_A^2(\omega_b^*,\beta_b^*)}{\omega_b^{*4}} + \frac{\varepsilon^2 L_1^2\, S_A^2(\omega_s^*,\beta_s^*)}{\omega_s^{*4}} \right\}^{1/2} \tag{4.35}$$

$$\mathbf{v} = q_1 \phi^1 \tag{4.36}$$

$$\mathbf{F} = \mathbf{K}\,\mathbf{v} = q_1 \mathbf{K}\,\phi^1 = q_1 \mathbf{M}\,\phi^1\,\omega_1^2 \tag{4.37}$$

$$\mathbf{r}^T\,\mathbf{F} = q_1\,\omega_1^2 L_1 M_1 \tag{4.38}$$

$$C_s\, m = \mathbf{r}\,\mathbf{F} \tag{4.39}$$

$$C_s = \frac{L_1 M_1}{m} \left\{ L_1^2\, S_A^2(\omega_b\,, \beta_b) + (1-\overline{\gamma}_1)^2\, \varepsilon^2 L_1^2\, S_A^2(\omega_s^*\,, \beta_s^*) \right\}^{1/2}$$

$$= \frac{L_1^2\, M_1}{m} \left\{ S_A^2(\omega_b\,, \beta_b) + (1-\overline{\gamma}_1)^2\, \varepsilon^2 S_A^2(\omega_s^*\,, \beta_s^*) \right\}^{1/2} \tag{4.40}$$

$$F_x = C_{V_x}\, V \tag{4.41}$$

$$C_{V_x} = \frac{W_x\, h_x^j}{\sum_i W_i\, h_i^j}$$

$$V_x = \sum_{i=x}^{N} F_i \tag{4.42}$$

$$F_x = C_{V_x} V \tag{4.43}$$

$$C_{V_x} = \frac{W_x}{W} \tag{4.44}$$

$$V_x = \sum_{i=x}^{N} C_{V_i} V = V \sum_{i=x}^{N} \frac{W_i}{W} = C_s \sum_{i=x}^{N} W_i \tag{4.45}$$

NSSP
Nuclear Systems Safety Program

Seismic Design Guidance for DOE Facilities

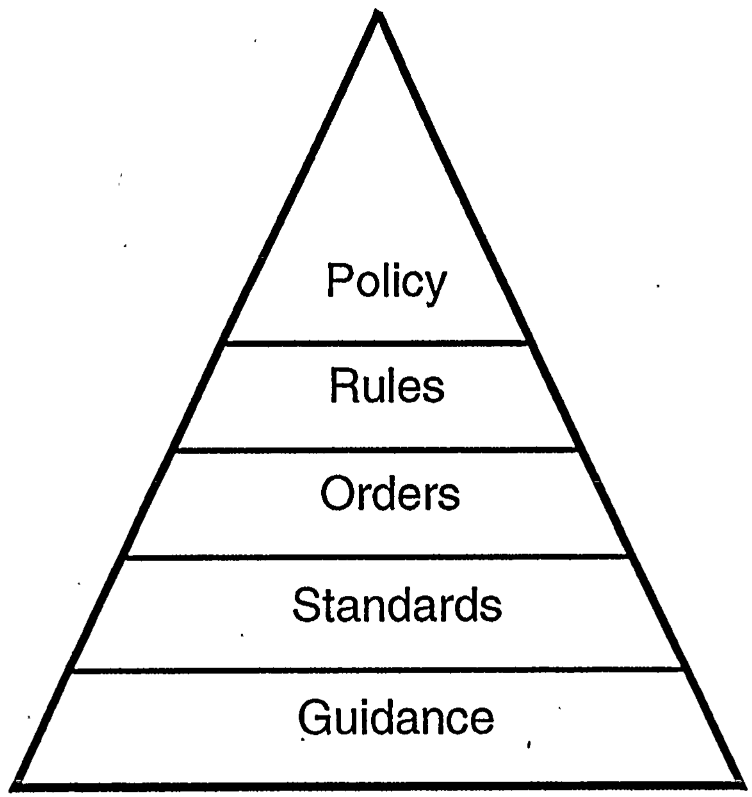
Tom Nelson
Nuclear Systems Safety Program
Lawrence Livermore National Laboratory

Department of Energy
Short Course on
Seismic Base Isolation

Berkeley Marina Marriott
Berkeley, California

August 10–14, 1992

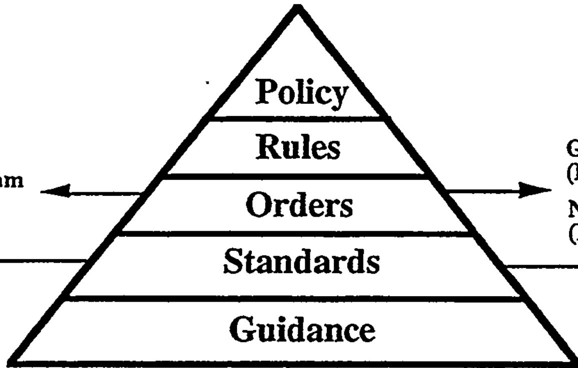
DOE Natural Phenomena Hazard Program has a hierarchy of requirements



APL92-365/V-2

Existing, Draft, and Future Standards and Guidance

Seismic Safety Program
(DOE 5480.Safety)



General Design Criteria
(DOE 6430.1A)

Natural Hazards Regulations
(DOE 5480.NPH)

DOE STANDARDS

- o Natural Phenomena Hazards Design and Evaluation Standard for Department of Energy Facilities
- x Natural Phenomena Hazards Performance Categorization Standard for Structures, Systems, and Components at Department of Energy Facilities
- • Natural Phenomena Hazards Prioritization and Screening Standard for Evaluation and Upgrade of Existing Department of Energy Facilities

NATIONAL CODES AND STANDARDS

Model Building Codes - UBC (ICBO)
NEHRP Guidelines
ASCE Standard 4 and 7
ASME
ACI, AISC, AISI
Seismic Experience Data & Procedures
NRC Guidelines
DOD Manuals
ATC Manuals
FEMA Documents

GUIDANCE

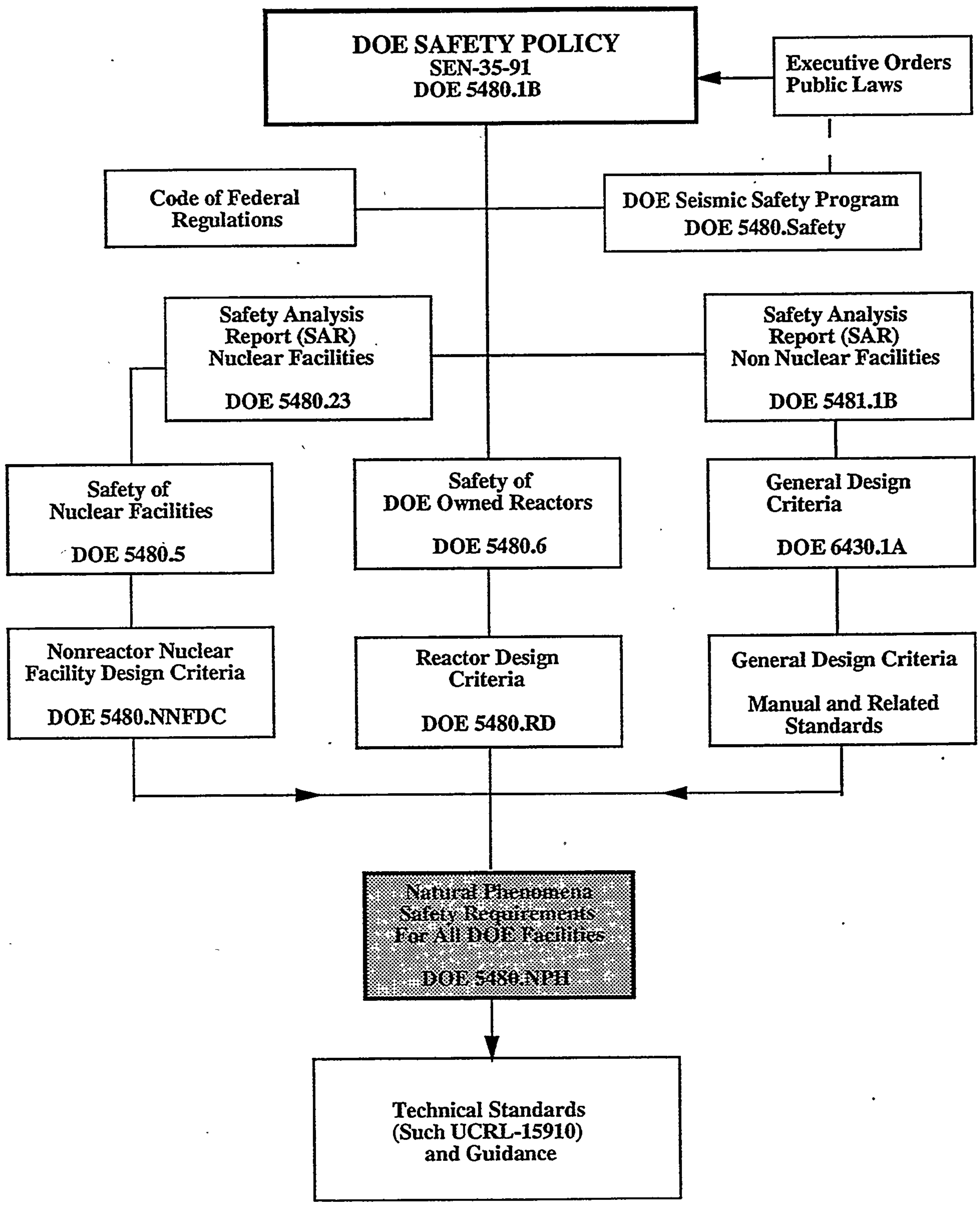
- o Walkdown Guide
- o Seismic Design and Detailing
- o Wind Design and Detailing
- o Suspended Ceilings Bracing
- o Equipment Tiedown
- x Site Characteristics
- x Hazard Assessment for Seismic and Wind
- x Interim Position on Seismic Hazard
- x Existing Facility Evaluation
- x Tornado Missile Study
- x Seismic Instrumentation
- • Expanding evaluation of existing systems & components
- • Expanded evaluation of existing buildings
- • Design of systems & components by experience data
- • Expanded GSG Studies
- • Lifelines Evaluation

- o Seismic Safety Guide
- o Seismic Hazard Studies
- o Wind Hazard Studies
- o Flood Hazard Studies
- o High Level Waste Tanks
- x Response Spectra Development Methodology
- • Spectra Compatible PSD & Time-History Generation
- • Generic site-dependent spectra using new data
- • Generic site correction factors for WNA and ENA
- • SSI Guidance
- • New LLNL East US Seismic Hazard Assessment
- • Seismic evaluation of piping
- • Response spectra, PSD & time history generation
- • Base Isolation Design Criteria
- • Masonry Wall Criteria
- • Anchorage Design

o existing
x draft
• future

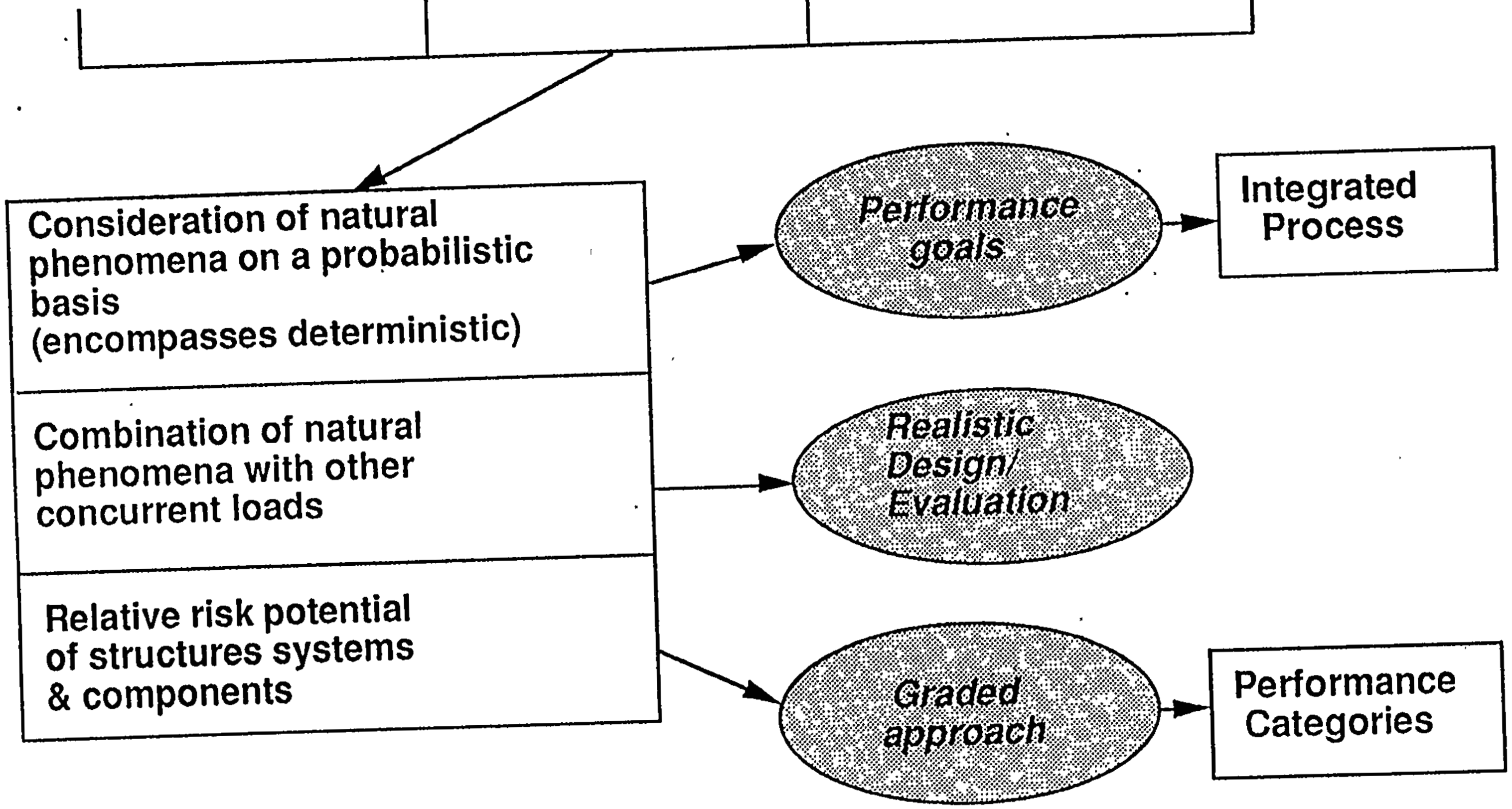
APL92-365/V-3

DOE Safety Documents for Natural Phenomena Hazards



Proposed DOE NPH Policy encompasses the following aspects:

- Design of new facilities
- Evaluation of existing facilities
- Additions, modifications and upgrades
- Evaluate and update site natural phenomena hazards



APL92-365/V-5

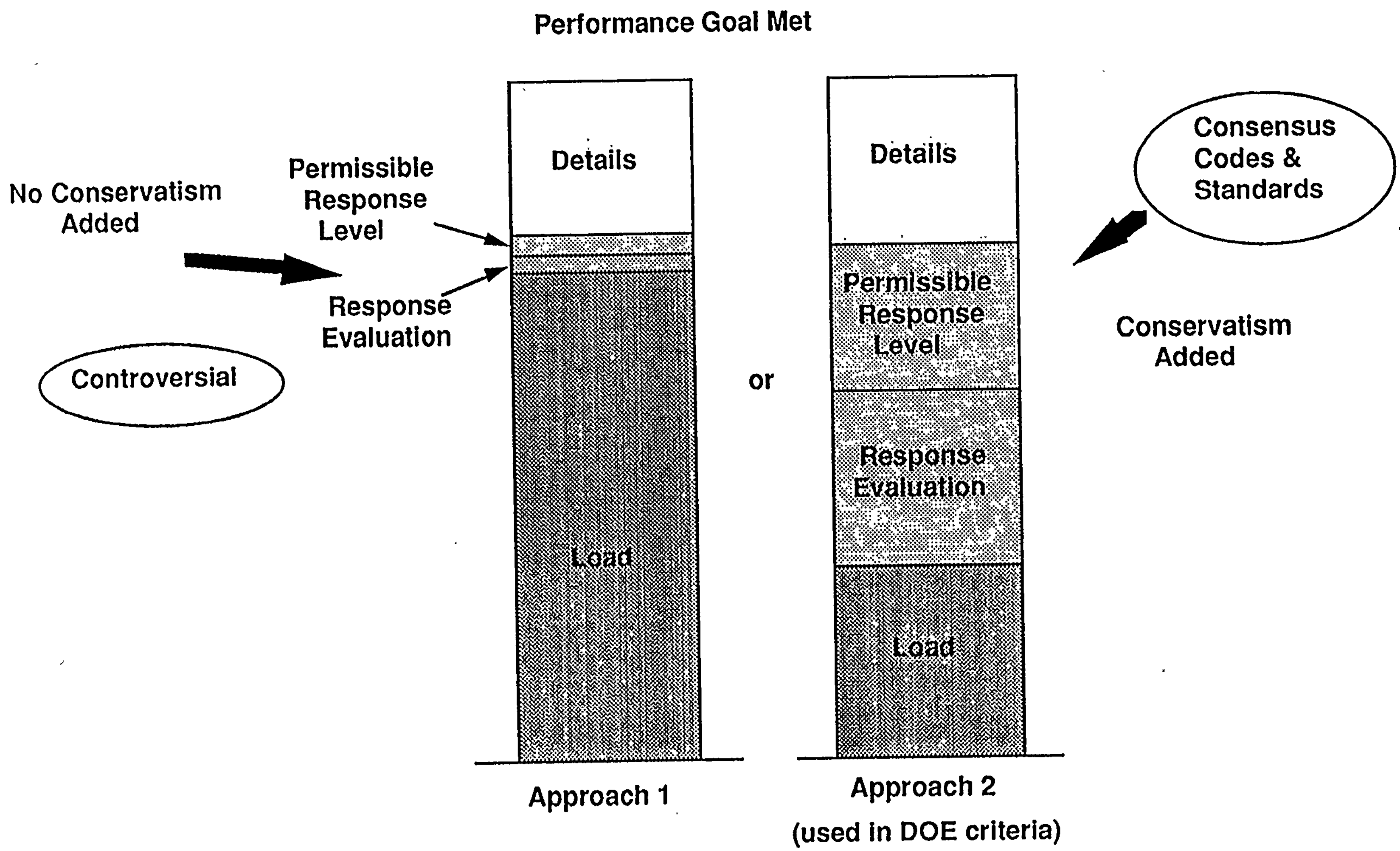
Motivation for performance goals

- UBC approach (recent)
- Used for DOE NPRs
- New NRC approach (e.g. ALWR & 10 CFR 100 Appendix B)
- Quantifies graded approach
- Provides means for meeting DOE safety goals
- Enables development of consistent criteria
  - Across earthquakes, winds, and floods
  - Facilities across U.S.

Performance Goal ═ Annual probability of exceeding acceptable behavior limits when subjected to Natural Phenomena Hazards

APL92-365/V-6

Possible approaches to achieve performance goals



APL92-365/V-7

Motivation for graded approach

- Graded approach is common practice
  - UBC
  - NRC
  - DOD
- Graded approach enables facility design/evaluation consistent with cost, mission, and hazard
  - Few "reactor" facilities
  - Many facilities with wide variety of risk potential
- Graded approach enables cost-benefit studies and establishment of priorities for existing facilities
  - Few new designs
  - Evaluation of existing facilities requires:
    - cost benefit consideration
    - prioritize upgrading and retrofit

APL92-365/V-8

Recommended NPH Policy is consistent with DOE Nuclear Safety Policy

DOE Draft Nuclear Safety Policy	Draft recommended NPH Policy
• No individual bears significant added risk from DOE operations above risks of general population normal exposure	graded approach performance goals (relative risk)
• Use safety goals as aiming points (targets) for performance	performance goals (targets)
• Maintain proper balance of safety, production goals, and cost, such that safety is fully integrated	graded approach (cost-benefit)
• Remain consistent with latest developments in science and engineering	Up to date (state-of-the-art))
• Proposed modifications to existing facilities shall be prioritized based on their safety significance	graded approach (prioritize)

APL92-365/V-9

UCRL 15910 is the current DOE document for design and evaluaton of new and existing facilities for natural phenomena hazards

- Developed by team (consensus document)
- Simplified and consistent approach
- Prescribed for new design by DOE 6430.1A
- Used for existing facilities

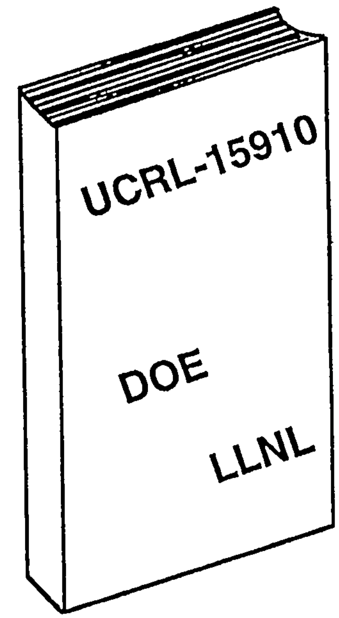
Hanford	Mound
Savannah River	LLNL
ORNL	Kansas City Plant

- Presented in technical papers since 1985 (hazard since 1975)
- 3 conferences: ~ 500 participants
- 6 workshops; over 300 participants — 2 /yr planned
- Adopted by DOE/DP for restart (SEP)

APL92-365/V-10

The Guidelines: UCRL-15910

- Provide the natural phenomena design and evaluation basis for DOE facilities
  - Performance based guidelines for earthquakes, winds, and floods such that specified levels of protection are achieved
- Applies to structures, systems, and components
  - 4 sets of deterministic procedures ranging:
    - from those provided by the UBC for normal facilities
    - to near those provided by nuclear power plant criteria



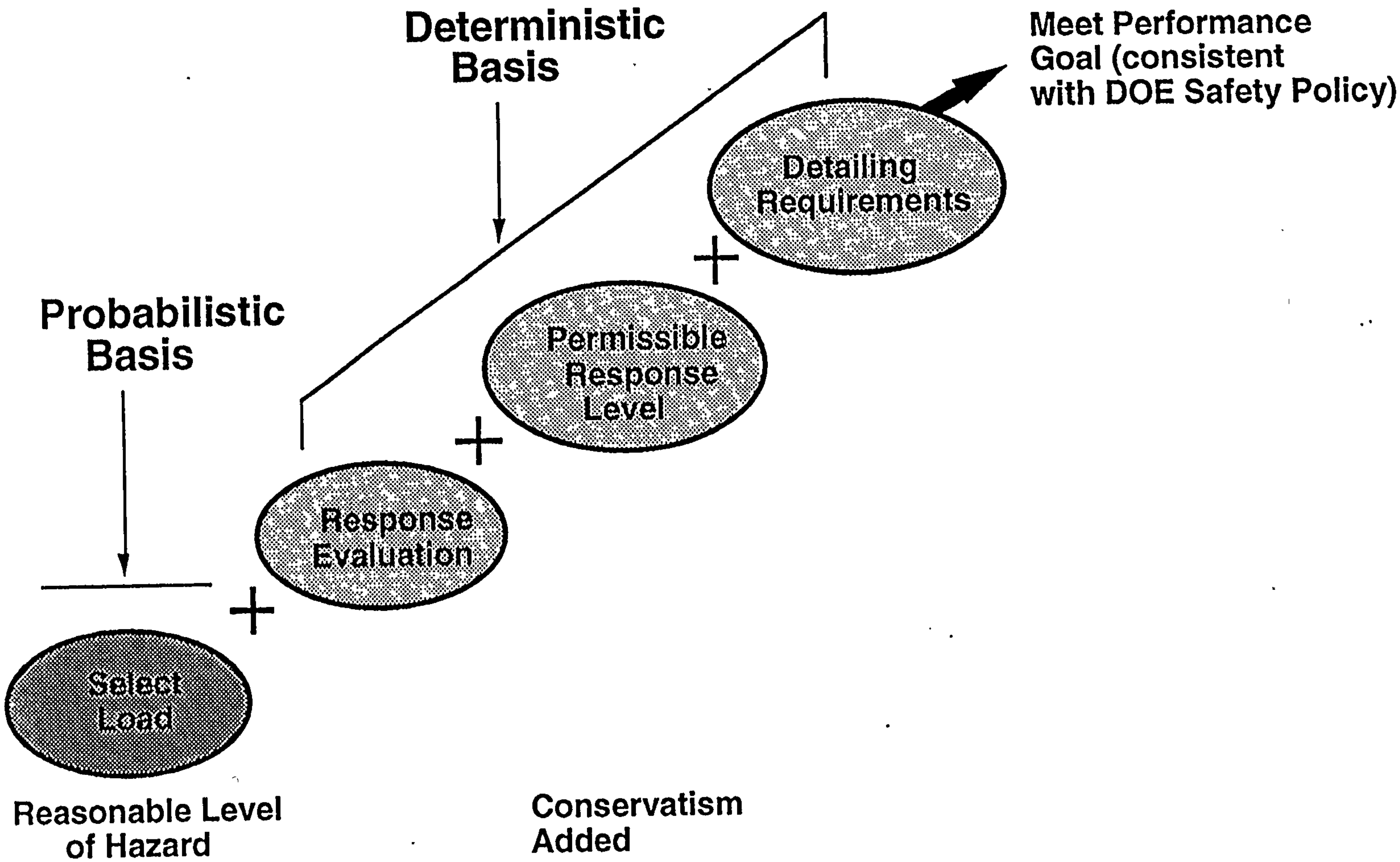
APL92-365/V-11

Approach and philosophy for UCRL-15910

- Graded Approach
  - Different sets of criteria to apply depending on cost, importance, mission, level of safety
  - Range from low probability of damage/failure to very high confidence of extremely low probability of damage/failure
- Performance Goal Philosophy
  - Specification of NPH hazard probability to define loadings
  - Deterministic design/evaluation procedures with controlled conservatism to meet probabilistic performance goal
  - Means of achieving consistent criteria for all NPH at all locations across U.S.

APL92-365/V-12

NPH Design Criteria is a system!



APL92-365/V-13

UCRL 15910 Improvements

- "Reactor Facility" criteria
- Link safety categories to component performance goals
- Expand requirements for existing facilities
- Expand to other systems and components
- Maintain document with improvements in state-of-the-art engineering
  - Eventually, base isolation guidance will be added

APL92-365/V-14

Proposed performance categories

Performance		Description
UBC Performance	0	No safety significance
	1	Life safety only
	2	Minimal interruption to function
P_u facility performance	3	High confidence of confinement / function
or Power reactor reevaluation performance	4	Very high confidence of confinement / functuion
Power reactor performance	5	Extremely high confidence of confinement / function

APL92-365/V-15

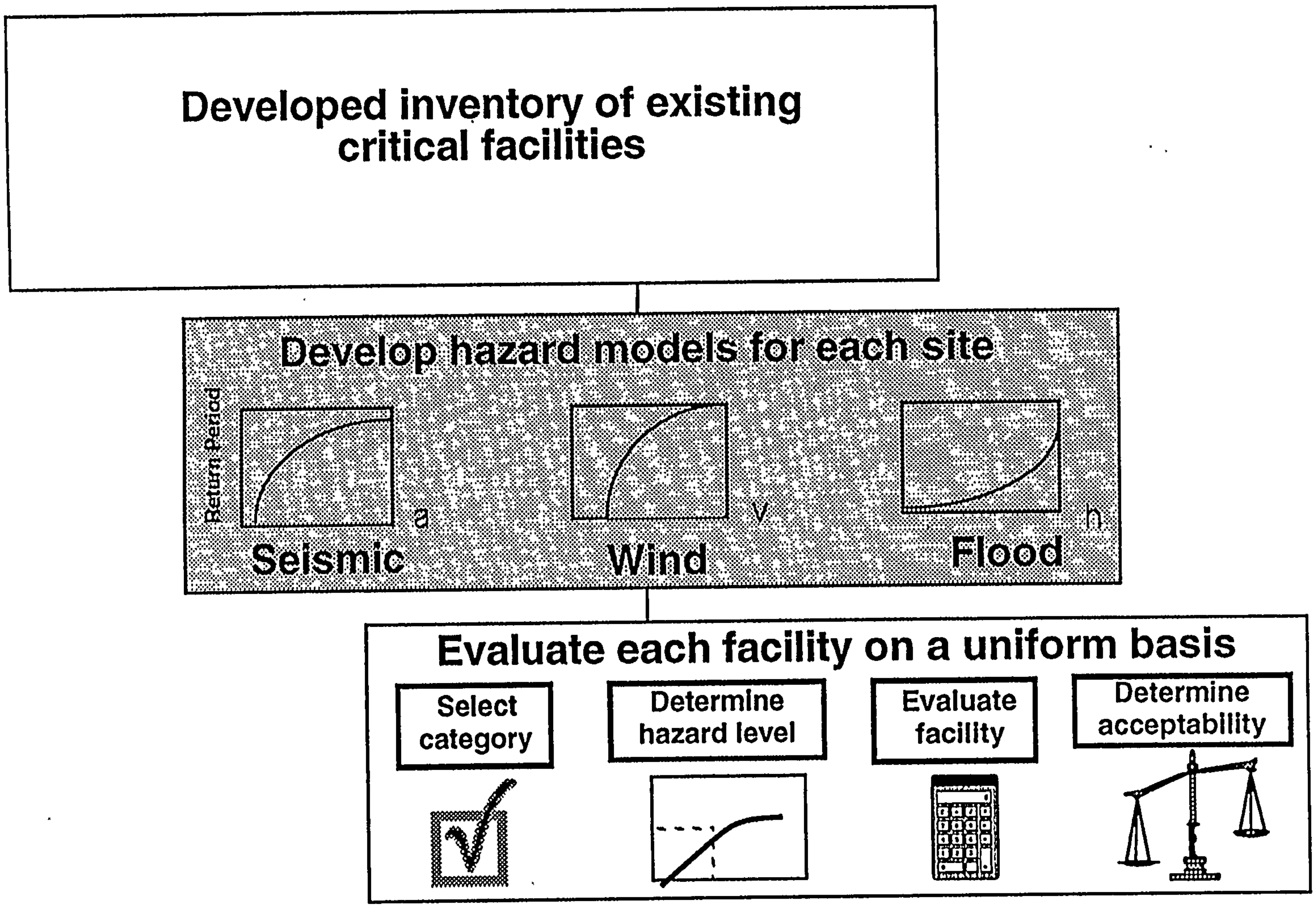
Performance Goals & Recommended Hazard Probabilities

Performance Category	Performance Goal	Hazard Exceedance Probability	Ratio of Hazard to Probability *
PC1	1×10^{-3}	2×10^{-3}	2
PC2	5×10^{-4}	1×10^{-3}	2
PC3	1×10^{-4}	1×10^{-3}	10
PC4	1×10^{-5}	2×10^{-4}	20
PC5	Explicit criteria not given		

* Differences in performance goal and hazard probabilities indicate that conservatism must be introduced in the evaluation approach

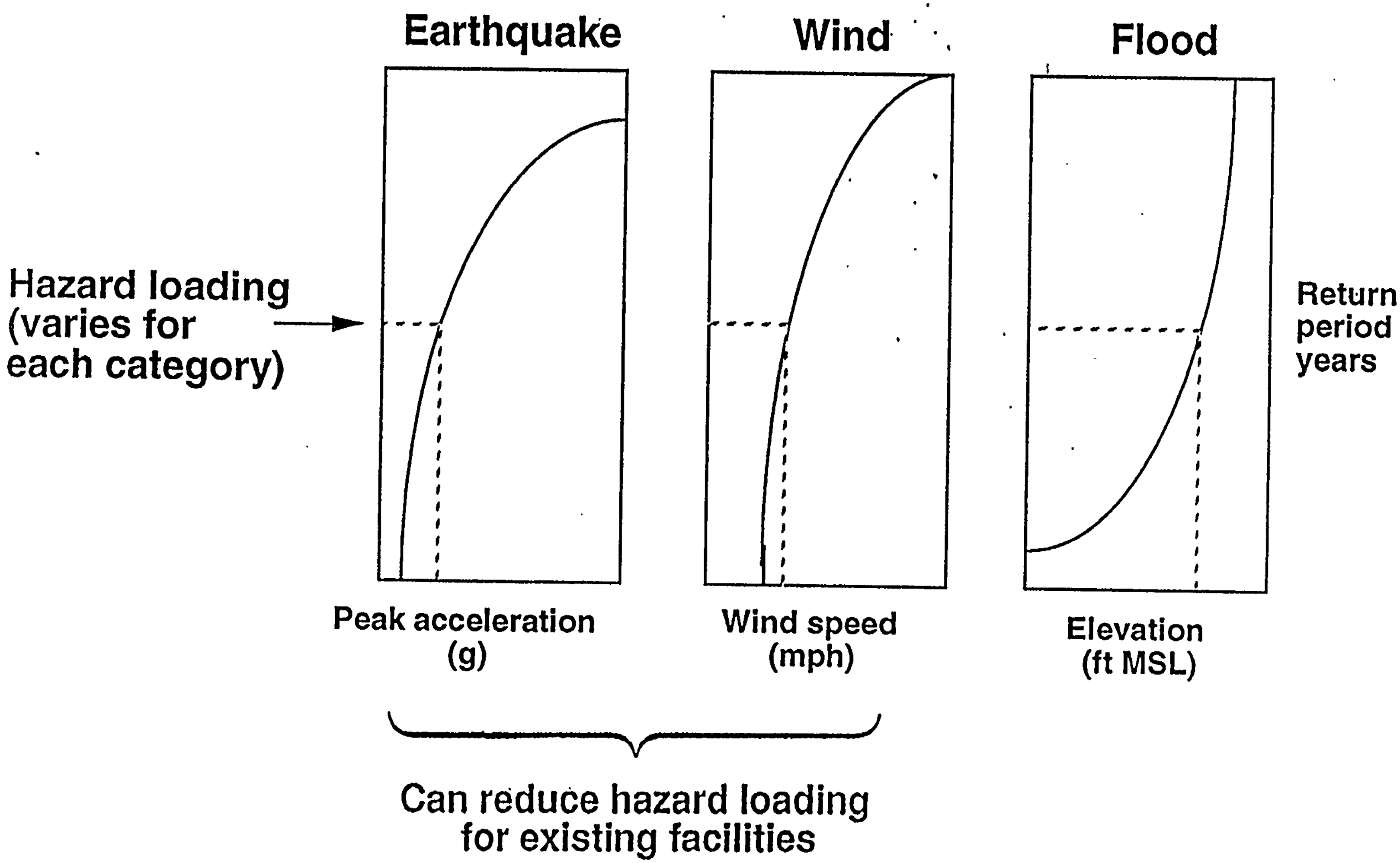
APL92-365/V-16

We have developed a simplified approach in UCRL 15910 Natural Phenomena Hazards protection



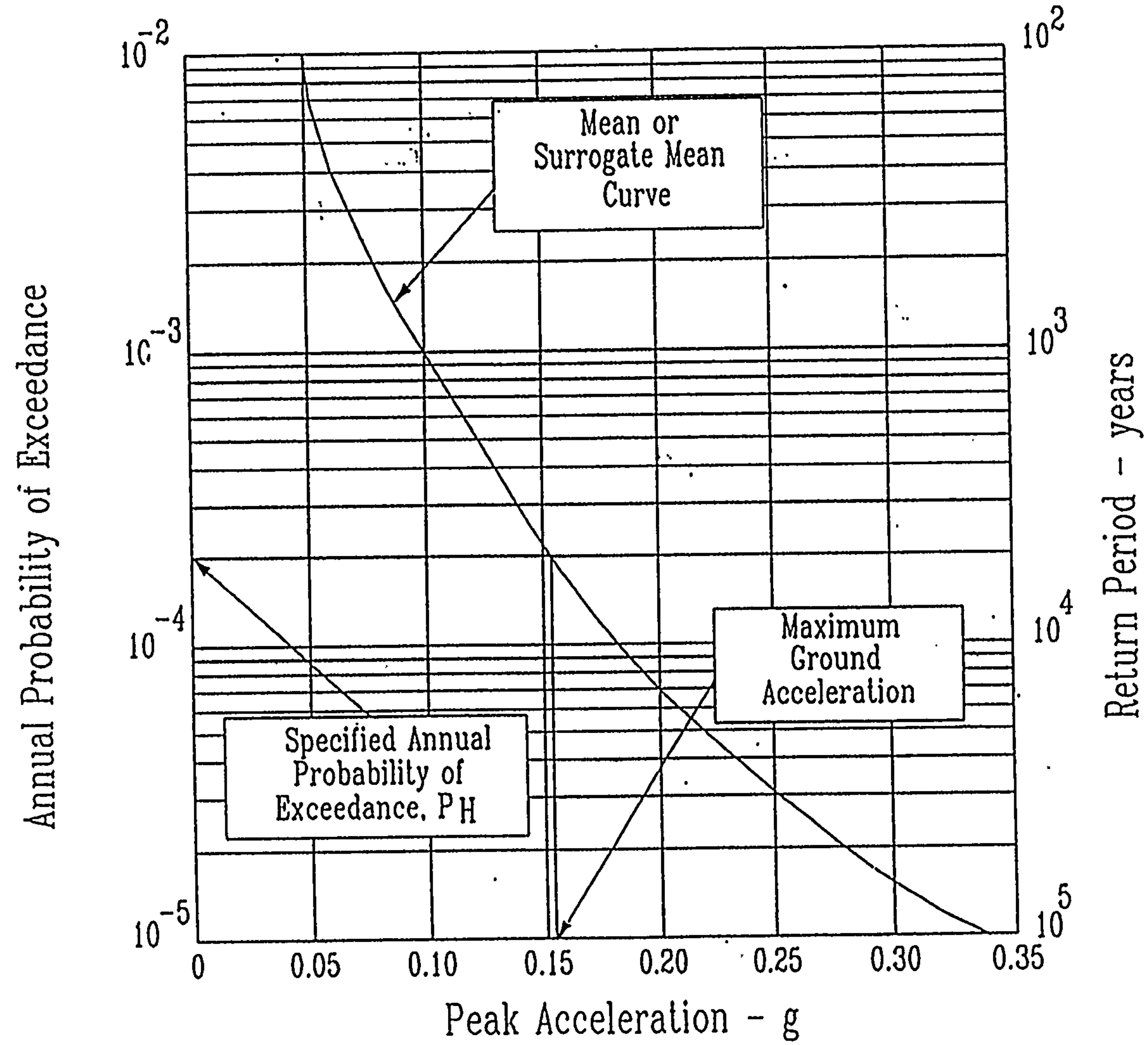
APL92-365/V-17

Determine the hazard loading for facilities being designed or evaluated



APL92-365/V-18

Seismic Hazard Curve



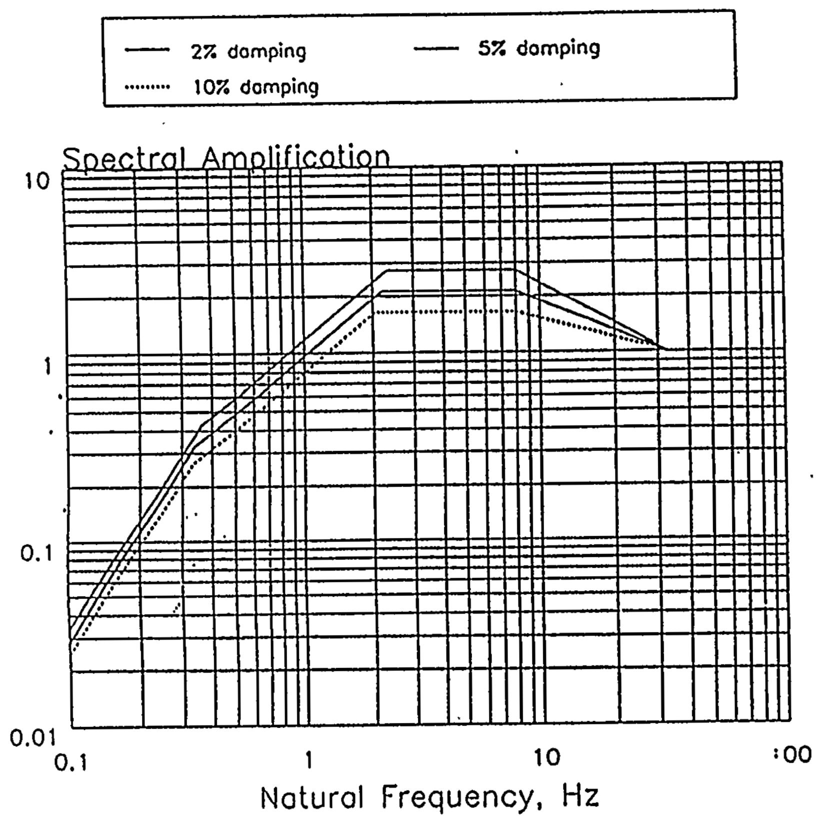
APL92-365/V-19

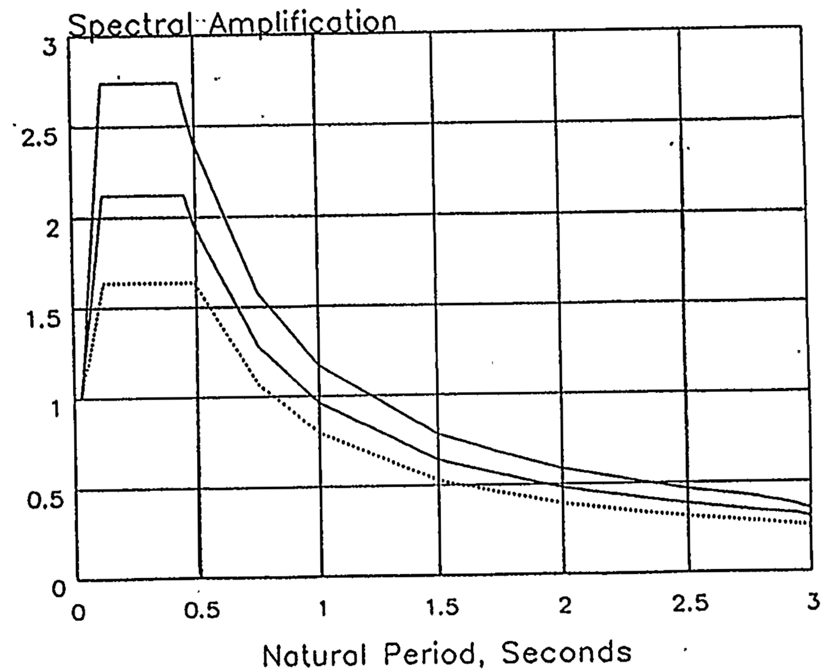
Earthquake Ground Response Spectra

- Design/evaluation spectra smoothed & broadened to provide practical input to seismic analyses
- Spectra gives frequency content of earthquake motion
- Spectral amplification depends strongly on site conditions
- Use median amplification spectra based on site specific geotechnical studies

APL92-365/V-20

NUREG/CR-0098 Ground Response Spectra
Rock Site





APL92-365/V-21

This approach has:

- Different categories of performance
  - 4 set of guidelines provided
  - Performance goals
    UBC ⟶ near NRC
- *Probabilistic* hazard loading
- *Deterministic* evaluation methods → Response, Capacity, Detailing } Using Industry Codes and Standards

APL92-365/V-22

LLNL/DOE SHORT COURSE OF SEISMIC (BASE) ISOLATION

SELECTION OF ISOLATION AS A DESIGN STRATEGY

BY

Charles A. Kircher Ph.D., P.E.

August 11, 1992

SELECTION OF ISOLATION AS A DESIGN STRATEGY

PRESENTATION TOPICS

SELECTION OF ISOLATION AS A DESIGN STRATEGY

BENEFITS OF ISOLATION

- REDUCE EARTHQUAKE RISK
  - Improve life-safety
  - Protect function:
    - Reduce likelihood (and duration) of closure of essential facilities (e.g., hospitals and emergency operation centers)
    - Reduce likelihood of failure of safety-related systems and equipment (e.g., nuclear/hazardous materials facilities)
  - Protect investment:
    - Reduce direct economic loss due to earthquake damage of structural and architectural elements, nonstructural components and contents of buildings
    - Reduce direct economic loss due to earthquake damage of fragile equipment (e.g., computers on raised-access flooring, electrical switchyard gear, SLAC research equipment)
    - Reduce collateral economic loss due to business interruption, temporary space rental and relocation costs

SELECTION OF ISOLATION AS A DESIGN STRATEGY

BENEFITS OF ISOLATION

- IMPROVE DESIGN FEASIBILITY
  - Reduce seismic demand on vulnerable structural systems (e.g., "irregular" structures) and fragile equipment (e.g., ALMR vessels)
  - Reduce extent of seismic modifications required for upgrade of existing structures (e.g., Golden Gate Bridge)
  - Preserve the architecture and materials of historical buildings (e.g., Salt Lake City and County Building)
- PROVIDE CONSTRUCTION ECONOMY
  - Reduce cost of constructing new buildings that have special seismic performance requirements (e.g., Los Angeles County Fire Command Center)
  - Reduce cost of renovating existing buildings that would otherwise require extensive modification of structural and architectural elements and nonstructural components (e.g., San Francisco and Oakland City Halls)

SELECTION OF ISOLATION AS A DESIGN STRATEGY

EXPECTED EARTHQUAKE PERFORMANCE OF CONVENTIONAL (NON-ISOLATED) BUILDINGS DESIGNED TO "CODE"

Chapter 1 Commentary: SEAOC Blue Book[1]

"The primary function of these recommendations is to provide minimum standards for use in building design regulation to maintain public safety in the extreme earthquakes likely to occur at the building's site. These recommendations primarily are intended to safeguard against major failures and loss of life, not to limit damage, maintain functions, or provide for easy repair. It is emphasized that the purpose of these recommended design procedures is to provide buildings that are *expected* to meet this life safety objective."

"Structures designed in accordance with these Recommendations should, in general, be able to:

1. Resist a minor level of earthquake ground motion without damage;
2. Resist a moderate level of earthquake ground motion without structural damage, but possibly experience some nonstructural damage;
3. Resist a major level of earthquake ground motion having an intensity equal to the strongest either experienced or forecast for the building site, without collapse, but possibly with some structural as well as nonstructural damage."

1. Seismology Committee, *Recommended Lateral Force Requirements and Commentary,* Structural Engineers Association of California, Sacramento, California, 1990.

SELECTION OF ISOLATION AS A DESIGN STRATEGY

EARTHQUAKE PERFORMANCE OF NEW STATE BUILDINGS[1]

Earthquake Performance Objectives	Post-Earthquake Functions Within	Building Standards [1]	Occupancy Categories [2]: Hospitals, Essential Services	Hazardous Materials	Public Schools	Nursing, Prisons	University, Research	Offices, Courts	Other Occupancies	
Fully Functional, no significant damage	Immediate	Nuclear Reg. Commission	✻	✻	✻	✻	✻	✻	✻	
Immediate Occupancy, minimal post-earthquake disruption, some non-structural cleanup required	Hours	Title 24 I = 1.50, 1.25	✦ [3]	✦ [4]	○	○	○	○	○	
Repairable Damage, some structural and nonstructural damage, will not significantly jeopardize life	Days to Months	Title 24 I = 1, 1.15 / Current UBC[5]	●	●	✦	✦	✦	✦	✦	
Substantial Life Safety, significant damage may not be repairable, will not significantly jeopardize life	Year(s)	None	●	●	●	●	●	●	●	
Life Hazards Reduced, unrepairable damage very likely, some falling hazards, building may be a total loss, low life hazards.	No Limit	None	●	●	●	●	●	●	●	
Very Poor Life Safety, collapse likely, unrepairable damage and total loss highly likely, significant life hazards	No Limit	None	●	●	●	●	●	●	●	
Unsafe for Occupancy	No Limit	None	●	●	●	●	●	●	●	
Unknown Performance	No Limit	None	●	●	●	●	●	●	●	

Key:

✦ = Minimum Acceptable Earthquake Performance Objective
○ = Acceptable Earthquake Performance Objective
● = Unacceptable Earthquake Performance Objective
✻ = Typically does not apply, except to nuclear facilities

Abbreviations:

I—Occupancy Importance Factor (pursuant to Ch. 23, Title 24)
Title 24 (Part 2, California Code of Regulations)—California Building Code
UBC—Uniform Building Code

Footnotes:

1—Applies to all new state buildings in accordance with existing state laws and regulations.
2—Emergency and recovery plans required for all occupancies.
3—Communications, emergency services, and acute care services shall be capable of functioning after earthquakes, as well as having immediate occupancy throughout the building.
4—Acceptable if chance of release of hazardous materials is remote.
5—Applies to state leased buildings.

1. State of California Seismic Safety Commission, "Policy on Acceptable Levels of Earthquake Risk in State Buildings," Report No. SSC 91-1, January 1991.

Charles Kircher & Associates
Consulting Engineers

SELECTION OF ISOLATION AS A DESIGN STRATEGY

EARTHQUAKE PERFORMANCE OF EXISTING STATE BUILDINGS[1]

Earthquake Performance Objectives	Post-Earthquake Functions Within	Building Standards[1]	Occupancy Categories[2]: Hospitals, Essential Services	Hazardous Materials	Public Schools	Nursing, Prisons	University, Research	Offices, Courts	Other Occupancies	Historic, Non-essential
Fully Functional, no significant damage	Immediate	Nuclear Reg. Commission	✻	✻	✻	✻	✻	✻	✻	✻
Immediate Occupancy, minimal post-earthquake disruption, some non-structural cleanup required	Hours	Title 24 I = 1.50, 1.25	✦[3]	○[4]	○	○	○	○	○	○[5]
Repairable Damage, some structural and nonstructural damage, will not significantly jeopardize life	Days to Months	Title 24 I = 1, 1.15 Current UBC[6]	●	○[4]	○	○	○	○	○	○[5]
Substantial Life Safety, significant damage may not be repairable, will not significantly jeopardize life	Year(s)	75% of the 1988 UBC; ATC 14 & 22; or 1973 UBC[7]	●	✦[4]	✦	✦	✦	✦	✦	○[5]
Life Hazards Reduced, unrepairable damage very likely, some falling hazards, building may be a total loss, low life hazards.	No Limit	UCBC Appendix Ch. 1 for URM Bearing Wall Buildings	●	●	●	●	✦[8]	✦[8]	✦[8]	✦[5]
Very Poor Life Safety, collapse likely, unrepairable damage and total loss highly likely, significant life hazards	No Limit	None	●	●	●	●	●	●	●	●
Unsafe for Occupancy	No Limit	None	●	●	●	●	●	●	●	●
Unknown Performance	No Limit	None	●	●	●	●	●	●	●	●

Key:
✦ = Minimum Acceptable Earthquake Performance Objective
○ = Acceptable Earthquake Performance Objective
● = Unacceptable Earthquake Performance Objective
✻ = Typically does not apply, except to nuclear facilities

Abbreviations:

ATC—Applied Technology Council
I—Occupancy Importance Factor (pursuant to Ch. 23, Title 24)
Title 24 (Part 2, California Code of Regulations)—California Building Code
UBC—Uniform Building Code
UCBC—Uniform Code for Building Conservation
URM—Unreinforced Masonry

Footnotes:
1—Most building standards are not currently required by law for existing buildings, unless triggered by voluntary or mandatory strengthening, major alterations, additions, or changes of occupancy. This policy recommends that all existing state government buildings meet minimum earthquake performance objectives by the year 2000.
2—Emergency and recovery plans required for all occupancies.
3—Communications, emergency services, and acute care services shall be capable of functioning after earthquakes, as well as having immediate occupancy throughout the building.
4—Acceptable if chance of release of hazardous materials is remote.
5—Acceptable if anticipated earthquake damage is repairable, and the building also complies with the State Historical Building Code.
6—Applies to state leased buildings.
7—A uniform seismic retrofit building standard must be developed.
8—Acceptable for strengthened URM bearing wall buildings only.

1. State of California Seismic Safety Commission, "Policy on Acceptable Levels of Earthquake Risk in State Buildings," Report No. SSC 91-1, January 1991.

SELECTION OF ISOLATION AS A DESIGN STRATEGY

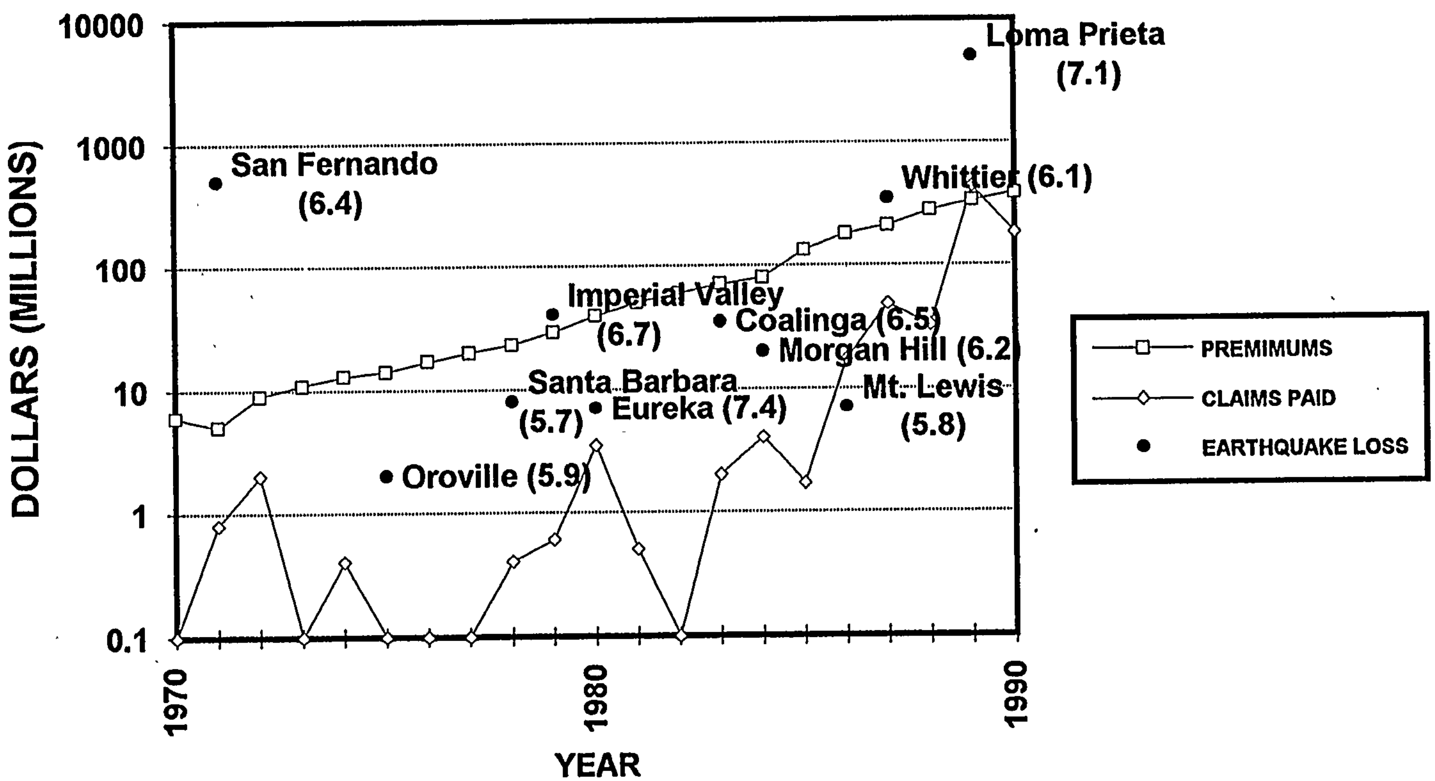
EXPECTED LOSSES IN A MAJOR EARTHQUAKE: LOS ANGELES AND SAN FRANCISCO AREAS[1]

Fault Creating the Earthquake	Magnitude of Earthquake	Deaths	Hospitalized Injuries	Property Damage
San Andreas Fault (Los Angeles)	8.3	3,000 to 12,000	12,000 to 48,000	$20 Billion to $50 Billion
Newport-Inglewood Fault	7.5	4,000 to 21,000	16,000 to 84,000	$30 Billion to $60 Billion
San Andreas Fault (San Francisco)	8.3	3,000 to 11,000	12,000 to 44,000	$20 Billion to $40 Billion
Hayward Fault	7.5	1,000 to 3,000	4,000 to 10,000	$5 Billion to $20 Billion

1. James M. Gere and Haresh C. Shah, *Terra Non Firma*, Stanford Alumni Association, Stanford, California, 1984.

SELECTION OF ISOLATION AS A DESIGN STRATEGY

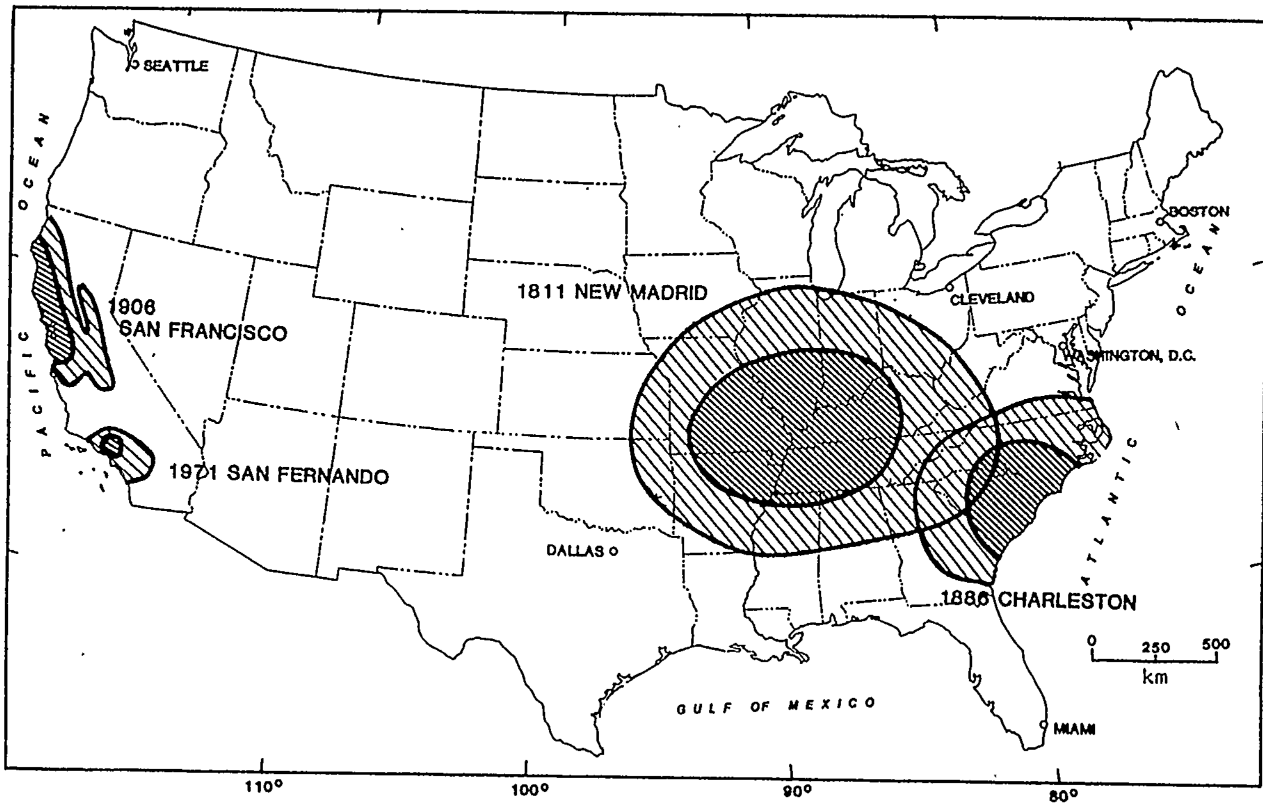
CALIFORNIA EARTHQUAKE LOSS, INSURANCE PREMIUMS AND CLAIMS PAID 1970 - 1990[1]



1. California Department of Insurance, "California Earthquake Zoning and Probable Maximum Loss Evaluation Program," 1990.

SELECTION OF ISOLATION AS A DESIGN STRATEGY

COMPARISON OF MODIFIED MERCALLI INTENSITY DATA FOR FOUR MAJOR EARTHQUAKES[1]



1. S. T. Algermissen, *An Introduction to the Seismicity of the United States*, Earthquake Engineering Research Institute, Berkeley California, 1983 (figure adapted form O. W. Nuttli, "Seismicity of the Central United States," *Geology in Siting of Nuclear Power Plants*, Geological Society of America, Reviews in Engineering Geology, Vol. 4, 1979, pp. 67-94.

SELECTION OF ISOLATION AS A DESIGN STRATEGY

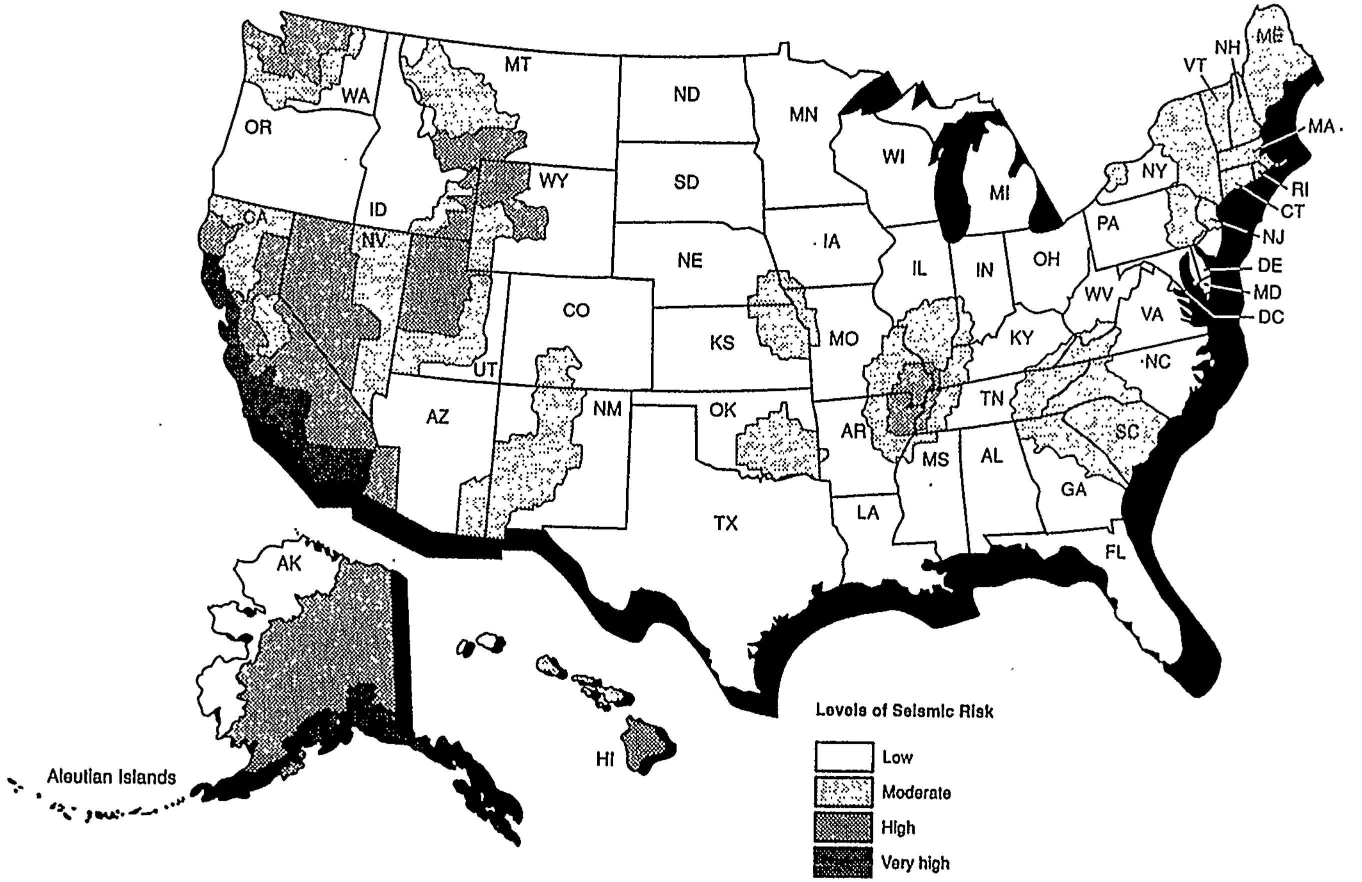
APPROXIMATE RELATIONSHIP BETWEEN RICHTER MAGNITUDE AND MAXIMUM MODIFIED MERCALLI INTENSITY[1]

Richter Magnitude	Maximum Intensity	Typical Effects
3.0	III	Felt indoors by some people; no damage.
4.0	IV - V	Felt by most people; objects disturbed; no structural damage.
5.0	VI - VII	Some structural damage, such as cracks in walls and chimneys.
6.0	VII - VIII	Moderate damage, such as fractures of weak walls and toppled chimneys.
7.0	IX - X	Major damage, such as collapse of weak buildings and cracking of strong buildings.
8.0 and over	XI - XII	Damage total or nearly total.

1. James M. Gere and Haresh C. Shah, *Terra Non Firma*, Stanford Alumni Association, Stanford, California, 1984.

SELECTION OF ISOLATION AS A DESIGN STRATEGY

SEISMIC RISK ZONES IN THE UNITED STATES[1]



1. United States General Accounting Office, "Federal Buildings Many Are Threatened by Earthquakes, but Limited Action Has Been Taken," GAO/GGD-92-62, May 1992 (figure adapted from NEHRP seismic maps, FEMA 222, January 1992.

SELECTION OF ISOLATION AS A DESIGN STRATEGY

FEDERAL BUILDINGS IN SEISMIC RISK ZONES NATIONWIDE[1]

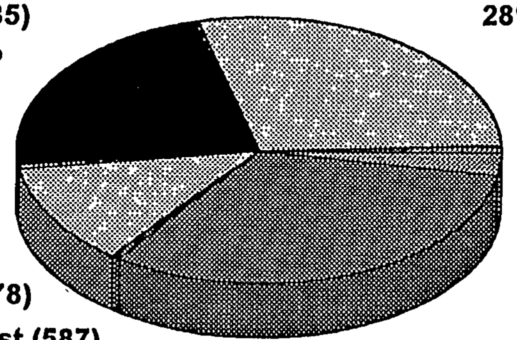
Seismic Risk Zone	Damage Level	Number of Buildings	Gross Square Footage	Number of Employess
Very High	Most Buildings	31,915	224,122,094	218,000
High	Many Buildings	51,879	299,787,070	224,000
Moderate	Some Buildings	99,082	652,074,523	668,000
Low	No Buildings	234,416	1,584,735,459	1,759,000
All		417,292	2,760,719,146	2,869,000

1. United States General Accounting Office, "Federal Buildings Many Are Threatened by Earthquakes, but Limited Action Has Been Taken," GAO/GGD-92-62, May 1992.

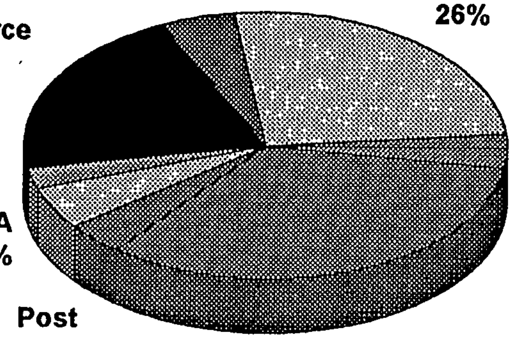
SELECTION OF ISOLATION AS A DESIGN STRATEGY

FEDERAL AGENCIES WITH BUILDINGS LOCATED IN HIGH OR VERY HIGH SEISMIC RISK ZONES[1]

Number of Buildings (83,794)



Building Space (523,909,164 sq. ft.)



1. United States General Accounting Office, "Federal Buildings Many Are Threatened by Earthquakes, but Limited Action Has Been Taken," GAO/GGD-92-62, May 1992.

SELECTION OF ISOLATION AS A DESIGN STRATEGY

IMPEDIMENTS TO ISOLATION

- ACCEPTANCE AND EXPERIENCE OF THE ENGINEERING PROFESSION
  - Engineers do not understood or are slow to accept the isolation concept
  - Engineers are not familiar with isolation system products and do not have confidence in these products or their suppliers
  - Engineers are not experienced with design methods, analysis techniques and regulations (i.e., 1991 UBC) governing design of isolated structures
  - Engineers are not experienced with design of isolation systems and specifcation of isolation system products
- ISOLATION SYSTEM PRODUCTS AND SUPPLIERS
  - Overly aggressive competition among isolation system suppliers inhibits advancement of the industry as a whole
  - Industry has not developed a standard set of specifications governing materials, fabrication and quality of seismic isolators
  - Most isolation system products are not available as a "standard" part, but must be designed and specified uniquely for each project
  - Non-standard, proprietary products pose special problems for applications requiring competitive bidding

SELECTION OF ISOLATION AS A DESIGN STRATEGY

IMPEDIMENTS TO ISOLATION

- EARTHQUAKE EXPERIENCE
  - Isolated structures have yet to experience very strong ground shaking and demonstrate superior earthquake performance
  - Some isolated structures have not performed as well as expected during low or moderate levels of ground shaking
- REGULATORY REVIEW AND APPROVAL
  - Isolated buildings require special design review by the regulatory agency that can effect project schedule
- EARTHQUAKE PERFORMANCE AND COST
  - In general, the initial cost of construction is more for a structure with an isolation system than for the same structure with a conventional fixed base, designed to meet only minimum "code" requirements
  - Life-cycle cost studies must be used to compare costs of fixed-base and isolated design alternatives, unless the two concepts are developed to meet identical earthquake performance criteria

SELECTION OF ISOLATION AS A DESIGN STRATEGY

COST FACTORS FOR COMPARISON OF ISOLATED AND CONVENTIONAL (FIXED-BASE) CONSTRUCTION

Cost Increase of Isolation	Cost Reduction of Isolation
Initial Construction	**Initial Construction**
1. Seismic Isolation System	1. Reduction in Size of Superstructure
2. Additional Structure and Architectural Details to Accommodate the Seismic Isolation System	2. Reduction in Size of Architectural Components/Details Governed by Drift (e.g., Curtain Wall Details)
3. Special Mechanical/Electrical Connection Details	3. Reduction in Size and Extent of Nonstructural Component Anchorage
4. Additional Design Costs	
Life of Structure	**Life of Structure**
1. Periodic Inspection of Isolators	1. Reduction in Direct Earthquake Loss (i.e., Structural, Nonstructural and Contents Damage)
2. Replacement of Isolators (after a Major Earthquake or Design Life of Isolators)	2. Reduction in Indirect Loss (i.e., Business Interruption, Relocation Costs, etc.)
	3. Reduction in Earthquake Protection Costs (i.e., Insurance Premiums, Component Hardening, Contingency Planning, etc.)

SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY[1]

- PURPOSE OF STUDY
  - Develop a methodolgy for evaluating life-cycle cost of seismic retrofit and renovation of the State of California Building (SCJB)
  - Calculate and compare life-cycle costs for three design alternatives:
    - Base Isolated Scheme (i.e., base isolattion retrofit scheme that meets project-specifc performance criteria, including the goal of repairable damage for a major earthquake)
    - Repairable Fixed-Base Scheme (i.e., fixed-base retrofit scheme that meets, or almost meets, the same project-specifc performance criteria as the Base Isolation Scheme)
    - Life-Safety Fixed-Base Scheme (i.e., fixed-base retrofit scheme that that provides substantial life-safety, but may not be repairable following a major earthquake)
  - Evaluate sensitivity of life-cycle cost to variation of study parameters including interest and discount rates and other key economic factors

1. Rutherford & Chekene, "State of California Justice Building Life-Cycle Study," August 1992.

SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY

Performance Objectives - Base Isolated and Repairable Fixed-Base Schemes

Performance Objective	Probable Earthquake (50% in 50 years)	Design Basis Earthquake (10% in 50 years)	Maximum Credible Earthquake (M 8.0, San Andreas)
Life-Safety Protection	FULL	FULL	SUBSTANTIAL, WILL NOT SIGNIFICANTLY JEOPARDIZE LIFE
Functionality	IMMEDIATE OCCUPANCY; POST-EARTHQUAKE FUNCTION RESUMED WITHIN HOURS	POST EARTHQUAKE FUNCTION RESUMED WITHIN DAYS TO MONTHS	BUILDING MAY BE CLOSED INDEFINITELY
Damage Protection	NO STRUCTURAL DAMAGE, SOME NONSTRUCTURAL CLEAN-UP REQUIRED	REPAIRABLE DAMAGE, MINOR STRUCTURAL AND NONSTRUCTURAL DAMAGE	SIGNIFICANT STRUCTURAL AND NONSTRUCTURAL DAMAGE, MAY NOT BE REPAIRABLE

SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY

Performance Objectives - Life-Safety Fixed-Base Schemes

Performance Objective	Probable Earthquake (50% in 50 years)	Design Basis Earthquake (10% in 50 years)	Maximum Credible Earthquake (M 8.0, San Andreas)
Life-Safety Protection	FULL	SUBSTANTIAL, WILL NOT SIGNIFICANTLY JEOPARDIZE LIFE	
Functionality	POST EARTHQUAKE FUNCTION RESUMED WITHIN DAYS TO MONTHS	BUILDING MAY BE CLOSED INDEFINITELY	
Damage Protection	REPAIRABLE DAMAGE, MINOR STRUCTURAL AND NONSTRUCTURAL DAMAGE	SIGNIFICANT STRUCTURAL AND NONSTRUCTURAL DAMAGE, MAY NOT BE REPAIRABLE	

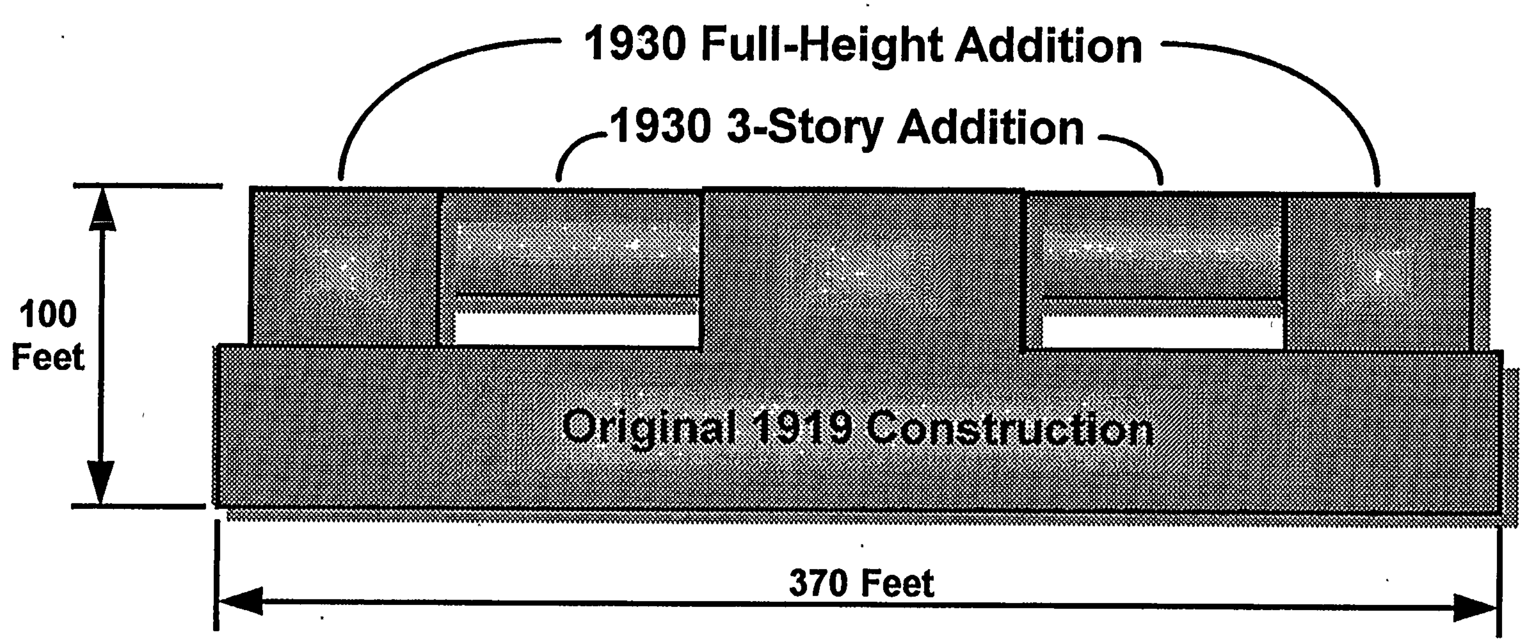
SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY

- BACKGROUND
  - The SCJB provides office and support space for the State Supreme Court, the First District Court of Appeal, State Police and State officials
  - The SCJB was closed following the Loma Prieta earthquake of October 1989 and will be fully renovated before being returned to service
  - In accordance with Senate Bill 920 (Rogers, Seismic Safety Building Design), the State Architect selected the SCJB to be the first state-owned building to be seismic retrofitted with base isolation technology
  - Preliminary design is complete, prospective isolation system suppliers have been evaluated and ranked; final design is pending notice to proceed by the State
- DESCRIPTION OF BUILDING
  - The building, located at 350 McAllister Street, is part of the Civic Center district of San Francisco and is of historical significance
  - The building was designed in 1915 by the architectural firm of Bliss and Faville and constructed in 1919 - 1920
  - In 1930, the building was enlarged by the addition of two full-height segments at each end of the building, making the building E-shaped in plan, and by partial 3-story infills between the legs of the "E"

SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY



PLAN VIEW - STATE OF CALIFORNIA JUSTICE BUILDING

SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY

- ♦ DESCRIPTION OF STRUCTURE
  - Structure is five stories in height with a full basement, approximately 100 feet by 370 feet in plan
  - Structural system consists of a vertical-load-supporting steel frame with unreinforced-masonry and concrete infill walls that resist lateral force
- ♦ DESCRIPTION OF RENOVATION WORK
  - Partial, 3-story segments of building will be demolished and full-height additions constructed to form an essentially rectangular building in plan
  - HVAC, fire and other nonstructural systems will be fully upgraded
- ♦ DESCRIPTION OF SEISMIC RETROFIT
  - Existing foundations will be strengthened and new retaining walls constructed around the building's basement
  - Existing basement columns and walls will be cut and seismic isolators will be installed just above the foundation
  - Existing basement elements will be strengthened and a new basement floor will be constructed
  - Superstructure of building will be strengthened by the addition of new reinforced-concrete shear walls at perimeter and interior locations

SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY

- APPROACH OF STUDY - LIFE-CYCLE COSTS
  - Initial construction costs included in the study:
    - General renovation costs (i.e., building enlargement, new HVAC, electrical, telephone, security, fire and life-safety systems)
    - Seismic retrofit costs
  - Earthquake losses included in the study:
    - Direct earthquake loss (i.e., structural, nonstructural and contents damage)
    - Indirect earthquake loss (i.e., loss of productivity, temporary space rental and relocation costs)
  - Earthquake losses and cost factors not included in the study:
    - Life safety (i.e., death and injuries)
    - Facility function (other than the cost of productivity loss)
    - Damage to historical features of building (beyond $10M included in the replacement value of the building for historical architecture)
    - Earthquake insurance

SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY

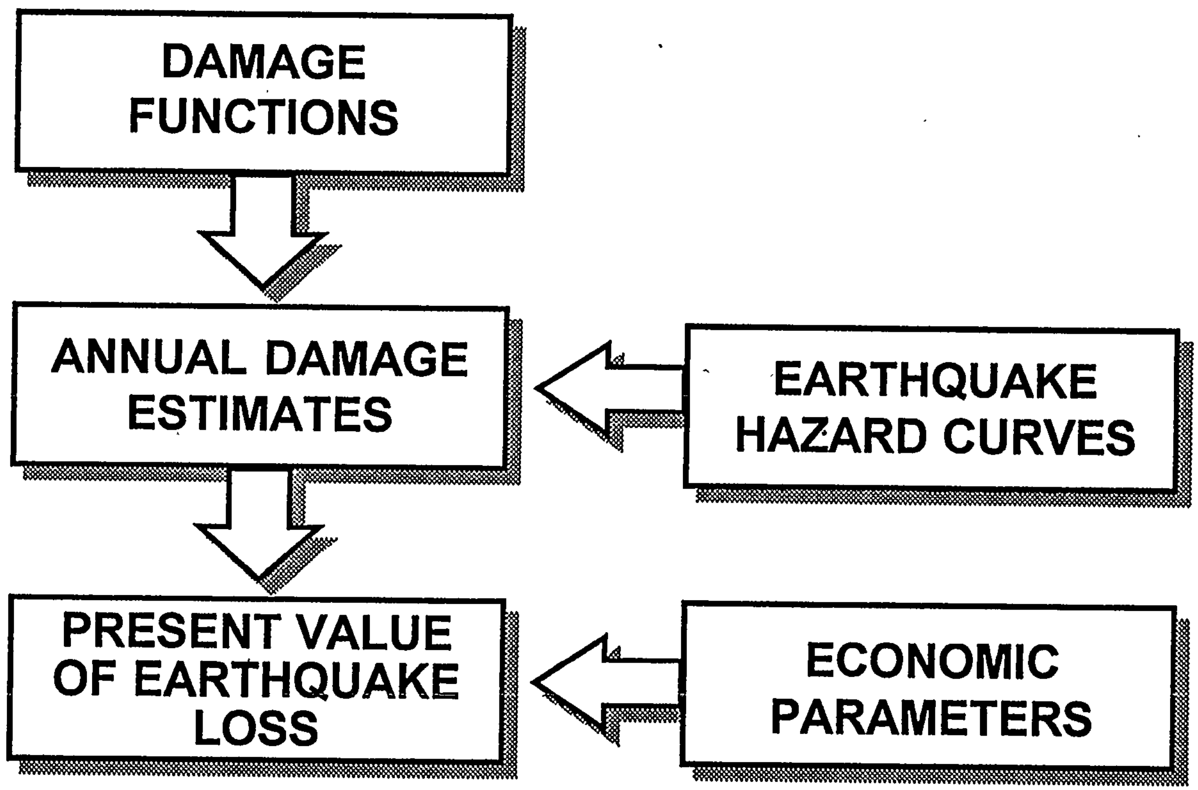
♦ APPROACH OF STUDY - METHODS AND ASSUMPTIONS

- Calculate earthquake loss from two perspectives:
  - Best estimate (i.e., loss calculated by integrating mean estimates of conditional earthquake loss over all hazard levels)
  - worst case estimate (i.e., loss calculated by integrating bounding estimates of conditional earthquake loss over all hazard levels; bounding estimates of loss taken as mean loss shifted one MMI unit)
- Evaluate sensitivity of life-cycle costs to economic parameters including:
  - inflation and discount rate (i.e., evaluate life-cycle cost for net, inflation less discount, rates of 1.5%, 3% and 4.5%)
  - economic life (i.e., evaluate life-cycle cost for economic lives of 30, 50 and 100 years)
  - cost of lost productivity (i.e., evaluate life-cycle cost for lost productivity costs of $85K/day, $170K and $340/day)
- Assume present value of building after renovation is $60M for all life-cycle evaluations (i.e., $45M for replacement plus $10M for historical architecture plus $5M for temporary space and moving costs)

SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY

Earthquake Loss Evaluation Process

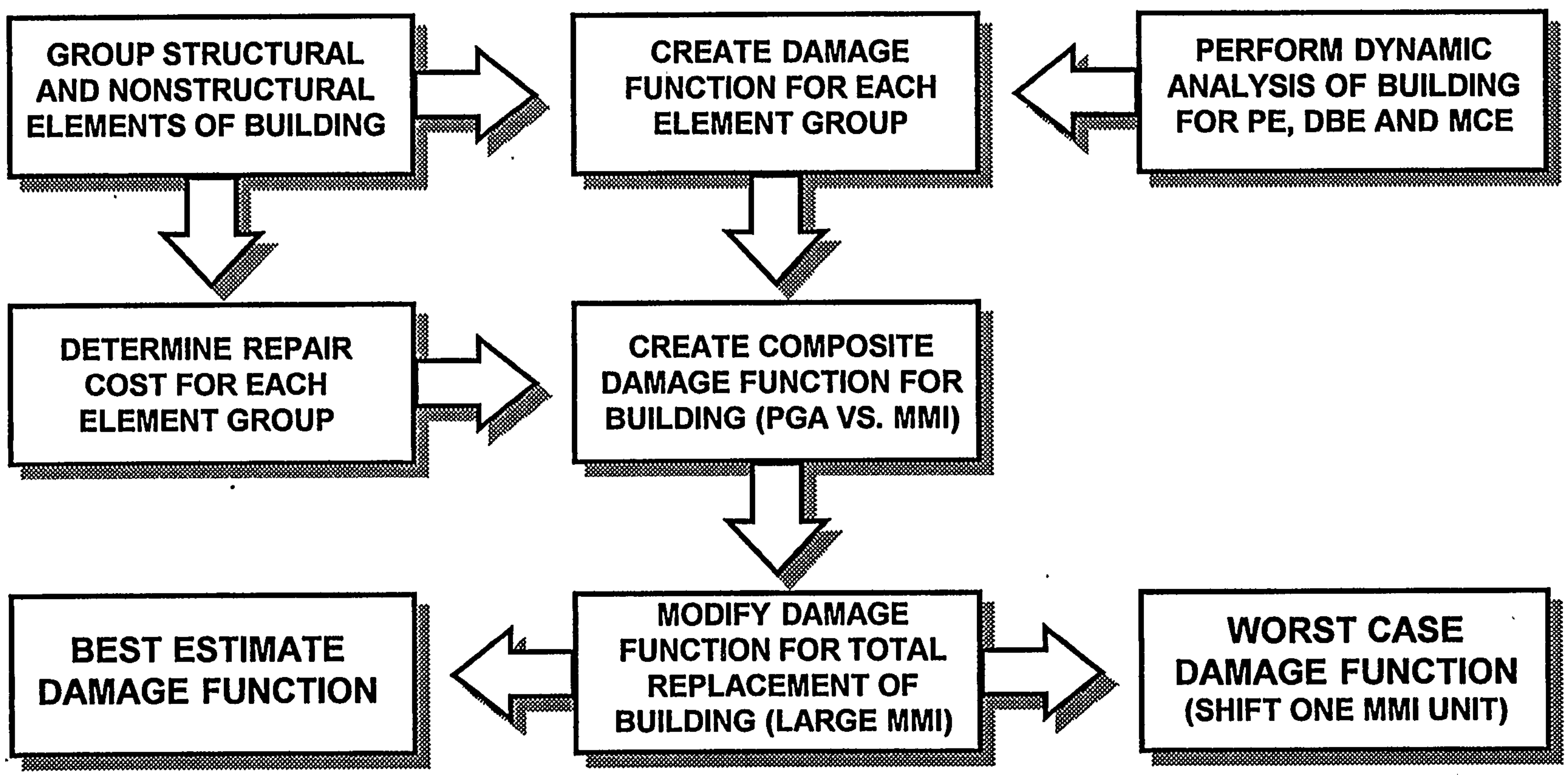


Charles Kircher & Associates
Consulting Engineers

SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY

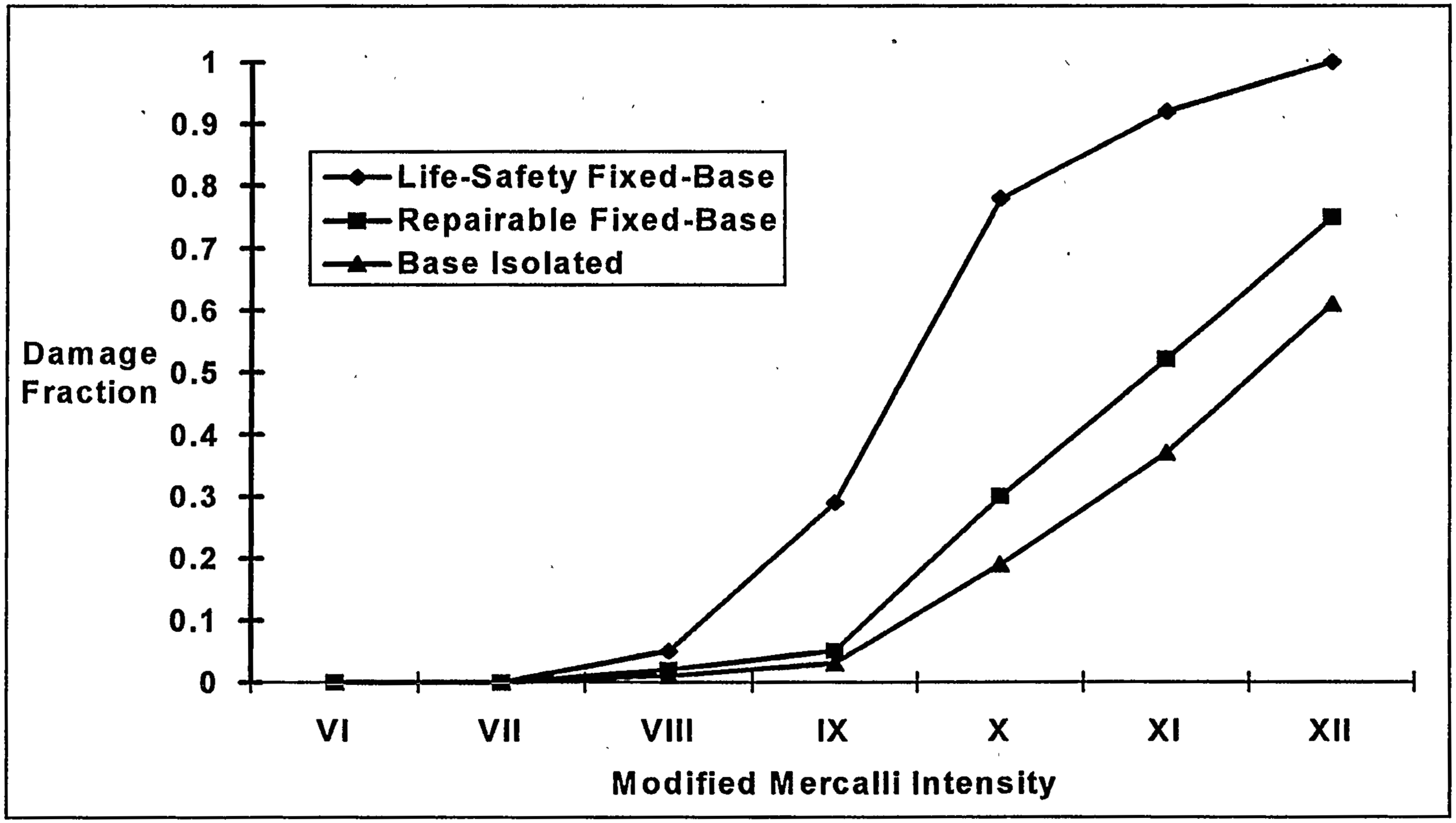
Building Damage Function Calculation



SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY

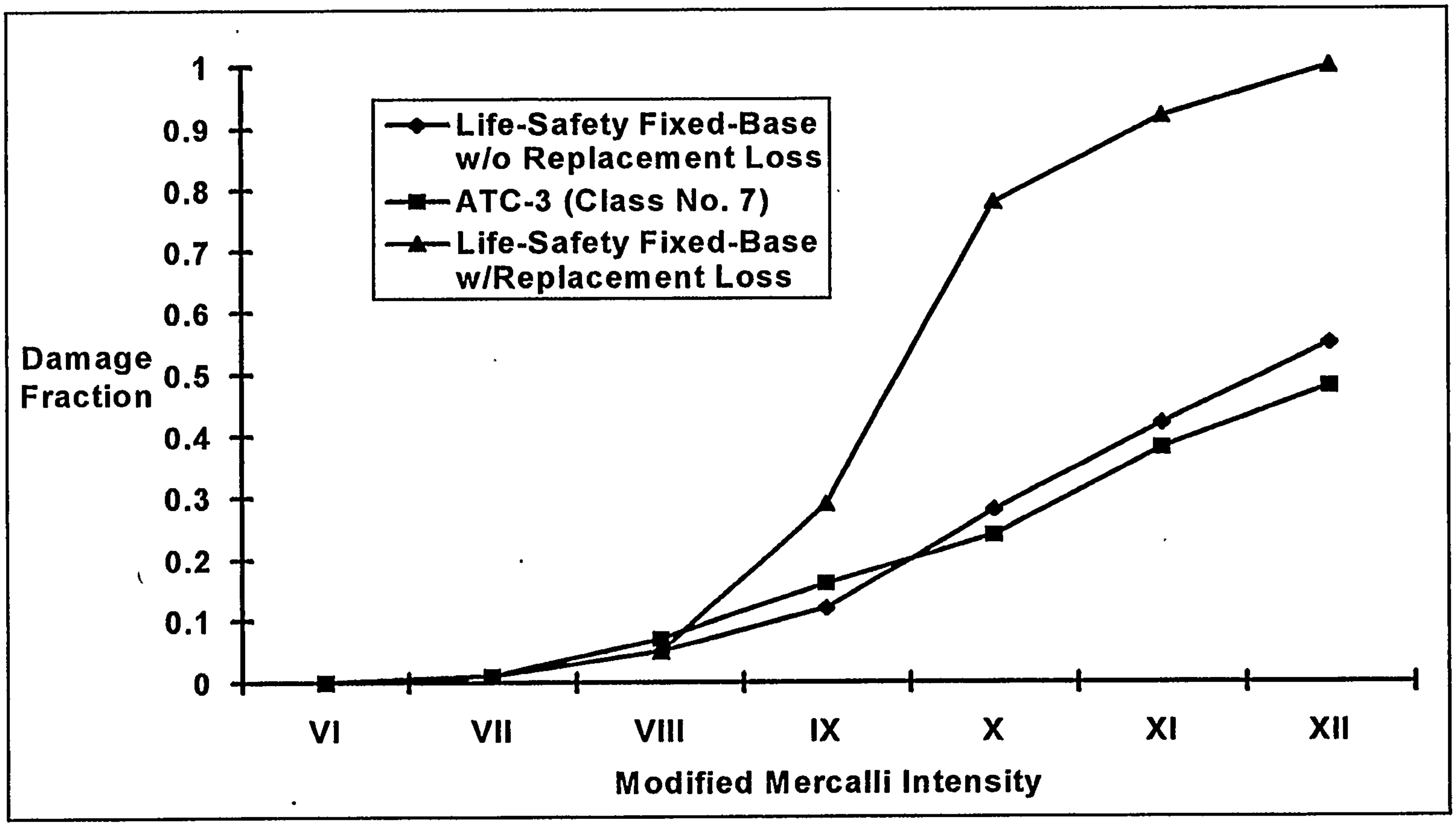
Building Damage Functions - Best Estimate



SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY

Comparison of Building Damage Functions: ATC-3 (Class No. 7) and Life-Safety Fixed-Base



SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY

$$\text{Annual Building Damage} = \sum_{i=1}^{7} f[\overline{D}|MMI_i]P[MMI_i]$$

where: $f[\overline{D}|MMI_i]$ = Building Damage Function

$P[MMI_i]$ = Annual Discrete Earthquake Hazard

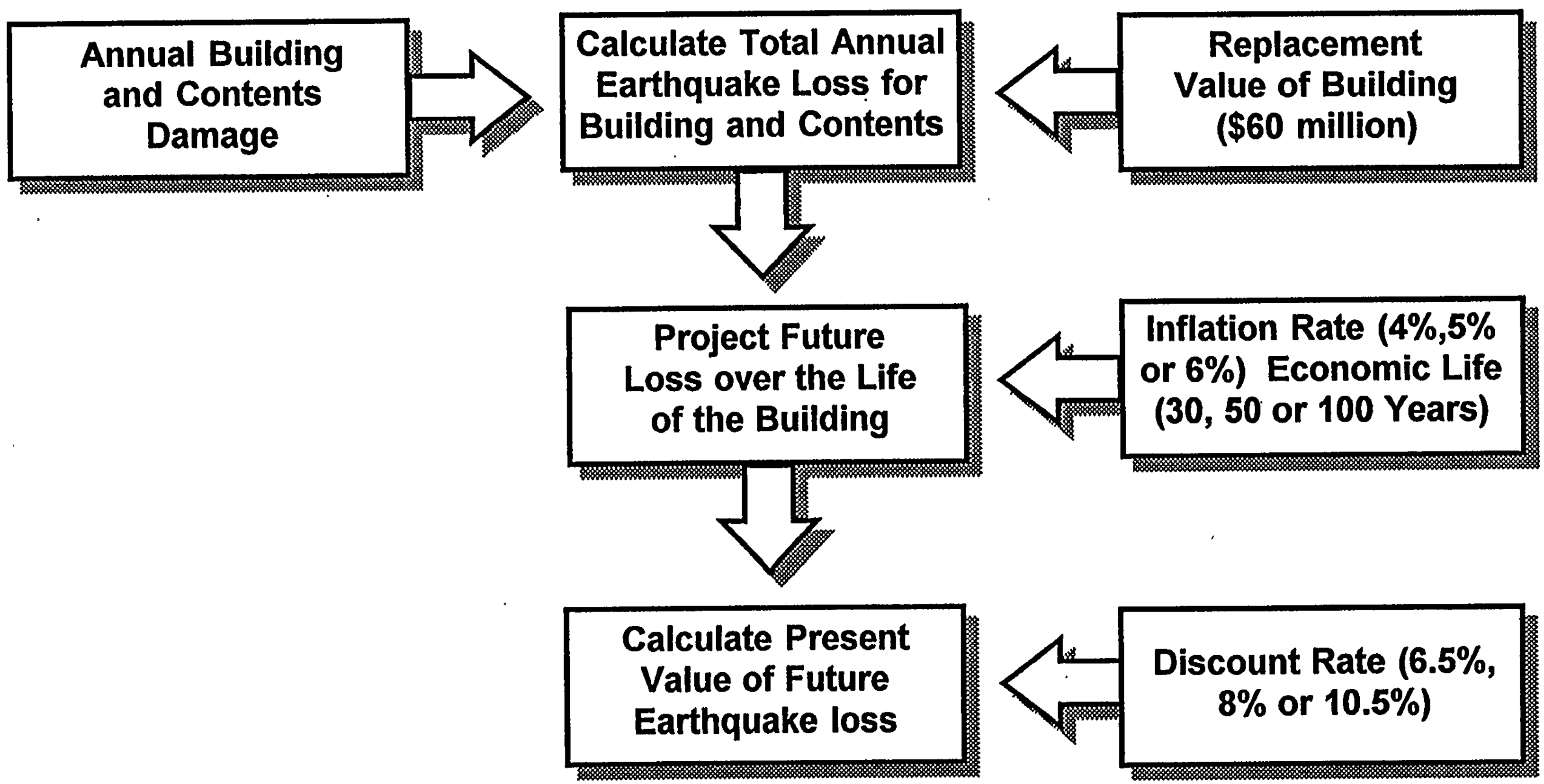
Annual Discrete Earthquake Hazard

MMI	VI	VII	VIII	IX	X	XI	XII
PGA	0.05g	0.11g	0.22g	0.44g	0.70g	0.90g	
$P[MMI_i]$	0.0913	0.0250	0.0105	0.0021	0.0003	0.0002	0.0001

SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY

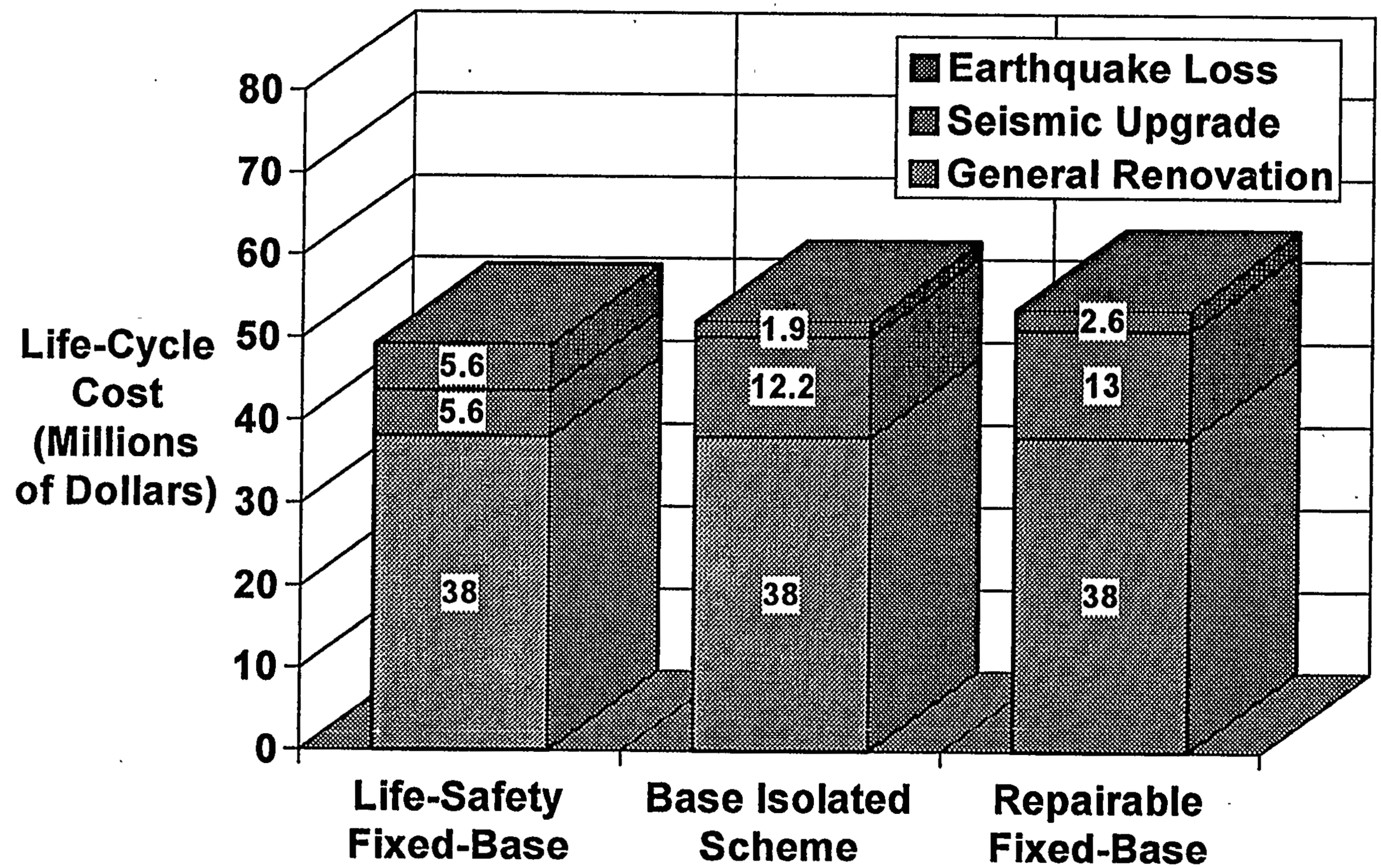
Present Value Calculation of Direct Earthquake Loss



SELECTION OF ISOLATION AS A DESIGN STRATEGY

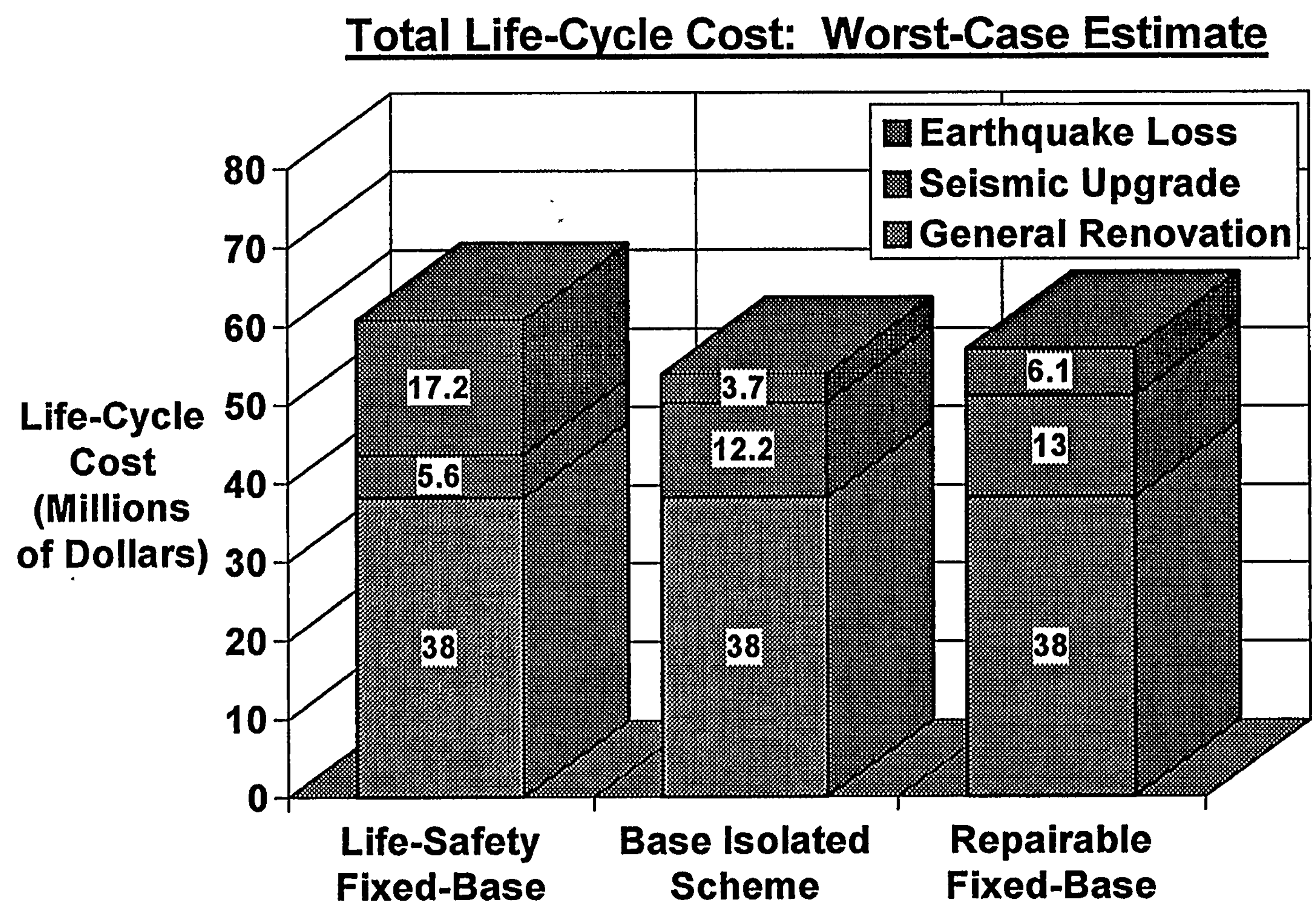
STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY

Total Life-Cycle Cost: Best Estimate



SELECTION OF ISOLATION AS A DESIGN STRATEGY

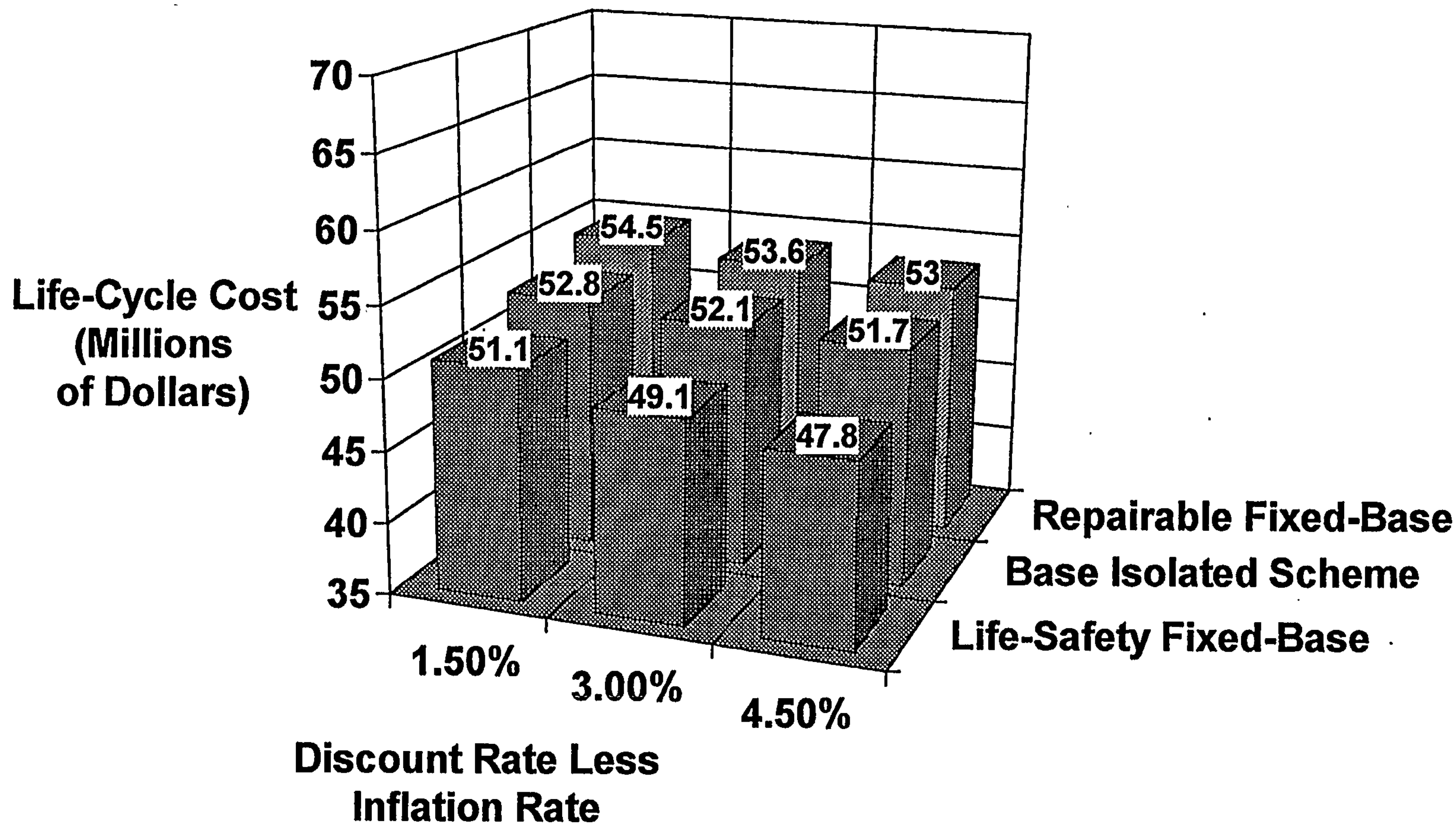
STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY



SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY

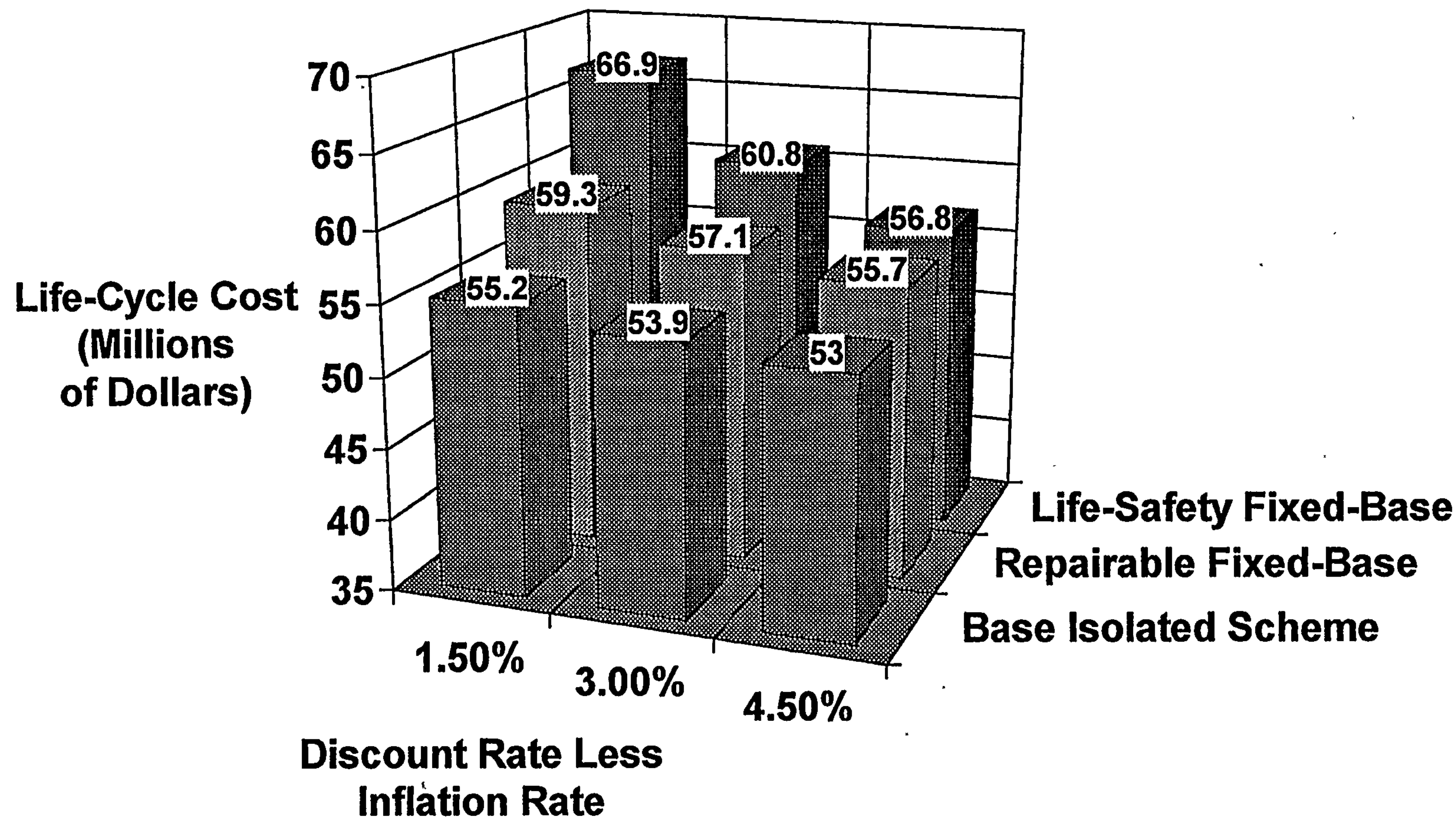
Life-Cycle Cost Sensitivity to Interest Rates: Best Estimate



SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY

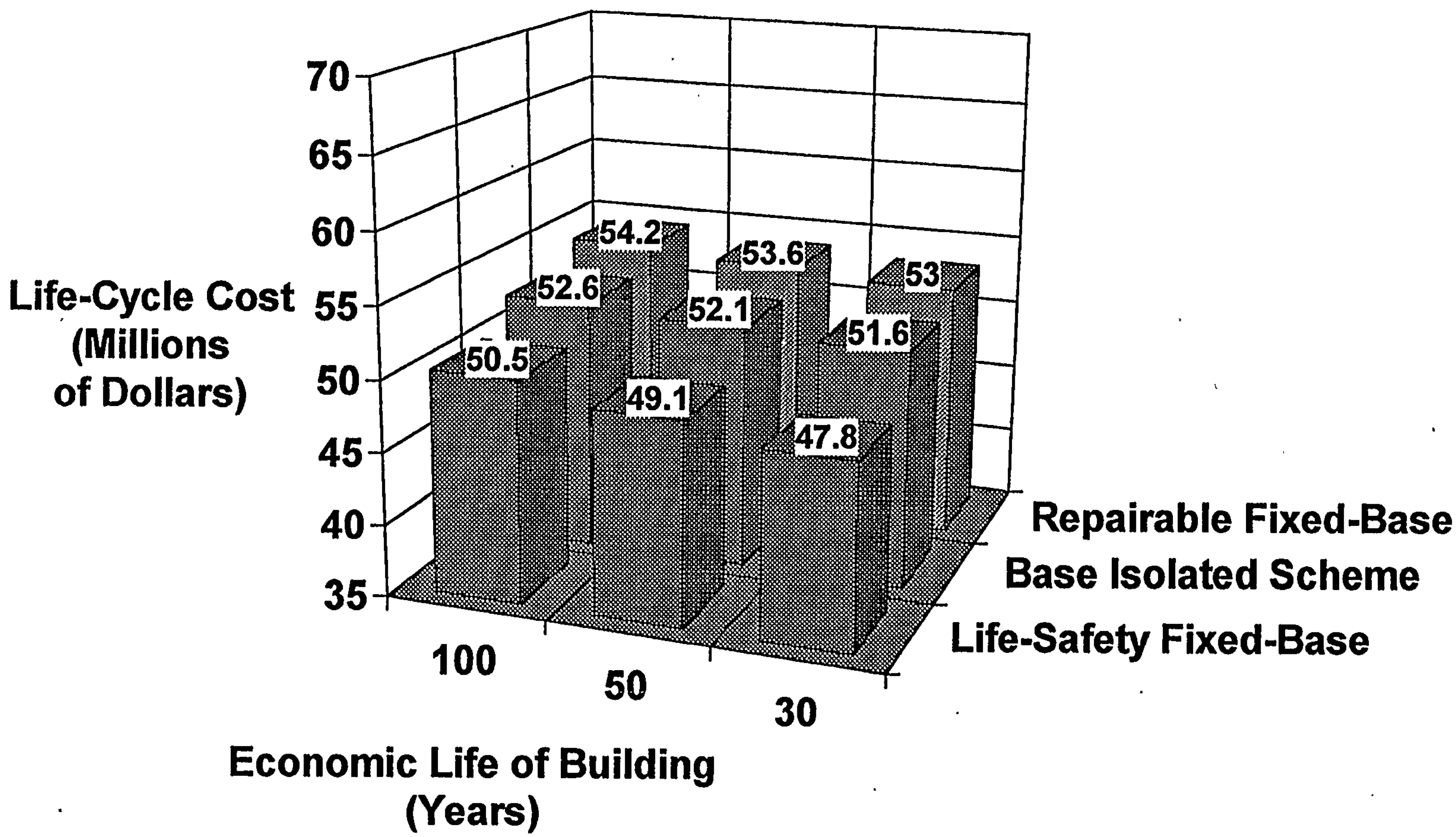
Life-Cycle Cost Sensitivity to Interest Rates: Worst-Case Estimate



SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY

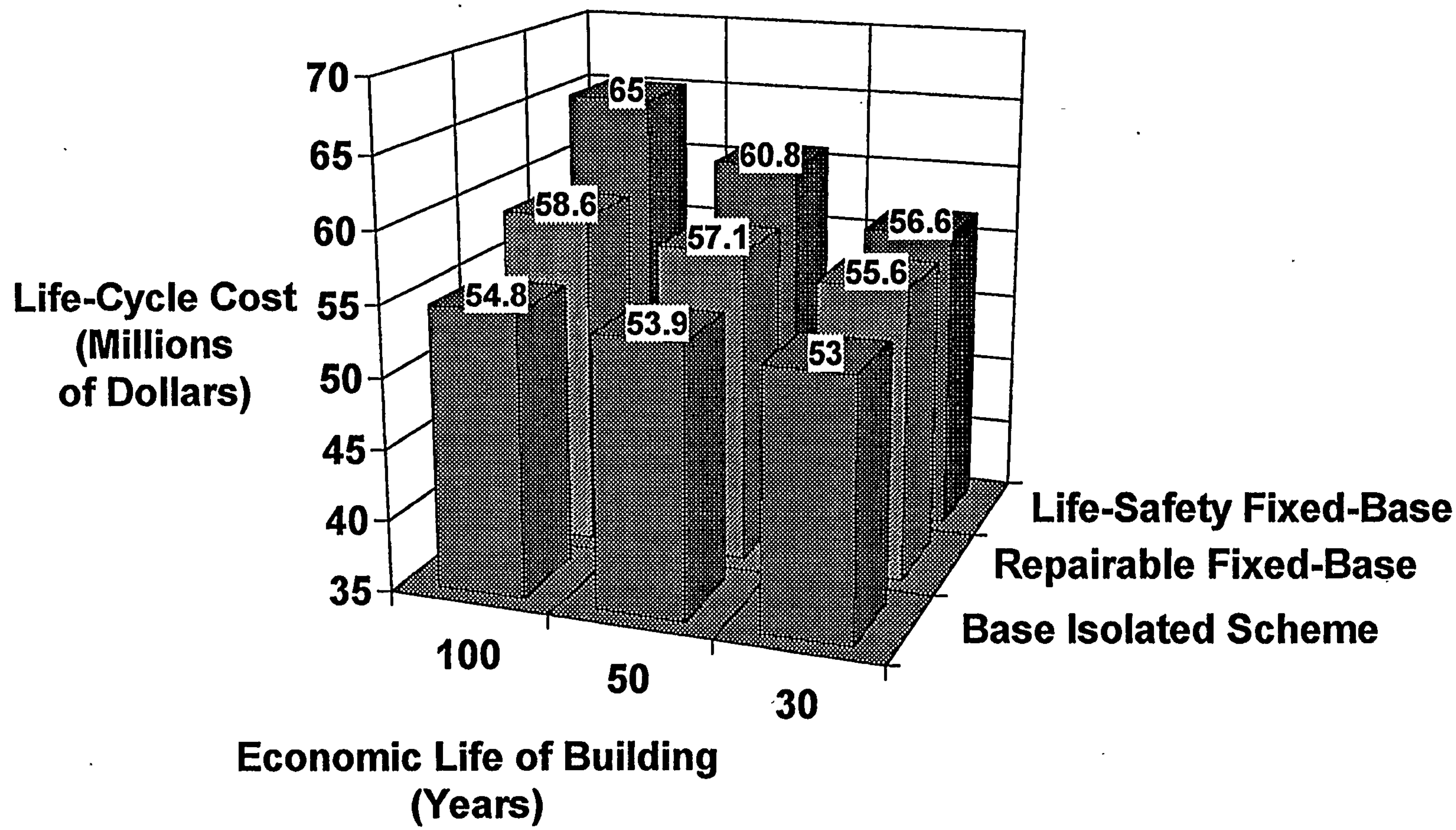
Life-Cycle Cost Sensitivity to Economic Life: Best Estimate



SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY

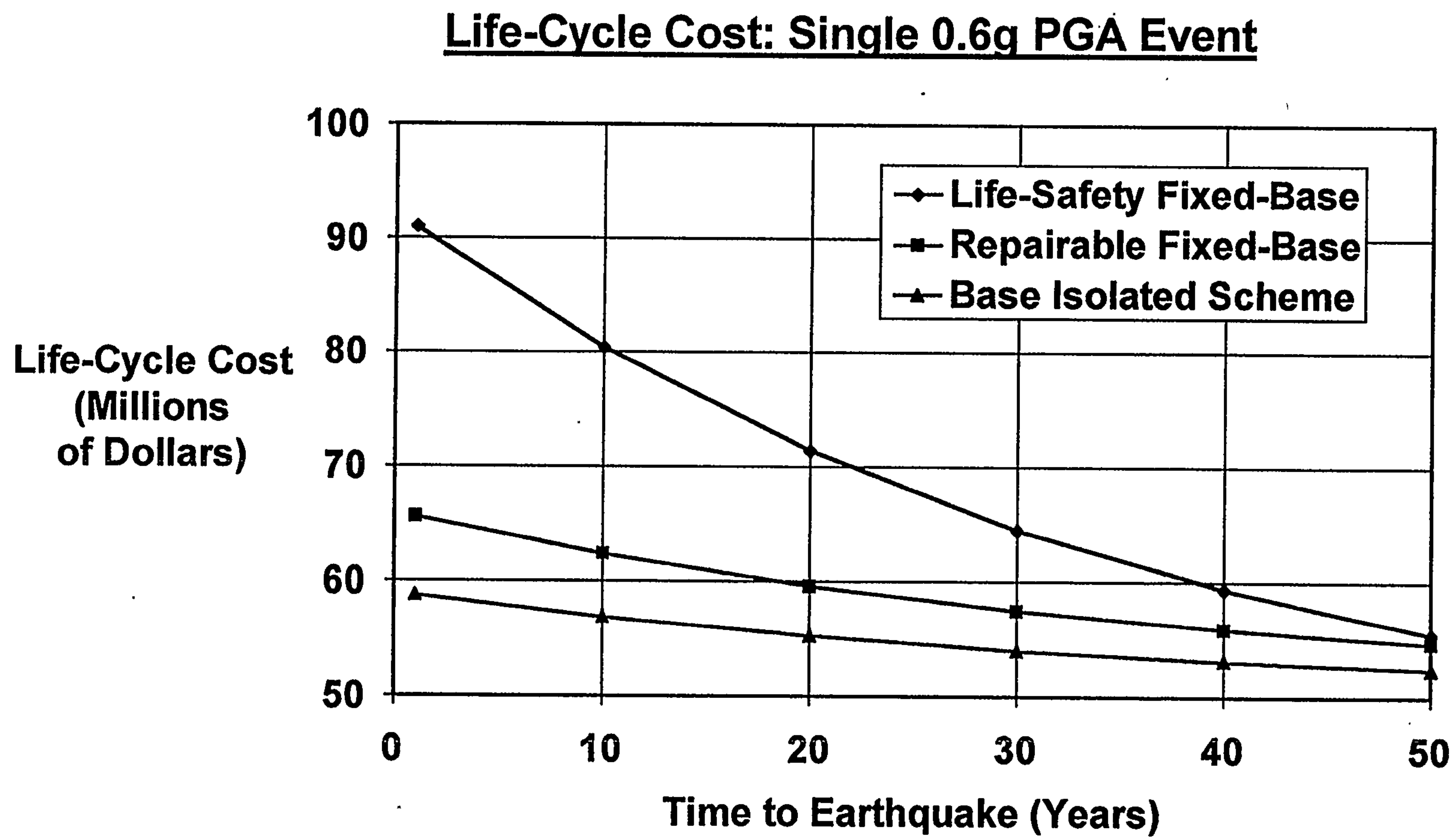
Life-Cycle Cost Sensitivity to Economic Life: Worst-Case Estimate



Charles Kircher & Associates
Consulting Engineers

SELECTION OF ISOLATION AS A DESIGN STRATEGY

STATE OF CALIFORNIA JUSTICE BUILDING LIFE-CYCLE COST STUDY



SELECTION OF ISOLATION AS A DESIGN STRATEGY

KAISER PERMANENTE BASE ISOLATION SPECIAL STUDY[1]

- PURPOSE OF STUDY
  - Investigate isolation for a new East Bay hospital building considering three design alternatives:
    - Moment frame with conventional base (MC)
    - Braced frame with conventional base (BC)
    - Braced frame with base isolation (BI)
  - Investigate isolation for other hospitals in the San Francisco Bay Area
- APPROACH OF STUDY
  - Study addressed three issues critical to the value of isolation to management:
    - Financial risk aversion
    - Member Inconvenience
    - Government approvals

1. Benjamin Wong, "Base Isolation Special Study," Facilities Design and Construction Kaiser Permanente, Oakland, California, March 1990.

SELECTION OF ISOLATION AS A DESIGN STRATEGY

KAISER PERMANENTE BASE ISOLATION SPECIAL STUDY

- APPROACH OF STUDY (Continued)
  - A mathematical model was developed that calulated values of operational and economic indicators:
    - Equivalent days of lost function - the percent of the structure not functioning multiplied by the number of days of lost function
    - Discounted gross (with insurance) earthquake cost - the discounted costs due to structural and content damage and liost function
    - Discounted net (with insurance) earthquake costs - the gross earthquake costs minus the estimated insurance
  - Two important types of quantitative results were developed:
    - Expected value (EV) - the sum of a range of outcomes, such as days lost or costs, weighted by each outcome's probability
    - Measures of risk - such as the probabilities of extremely bad outcomes
  - Additionally, four subjective issues were also considered: service disruption, archtectural design, delays in design approvals and image

SELECTION OF ISOLATION AS A DESIGN STRATEGY

KAISER PERMANENTE BASE ISOLATION SPECIAL STUDY

- ASSUMPTIONS AND VALUES OF KEY PARAMETERS
  - The (net) discount rate used was taken as 5 percent, and building lives of 33 and 50 years were considered
  - The total cost of the building was estimated to be $85.8 million, and the cost per day of lost function was estimated at $135,000
  - The site is located about 10 miles from the Hayward fault and about 0.6 miles from the Franklin fault, and is also
  - Probabilities of occurrence in 33 and 50-year periods were developed for median peak ground acelerations and equivalent MM intensities; in a 50-year time period the hazard level at the site was characterized by:
    - 60 percent probability of exceeding 0.2g PGA, taken as MMI IX
    - 20 percent probability of exceeding 0.4g PGA, taken as MMI X
    - 2 percent probability of exceeding 0.6g PGA, taken as MMI XI
  - Damage estimates were based on ATC-13, *Earthquake Damage Evaluation Data for California*, Applied Technology Council, 1985, and the "Navy method" developed by John Ferritto, "Economic of Seismic Design for New Buildings," ASCE Journal of Structural Engineering, Volume 110, No. 12, December 1984.

SELECTION OF ISOLATION AS A DESIGN STRATEGY

KAISER PERMANENTE BASE ISOLATION SPECIAL STUDY

Operational or Economic Paramter	Conventional Moment Frame	Conventional Braced Frame	Base Isolated Braced Frame
1. Expected value (EV) of equivalent days of lost function	6.0 days	4.8 days	0.1 days
2. Probability of equivalent days of lost function exceeding 10 days	18%	11%	<0.1%
3. Relative construction cost (in millions)	$1.1M	$0.0M	$3.6M
4. EV of earthquake cost - without insurance (in millions)	$5.0M	$3.5M	$0.1M
5. Probability of earthquake loss - without insurance exceeding $10M	20%	10%	<0.1%
6. Sum of construction and earthquake cost - without insurance	$6.1M	$3.5M	$3.7M
7. EV of earthquake cost - with insurance (in millions)	$3.0M	$2.1M	$0.0M
8. Probability of earthquake loss - with insurance exceeding $10M	15%	10%	<0.1%
9. Sum of construction and earthquake cost - with insurance	$4.1M	$2.1M	$3.6M

SELECTION OF ISOLATION AS A DESIGN STRATEGY

KAISER PERMANENTE BASE ISOLATION SPECIAL STUDY

- FINDINGS AND CONCLUSIONS
  - Financial Risk Aversion
    - Base isolation would not be the preferred design scheme when evaluated using expected value (EV) measures of loss; the life-cycle cost of the base isolation braced frame scheme is slightly less than the conventional moment frame scheme, but slightly more than the conventional braced frame scheme
    - Base isolation would be the preferred design scheme to avoid the chance of a very bad outcome; the probability of earthquake loss in excess of $10M is less than 0.1% for the base isolated brace frame scheme, but is 15% and 10%, respectively, for the conventional moment and braced frame schemes, even if additional protection is provided by earthquake insurance
  - Member Inconvenience
    - Base isolation would be the preferred scheme to avoid expected loss of hospital function; the expected value (EV) of time of lost function is measured in a few hours for the base isolated braced frame scheme, but is 6.0 days and 4.8 days, respectively, for the conventional moment and braced frame schemes

SELECTION OF ISOLATION AS A DESIGN STRATEGY

KAISER PERMANENTE BASE ISOLATION SPECIAL STUDY

- FINDINGS AND CONCLUSIONS
  - Member Inconvenience (continued)
    - Base isolation would be the preferred scheme to avoid extended closure of the hospital; the probability of lost hospital function exceeding beyond 10 days is less than 0.1% for the base isolated braced frame scheme, but is 18% and 11%, respectively, for the conventional moment and braced frame schemes
  - Government Approvals
    - All schemes meet California State codes now and in the foreseeable future
    - State approvals for the base isolation scheme may take 10 - 15 weeks longer than either of the conventional schemes
  - Subjective Issues
    - Kaiser's current standards call for moment frame construction to provide greater freedom for initial design, alterations and aesthetics, although an acceptable braced frame design would be permitted
    - Base isolation would enhance Kaiser's image as a company that takes extraordinary measures to protect its employees, members and community during an earthquake

SELECTION OF ISOLATION AS A DESIGN STRATEGY

SUMMARY AND CONCLUSION - LIFE-CYCLE STUDIES

- SUMMARY
  - Renovation of the SCJB and construction of a new Kaiser medical facility are very different projects, but certain life-cycle parameters are similar:
    - Both buildings are located at sites with very high seismic hazard
    - Both buildings are worth a great deal: $60M for the SCJB and $86M for Kaiser facility
  - The estimated additional construction cost of isolation varies from about 2.5% of total construction cost for the new Kaiser facility to about 13% for the SCJB (costs are significantly higher for the SCJB due to the special difficulties of isolating the existing building below the basement level)
  - The cost benefits of damage and loss reduction (as compared to conventional fixed-base construction) are significantly greater for the existing SCJB than for the new Kaiser facility (since new fixed-base construction is expected to perform better than fixed-base retrofit construction

<u>SELECTION OF ISOLATION AS A DESIGN STRATEGY</u>

SUMMARY AND CONCLUSION - LIFE-CYCLE STUDIES

- **SUMMARY (continued)**
  - Life-cycle cost differences between alternatives are relatively small (considering the total value of the buildings) when expected value or best estimates of earthquake loss are used to evaluate cost
  - Life-cycle cost differences between alternatives can be significant when "worst-case" estimates of loss or earthquake scenario are evaluated
- **CONCLUSION**
  - Life-cycle cost results are sensitive to parameters (e.g., estimated earthquake damage and economic forecast factors) that are not well known and results may not clearly suggest one alternative over another
  - Life-cycle cost results are sensitive to the risk perspective of the decision maker:
    - Results based on expected value or best estimates of loss slightly favor minimizing initial construction cost (i.e., fixed-base construction)
    - Results based on worst-case estimates of loss indicate a strong preference for isolation for decision makers who are adverse to risk

Charles Kircher & Associates
Consulting Engineers

LLNL/DOE SHORT COURSE OF SEISMIC (BASE) ISOLATION

DESIGN CONSIDERATIONS FOR ISOLATED STRUCTURES

BY

Charles A. Kircher Ph.D., P.E.

August 11, 1992

PRESENTATION TOPICS

Design Considerations for Isolated Structures

SEISMIC INPUT ISSUES

- ♦ GROUND MOTION CHARACTERIZATION
  - Ground motion characterization requires special consideration of site soil conditions and other factors that influence long-period motion (i.e., the periods that most influence response of an isolated-structure)
  - Ground motion characterization requires special consideration of near-field effects (i.e., directivity, fling, etc.) for sites close to active faults
  - Vertical ground motion input is required for isolation systems that provide vertical, as well as, horizontal isolation
- ♦ EARTHQUAKE TIME HISTORIES
  - Time histories should have frequency content, phasing and duration that represent site and source conditions (e.g., Fourier phases that are based on records of representative events)
  - Directional components of each time history should have a degree of correlation at long periods that represents site and source conditions (i.e, ground motion can be highly correlated at long periods)
- ♦ SOIL STRUCTURE INTERACTION
  - Soil structure interaction effects typically do not require special consideration for isolated structures

<u>Design Considerations for Isolated Structures</u>

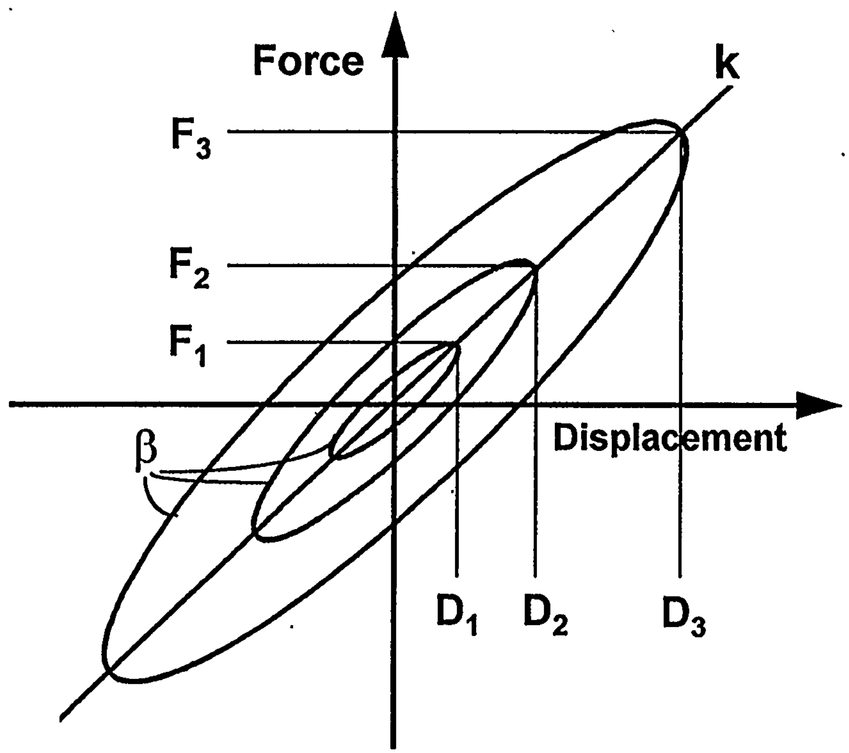
MODELING AND ANALYSIS ISSUES

- **ISOLATED STRUCTURE RESPONSE**
  - In general, isolated structure response can be calculated using linear-elastic modeling of the superstructure elements
  - P-Δ effects due to isolation system displacement require special modeling or a separate calculation
  - Higher-mode response of the superstructure may be underpredicted for nonlinear isolation systems, unless such systems are explicitly modeled with nonlinear elements
- **ISOLATION SYSTEM MODELING**
  - In general, isolators should be modeled as nonlinear elements and response evaluated using time history analysis (e.g., DRAIN-2DX, 3D-BASIS, etc.), exceptions:
    - isolation systems dominated by viscous, rather than hysteretic damping
    - isolation systems with effective damping less than 10% - 20% of critical
  - Linear elastic models can be used to estimate peak response of the isolation system, provided values of effective stiffness and effective damping approximate the force-deflection characteristics of the isolation system at the response <u>amplitude</u> of interest

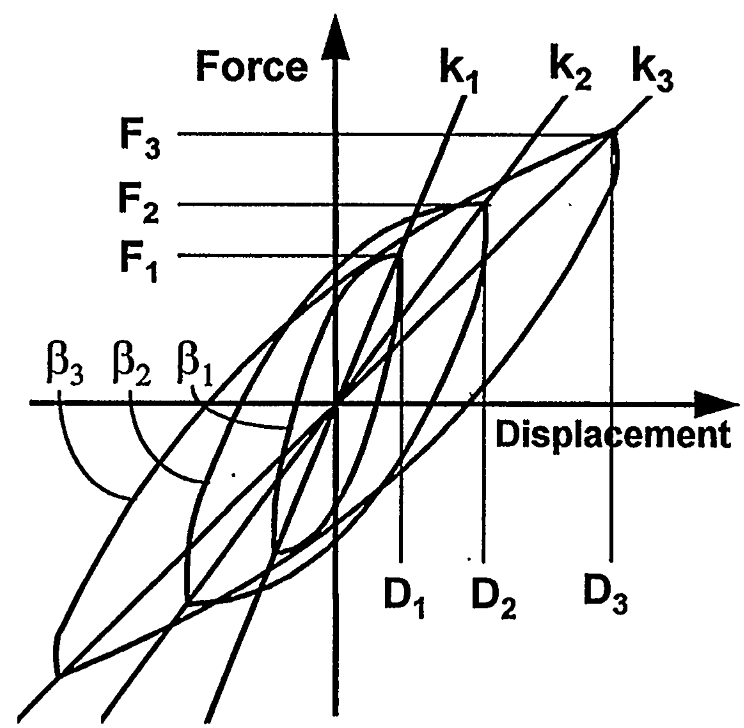
Charles Kircher & Associates
Consulting Engineers

Design Considerations for Isolated Structures

COMPARISON OF LINEAR AND NONLINEAR ISOLATION SYSTEMS



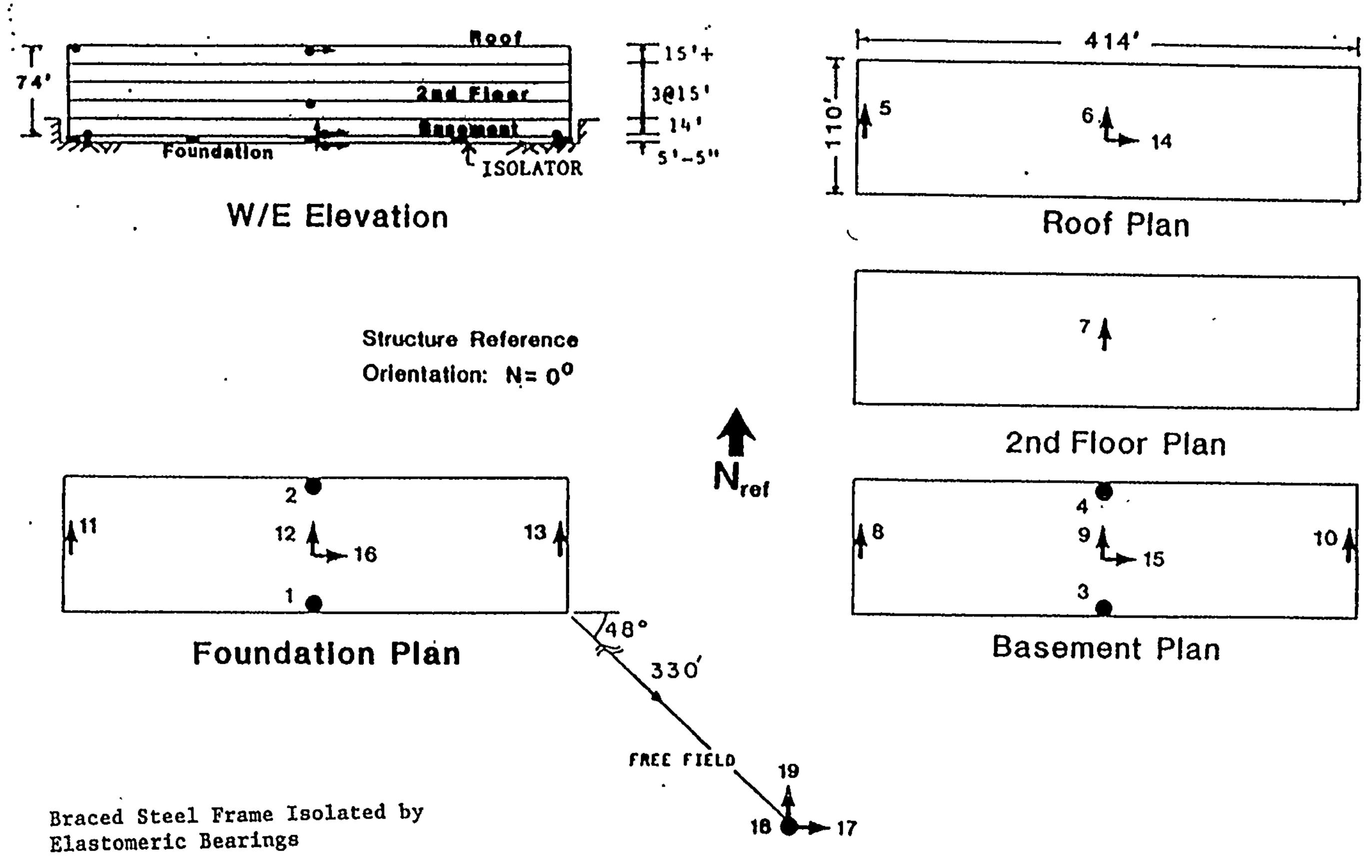
LINEAR SYSTEM
(Viscous Damping Only)



NONLINEAR SYSTEM
(Viscous Plus Hysteretic Damping)

Design Considerations for Isolated Structures

RANCHO CUCAMONGA LAW AND JUSTICE CENTER: CSMIP SENSOR LOCATIONS[1]

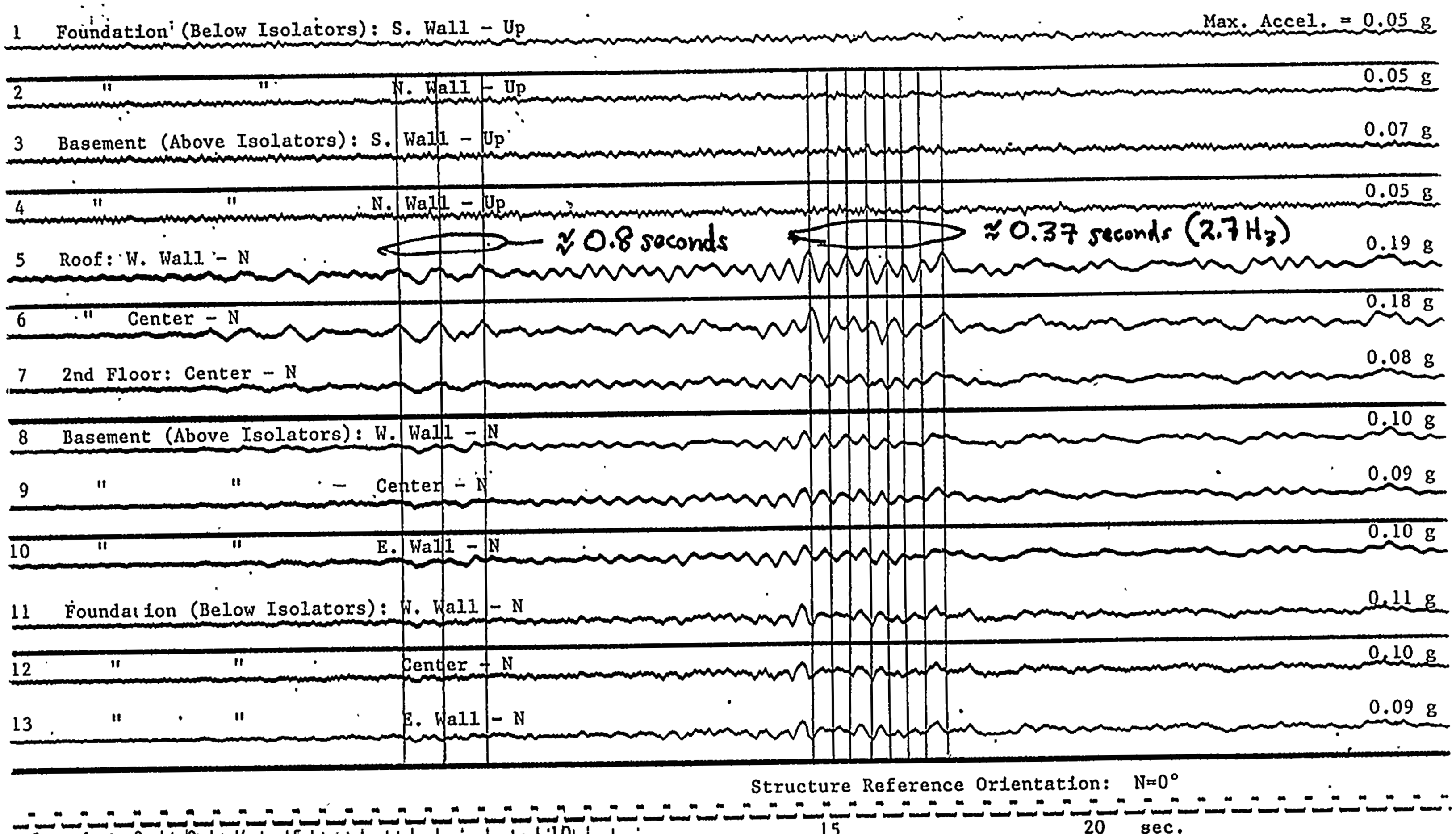


Braced Steel Frame Isolated by
Elastomeric Bearings
Design Date: 1983

1. State of California, Division of Mines and Geology, "Quick Report on CSMIP Strong-Motion Records from the June 28, 1992 Earthquake near Landers and Big Bear, California," CSMIP Report OSMS 92-06.

Design Considerations for Isolated Structures

RANCHO CUCAMONGA LAW AND JUSTICE CENTER: RECORDS FROM THE JUNE 28, 1992 LANDERS EARTHQUAKE (7.4)[1]



1. State of California, Division of Mines and Geology, "Quick Report on CSMIP Strong-Motion Records from the June 28, 1992 Earthquake near Landers and Big Bear, California," CSMIP Report OSMS 92-06.

<u>Design Considerations for Isolated Structures</u>

HIGHER-MODE EFFECTS, TORSION AND ROCKING/OVERTURNING

- **TO MINIMIZE HIGHER-MODE EFFECTS (e.g., for Protection of Fragile Structures, Equipment or Building Contents):**
  - Use a viscous-damped isolation system or a hysteretic-damped system with a low level of effective damping (e.g., avoid sliding systems and other hysteretic systems with high levels of effective damping)
  - Use an isolation system with an effective period several times that of the superstructure (on a fixed-base)
- **TO MINIMIZE TORSION (e.g., Buildings of Irregular Configuration):**
  - Use an isolation system that minimizes mass eccentricity effects (e.g., sliding system)
  - Position stiffer isolators along the perimeter of the structure
- **TO MINIMIZE ROCKING/OVERTURNING (e.g., Tall Structures or Tall Portions of a Structure):**
  - Use a highly-damped isolation system to minimize overturning forces on isolators
  - Use isolators that are "stiff" in the vertical direction to avoid coupling of vertical and horizontal response of isolators

ISOLATION SYSTEM DESIGN DETAILS

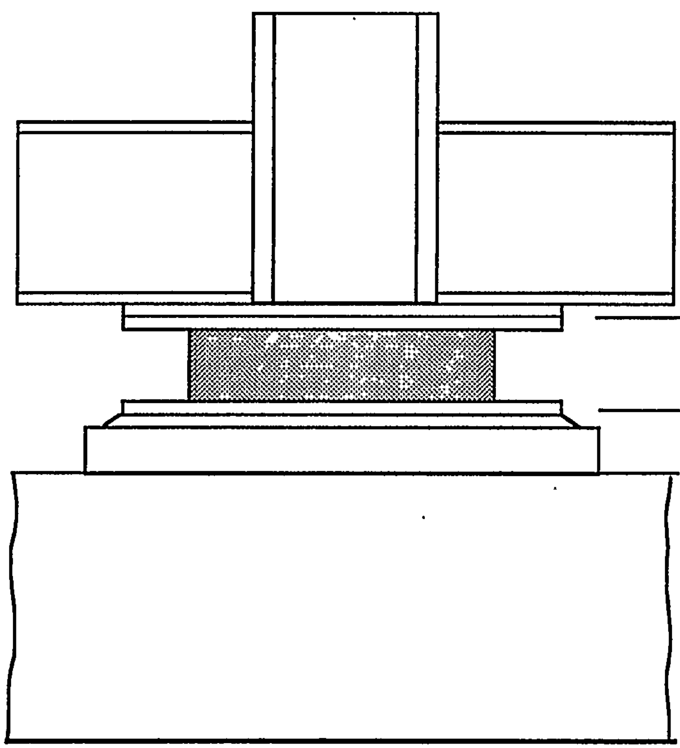
- FORCE TRANSFER AT THE ISOLATION INTERFACE
  - Total moment at the isolation system interface is a combination of the lateral seismic shear force, V, times the effective height, h, of the isolator, plus the P-Δ moment
  - Distribution of total moment between the foundation and the superstructure can vary significantly depending on the type of isolator (i.e., elastomeric or sliding)
  - Distribution of total moment between the foundation and the superstructure can vary significantly for elastomeric bearings depending on the method of connection of isolator to the structure (i.e., bolted or dowelled) and on the relative rotational stiffness of the superstructure (and foundation)
- DESIGN DETAILS AND COSTS
  - Design details (and costs) can be influenced significantly depending on the magnitude and distribution of total moment (particularly for building retrofit projects)

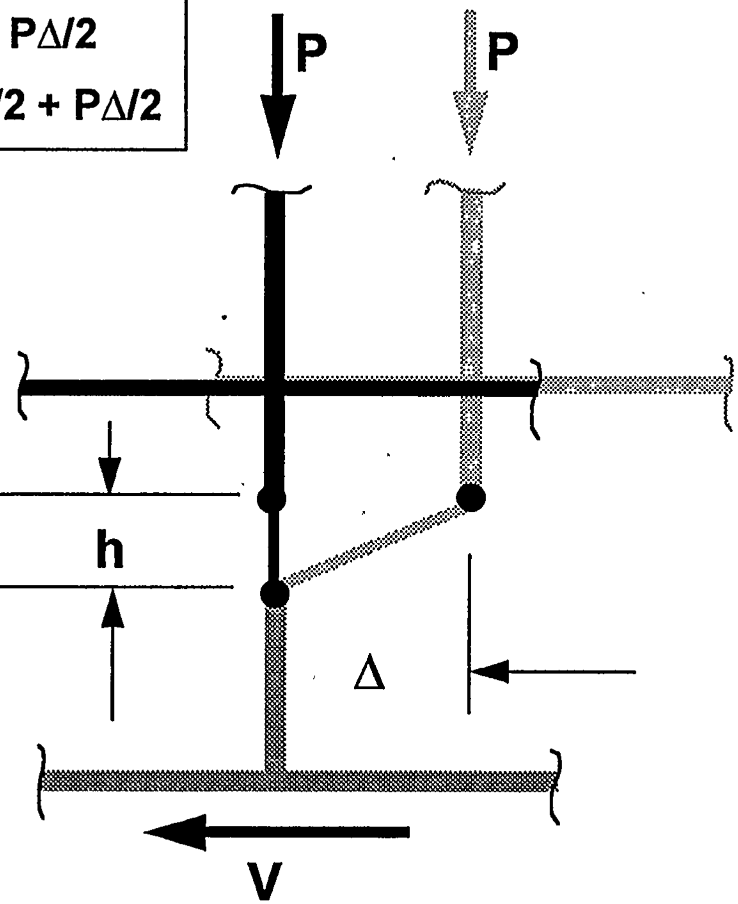
Design Considerations for Isolated Structures

FORCE TRANSFER AT ISOLATION INTERFACE:
ELASTOMERIC BEARINGS - BOLTED CONNECTIONS
(SUPERSTRUCTURE RESTRICTS ROTATION OF BEARING TOP PLATE)

MOMENT AT TOP OF BEARING: $Vh/2 + P\Delta/2$

MOMENT AT BOTTOM OF BEARING: $Vh/2 + P\Delta/2$

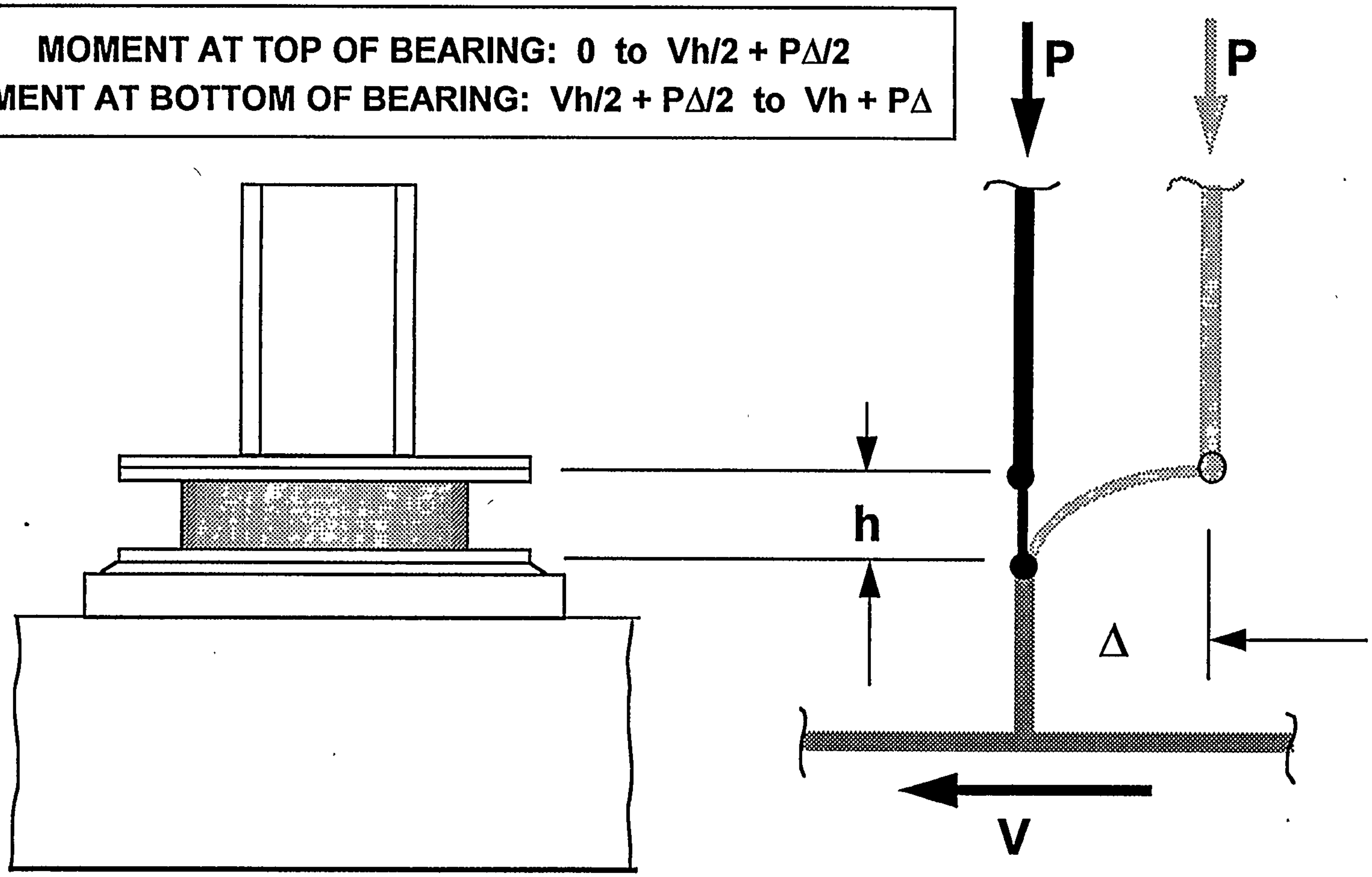




Design Considerations for Isolated Structures

FORCE TRANSFER AT ISOLATION INTERFACE:
ELASTOMERIC BEARINGS - DOWELED CONNECTIONS
(SUPERSTRUCTURE PERMITS SOME ROTATION OF BEARING TOP PLATE)

MOMENT AT TOP OF BEARING: 0 to $Vh/2 + P\Delta/2$
MOMENT AT BOTTOM OF BEARING: $Vh/2 + P\Delta/2$ to $Vh + P\Delta$

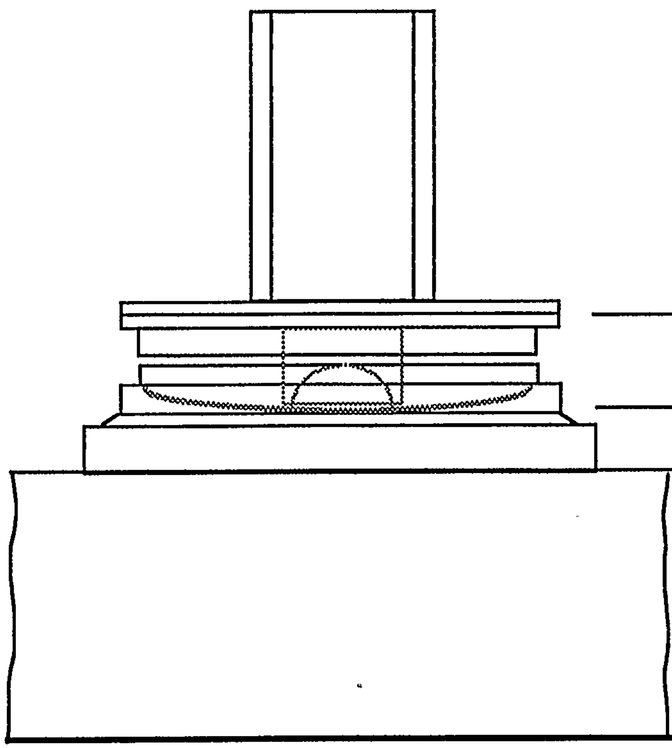


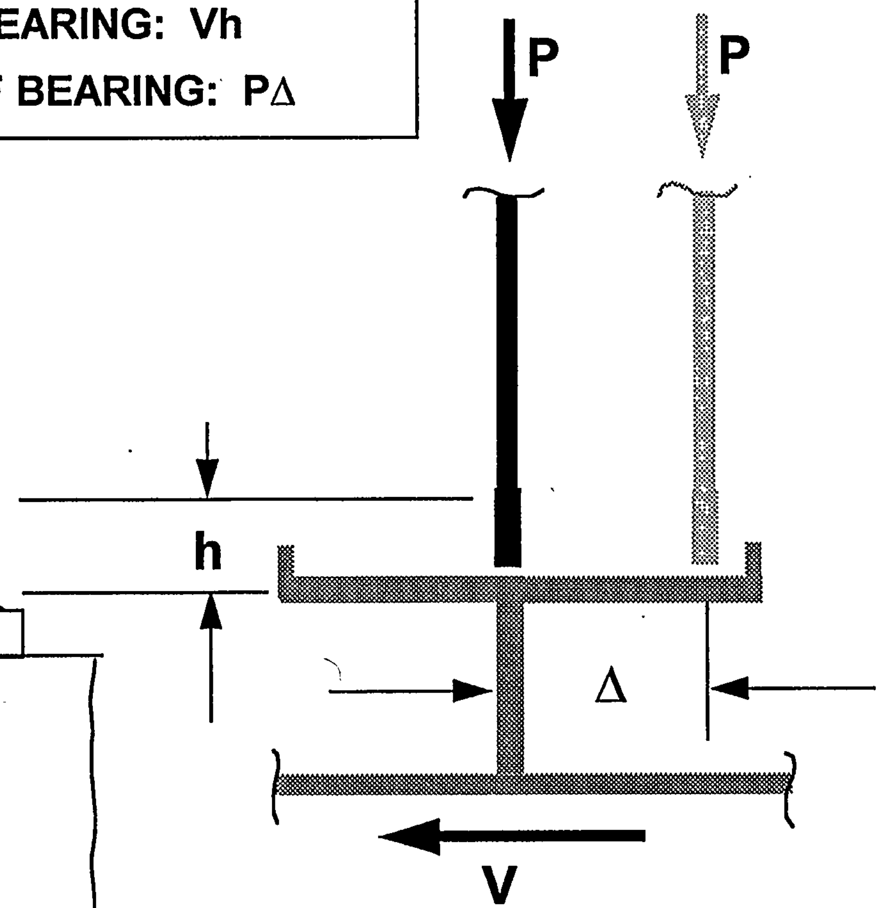
Design Considerations for Isolated Structures

FORCE TRANSFER AT ISOLATION INTERFACE: SLIDING BEARINGS - SLIDING SURFACE FACE UP

MOMENT AT TOP OF BEARING: Vh

MOMENT AT BOTTOM OF BEARING: PΔ





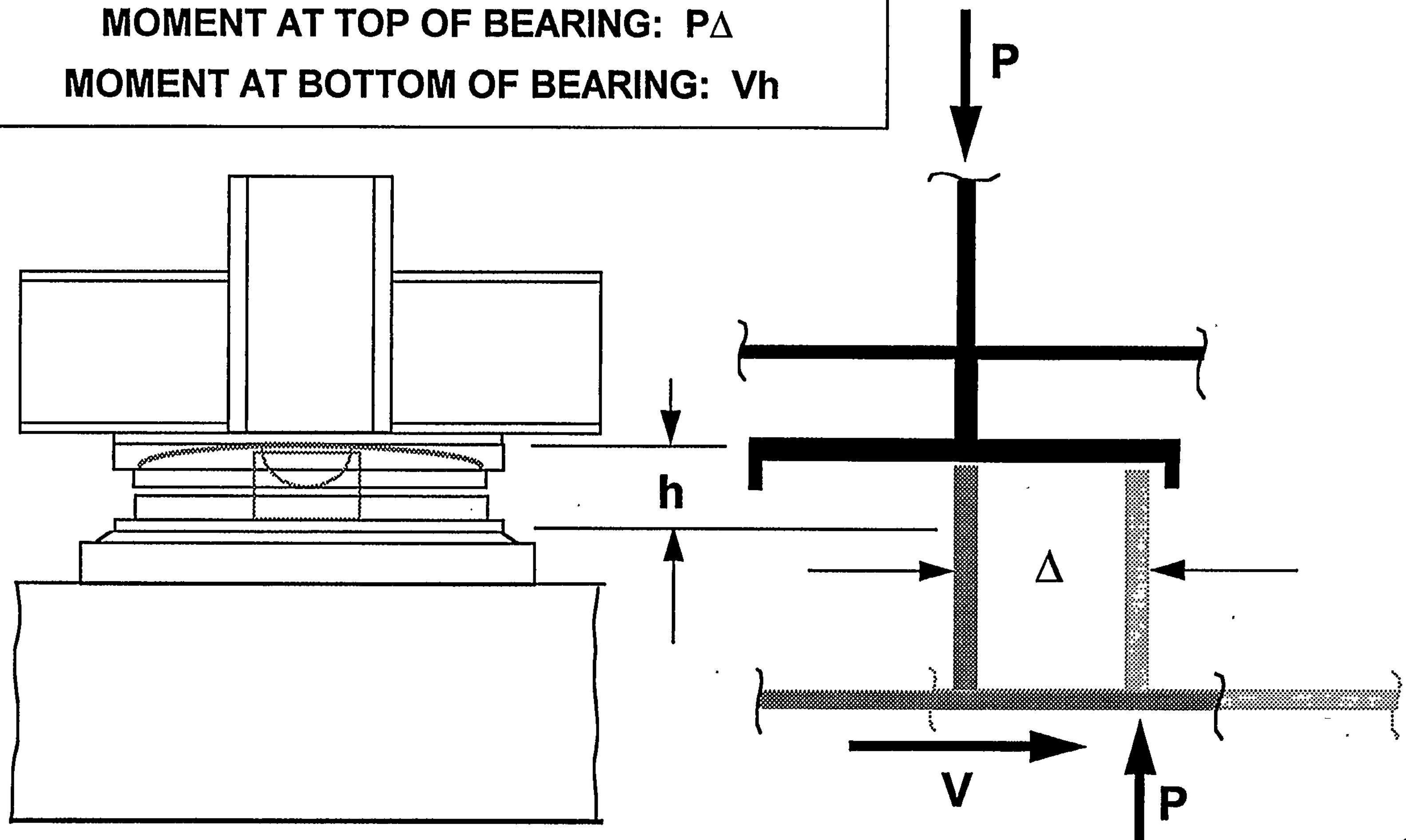
Charles Kircher & Associates
Consulting Engineers

<u>Design Considerations for Isolated Structures</u>

FORCE TRANSFER AT ISOLATION INTERFACE: SLIDING BEARINGS - SLIDING SURFACE FACE DOWN

MOMENT AT TOP OF BEARING: $P\Delta$

MOMENT AT BOTTOM OF BEARING: Vh



Design Considerations for Isolated Structures

NONSTRUCTURAL COMPONENT ISSUES

- **COMPONENTS THAT CROSS THE ISOLATION INTERFACE**
  - Conduit, piping and other equipment that cross or span the isolation interface require flexible couplings , etc
  - Architectural elements that cross the isolation interface require clearances and/or special slip details
- **COMPONENTS LOCATED IN THE ISOLATED STRUCTURE**
  - In general, forces on architectural components and equipment of a building will be much lower, exceptions:
    - Long-period equipment (e.g., piping and conduit on long, flexible hangers without lateral bracing and certain vibration-isolated equipment)
    - Buildings susceptible to higher-mode response (e.g., buildings with highly-damped, hysteretic isolation systems)
- **EQUIPMENT QUALIFICATION**
  - Seismic qualification of equipment located in buildings with "nonlinear" systems should use loads that include the full effects of higher-mode response

<u>Design Considerations for Isolated Structures</u>

WIND DESIGN

- ♦ BACKGROUND
  - Buildings and structures made flexible at their base by an isolation system may be more susceptible to motion due to wind
  - Wind loads have two components: a gradually varying pressure component and a dynamic pressure component
  - Wind loads can cause "ratcheting" displacement of hysteretic systems
  - Isolation system may require special evaluation for wind load:
    - Tall structures in areas of relatively high wind load
    - Isolation systems without a special wind restraint mechanism or an adequate level of lateral restoring force
    - Design wind load exceeds a conservative estimate of the "yield point" of the isolation system
- ♦ DESIGN CONCERNS
  - In general, life-safety and damage should not be a concern (exception: sliding systems without proper restoring force)
  - Hysteretic systems may be displaced during a wind storm and maintain a "permanent offset"
  - Building occupants are more likely to experience discomfort due to wind-induced motion

<u>Design Considerations for Isolated Structures</u>

VERTICAL ISOLATION

- POTENTIAL BENEFITS
  - Buildings with contents sensitive to vertical floor vibration
  - Structures and equipment vulnerable or sensitive vertical seismic motion
- EXAMPLES OF VERTICAL ISOLATION:
  - Buildings (very limited application)
  - Heavy equipment on flexible supports (e.g., turbines, etc.)
  - Liquid metal reactor (LMR) vessels (conceptual design only)
  - Computer floors (IBM design)
- WHEN IS VERTICAL ISOLATION FEASIBLE?
  - Structures/equipment that can accommodate rocking response
  - Structures/equipment that can accommodate changes in elevation due to seismic response or changes in vertical load
- PEAK DISPLACEMENT OF A 1-SECOND DYNAMIC SYSTEM

Peak Displacement (Inches) ≅ 10 x Peak Acceleration (g's)

<u>Design Considerations for Isolated Structures</u>

ISOLATION OF EQUIPMENT AND BUILDING CONTENTS

- POTENTIAL BENEFITS
  - Fragile or vibration sensitive equipment and building contents
  - Equipment and building contents that can not be anchored conventionally to avoid earthquake overturning
- EXAMPLES OF EQUIPMENT AND BUILDING CONTENTS ISOLATION
  - Electrical switchyard gear (A. D. Edmonston Pumping Plant)
  - Pipe Bridges (Sellafield Nuclear Reprocessing Facility, England)
  - Art objects (Getty Museum)
  - Emergency mobile-exciter equipment (Diablo Canyon design)
  - Computer floors (IBM design, Japan)
- DESIGN CONCERNS
  - In general, concepts are the same although isolation of building equipment or contents must also include the effects of building response
  - Isolated equipment or building contents require adequate clearance to other non-isolated portions of the building
  - Special isolation systems are required for equipment and building contents that are relatively light

Design Considerations for Isolated Structures

WEIGHT LIMITATIONS

- **HEAVY-LOAD LIMITS ON ISOLATORS:**
  - No practical limits on heavy loads for either elastomeric or sliding isolator; elastomeric isolators have been made for loads up to 1,000 - 2,000 kips per bearing
- **LIGHT-LOAD LIMITS ON ISOLATORS:**
  - None for sliding systems (e.g., lateral force due to friction is approximately proportional to supported weight)
  - 10 - 50 kips for elastomeric isolators, unless configured as a stacked collection of small modular units (e.g., A. D. Edmonston circuit breakers)
- **EVALUATION OF MINIMUM ELASTOMERIC BEARING LOAD, P**
  - Assume 2-second period and 50 psi modulus for isolator design
  - Diameter required for a 2-second period (from Kelly lecture on bearing design):

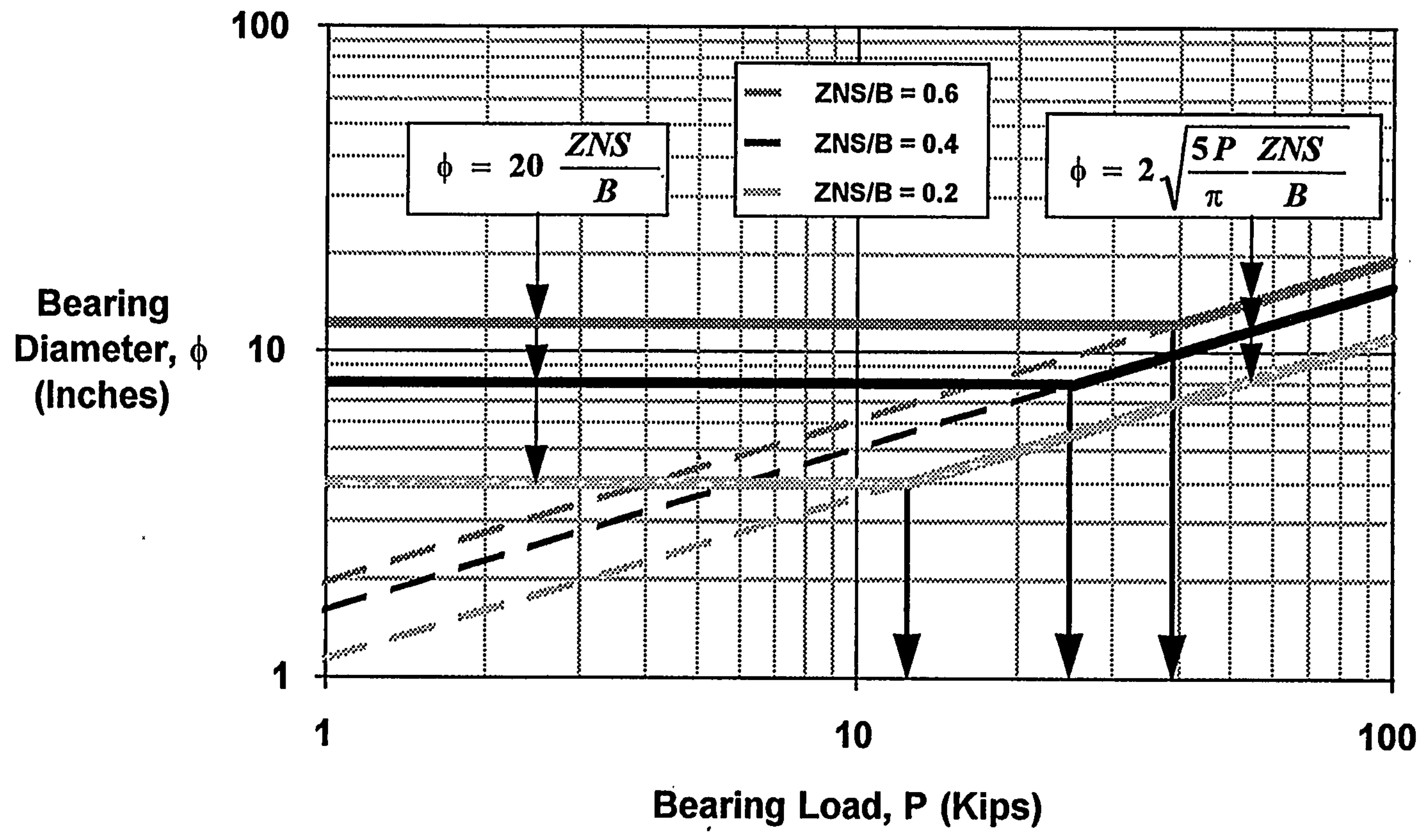
$$\phi = 2\sqrt{\frac{5P}{\pi}\frac{ZNS}{B}}$$

  - Minimum diameter for isolator stability (taken as the SEAOC/UBC design displacement, D):

$$\phi = 20\,\frac{ZNS}{B}$$

Design Considerations for Isolated Structures

MINIMUM BEARING LOAD REQUIRED TO EFFECT 2-SECOND PERIOD RESPONSE OF ELASTOMERIC ISOLATORS (50 PSI SHEAR MODULUS)



Design Considerations for Isolated Structures

HEIGHT LIMITATIONS

- The height of an isolated structure is limited by the ability of the isolators to support earthquake overturning load (both compression and uplift forces) and by the flexibility of the superstructure
- As the superstructure becomes more flexible (relative to the isolation system), the benefits of isolation are reduced; theory and observed earthquake performance suggest approximate values for the ratio, F_R, of the response of fixed-base and isolated structures:

$$F_R \approx 4 - 5 \qquad T_s / T_I \ll 1$$

$$F_R \approx 2 \qquad T_s / T_I \approx 1$$

$$F_R \approx 1 \qquad T_s / T_I \gg 1$$

- Period of superstructure, T_s, may be taken as:

$T_s = 0.049\,(h_n)^{3/4}$ Steel Frame Buildings

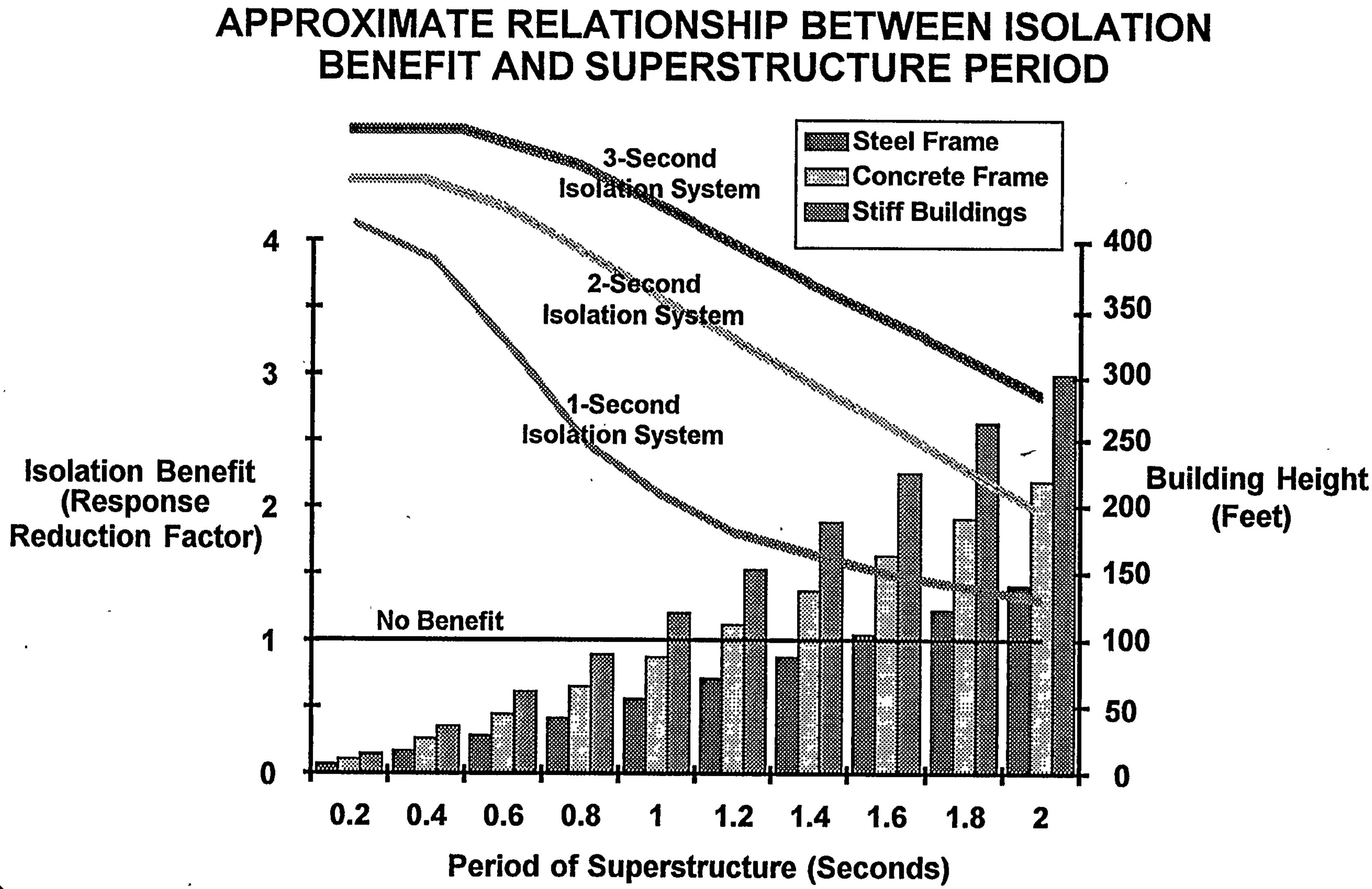
$T_s = 0.035\,(h_n)^{3/4}$ Concrete Frame Buildings

$T_s = 0.028\,(h_n)^{3/4}$ Other (Stiff) Buildings

T_s represents actual elastic period of superstructure, taken as 1.4 times values prescribed by the UBC (see ATC-3 Commentary)

Charles Kircher & Associates
Consulting Engineers

Design Considerations for Isolated Structures

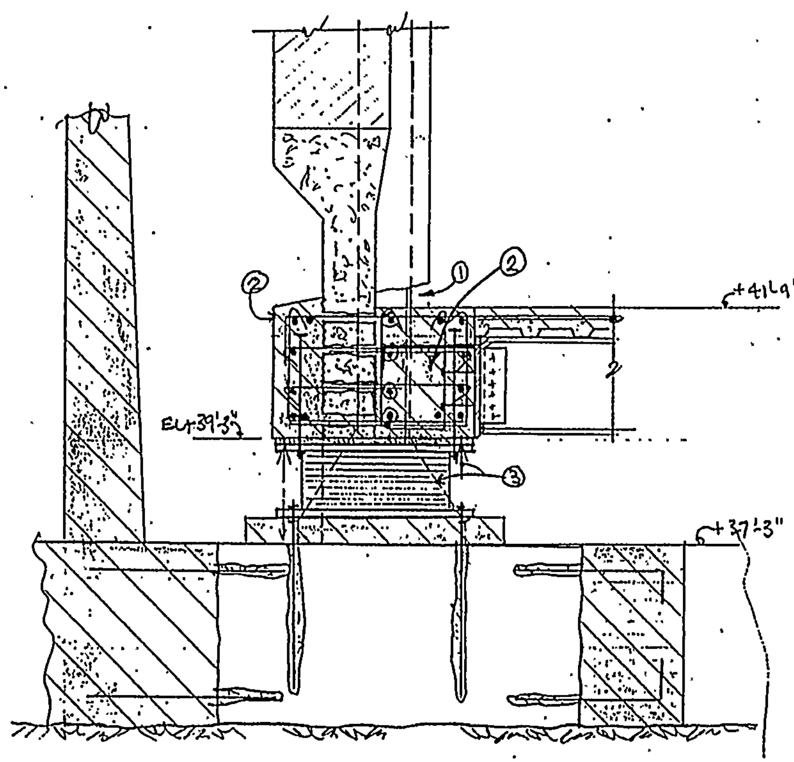


RETROFIT DESIGN

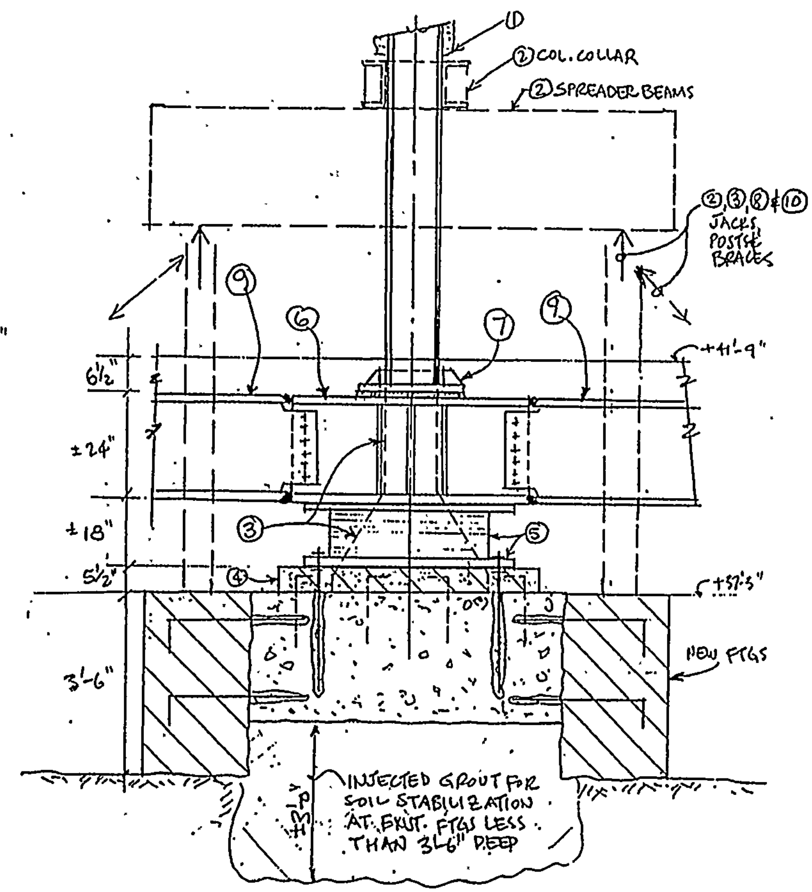
- SPECIAL DESIGN REQUIREMENTS
  - Project-specific design criteria should be developed (i.e., in general, building codes for new construction are not directly applicable)
  - Archaic materials, including unreinforced masonry material (URM), may require special investigation and testing to determine design properties
  - Selection of the "plane of isolation" requires special consideration of building conditions: space usage requirements, architectural features, clearances, access for construction, etc.)
  - Design of the isolation system requires special consideration of the strength of existing superstructure elements to minimize modifications
- SPECIAL CONSTRUCTION REQUIREMENTS AND COST ITEMS
  - Building may be occupied during retrofit
  - Foundations may require substantial strengthening
  - Excavation and addition of retaining walls may be required to create clearance for isolated structure
  - Structure will require local shoring while existing columns/walls are cut and isolators installed

EXAMPLE ISOLATION SYSTEM DETAILS: ASIAN ART MUSEUM[1]

PERIMETER



INTERIOR



1. Rutherford & Chekene, "Seismic Upgrading Study Of The Main Public Library Building For The Asian Art Museum At The Civic Center," Job No. 90066S, May 1992.

LLNL/DOE SHORT COURSE OF SEISMIC (BASE) ISOLATION

OVERVIEW OF SEISMIC CODE PROVISIONS

BY

Charles A. Kircher Ph.D., P.E.

August 11, 1992

OVERVIEW OF SEISMIC CODE PROVISIONS

VERY EARLY BUILDING CODE REQUIREMENTS

- If a builder builds a house for a man and does not make its construction firm and the house collapses and causes the death of the owner of the house - that builder shall be put to death
- If it destroys property, he shall restore whatever it destroyed, and because he did not make the house firm he shall rebuild the house which collapsed at his own expense
- If a builder builds a house for a man and does not make its construction meet the requirements and a wall falls - that builder shall strengthen the wall at his own expense

The Code of Hammurabi, c. 2250 B.C.

<u>OVERVIEW OF SEISMIC CODE PROVISIONS</u>

SEISMIC CODE DEVELOPMENT IN THE UNITED STATES[1]

- **1925 SANTA BARBARA EARTHQUAKE**
  - **Pacific Coast Building Officials (now known as the International Conference of Building Officials) publish the first edition of the *Uniform Building Code* (UBC) in 1927 with non-mandatory seismic provisions**
- **1933 LONG BEACH EARTHQUAKE**
  - **California State Legislature enacts Field Act: Division of Architecture (now known as the Office of the State Architect) responsible for approving design of public schools**
  - **Division of Architecture adopts regulations that specify: 0.10W for masonry buildings without frames, 0.02W - 0.05W for other buildings**
  - **California State Legislature enacts Riley Act: 0.02W established as minimum lateral force required for design most buildings**
  - **Minimum design lateral force of 0.08W required by City of Los Angeles and adopted in the 1935 edition of the UBC for Zone 3 (highest zone)**

1. Glen V. Berg, *Seismic Design Codes and Procedures,* Earthquake Engineering Research Institute, Berkeley, California, 1983.

OVERVIEW OF SEISMIC CODE PROVISIONS

SEISMIC CODE DEVELOPMENT IN THE UNITED STATES[1]

- PRE-SEAOC BLUE BOOK
  - City of Los Angeles adopts regulations in 1943 with seismic coefficient based on building height: $C = 0.6 / (N + 4.5)$, where N = number of stories
  - City of San Francisco adopts regulations with design lateral force ranging from 0.037W to 0.08W, depending on the number of building stories and soil conditions
  - Joint committee of SEAOC/ASCE (Northern/San Francisco Sections) develops recommendations for base shear, V: $V = CW$, where $C = 0.015/T$ and T, the building period is based on an empirical formula: $T = 0.05/D^{1/2}$
  - City of San Francisco adopts regulations based on SEAOC/ASCE recommendations, except: $C = 0.02/T$ and $0.035 < C < 0.075$
- SEAOC BLUE BOOK
  - Seismology Committee of SEAOC issues first edition of their "Recommended Force Requirements and Commentary" in 1960
  - International Conference of Building Officials (ICBO) incorporates SEAOC provisions in the 1961 UBC
  - Major Blue Book revisions made periodically: 1967, 1973-1974, 1980, and 1990 (current fifth edition); new editions of UBC based on Blue Book

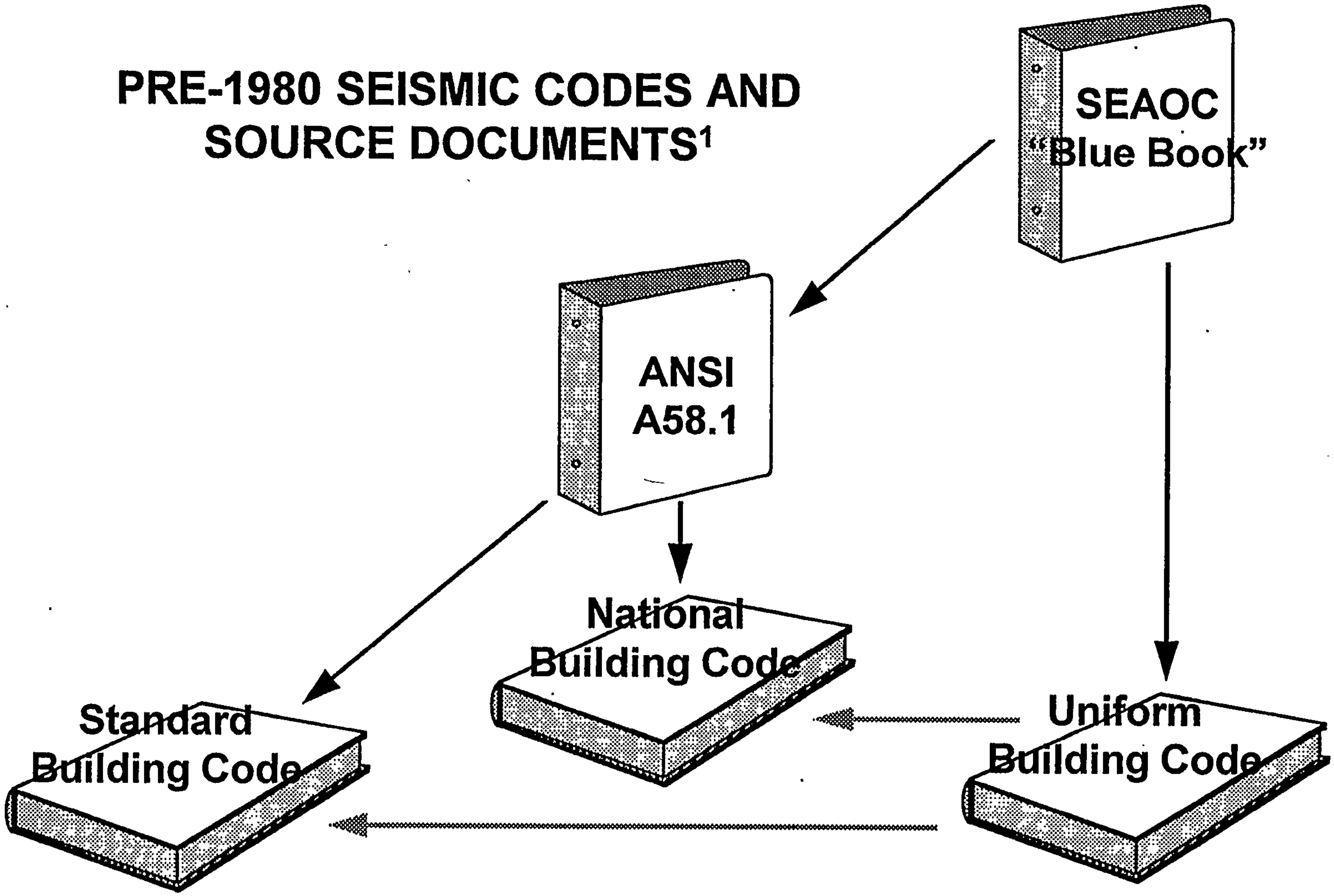
OVERVIEW OF SEISMIC CODE PROVISIONS

SEISMIC CODE DEVELOPMENT IN THE UNITED STATES[1]

- ♦ 1971 SAN FERANADO EARTHQUAKE
  - Substantial changes made to SEAOC/UBC provisions: increase in design lateral force level and more stringent "ductile" detailing requirements
  - Applied Technology Council (Research and development extension of SEAOC) embarks on National Science Foundation/National Bureau of Standards funded project in 1974 to develop a model seismic code
  - ATC publishes *Tentative Provision for the Development of Seismic Regulations for Buildings* (ATC-3) in 1978
  - Building Seismic Safety Council (BSSC)/Federal Emergency Management Agency (FEMA) develop first edition of the National Earthquake Hazard Reduction Program (NEHRP) provisions in 1985; provisions based on ATC-3
  - Seismic regulations of the National Building Code and the Standard Building Code based on the 1991 edition of NEHRP provisions

OVERVIEW OF SEISMIC CODE PROVISIONS

PRE-1980 SEISMIC CODES AND SOURCE DOCUMENTS[1]

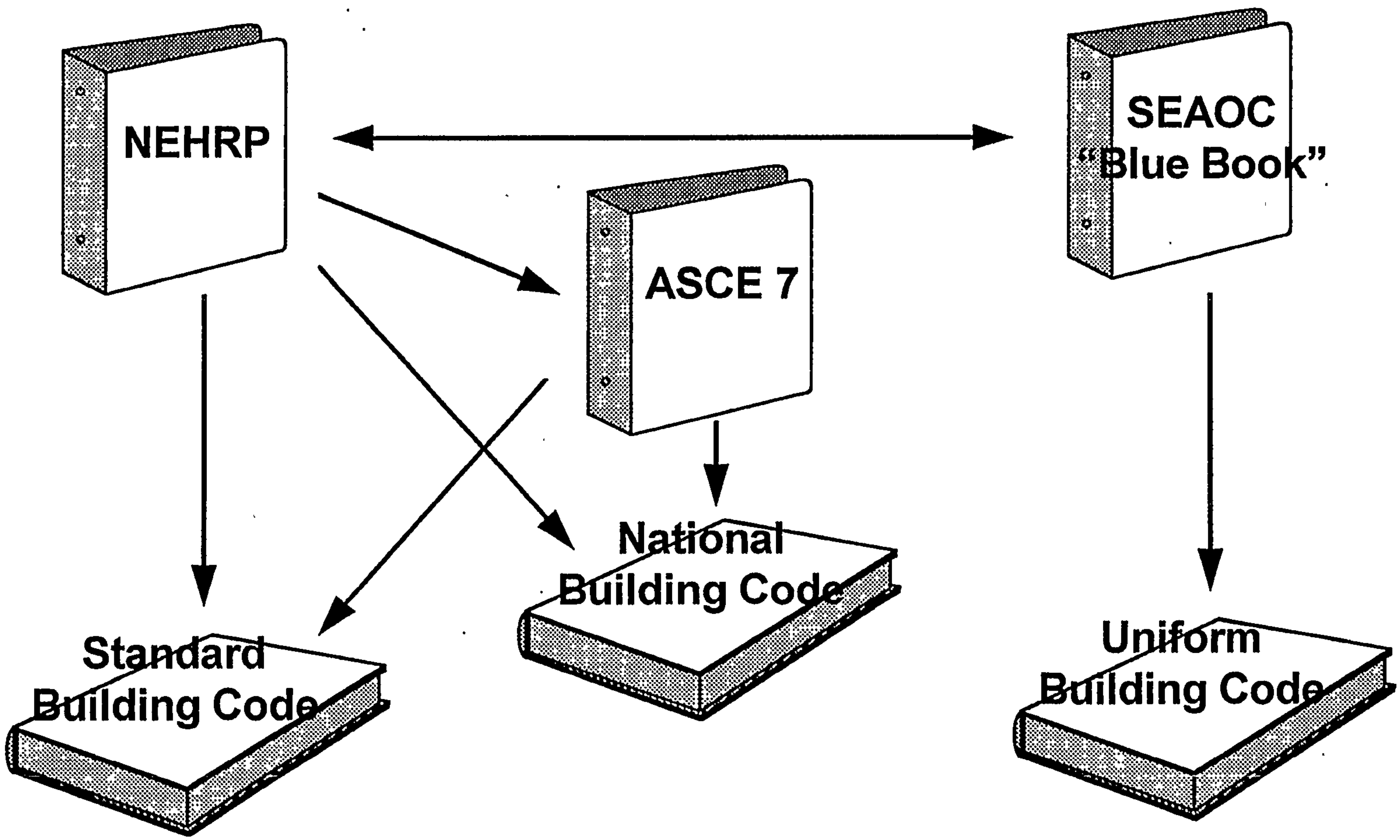


1. Diana Todd, "The World of Building Codes, *Phenomenal News*, Lawrence Livermore National Laboratoy, Volume 3, Number 3, April 1992

Charles Kircher & Associates
Consulting Engineers

OVERVIEW OF SEISMIC CODE PROVISIONS

CURRENT SEISMIC CODES AND SOURCE DOCUMENTS[1]



1. Diana Todd, "The World of Building Codes, *Phenomenal News*, Lawrence Livermore National Laboratoy, Volume 3, Number 3, April 1992

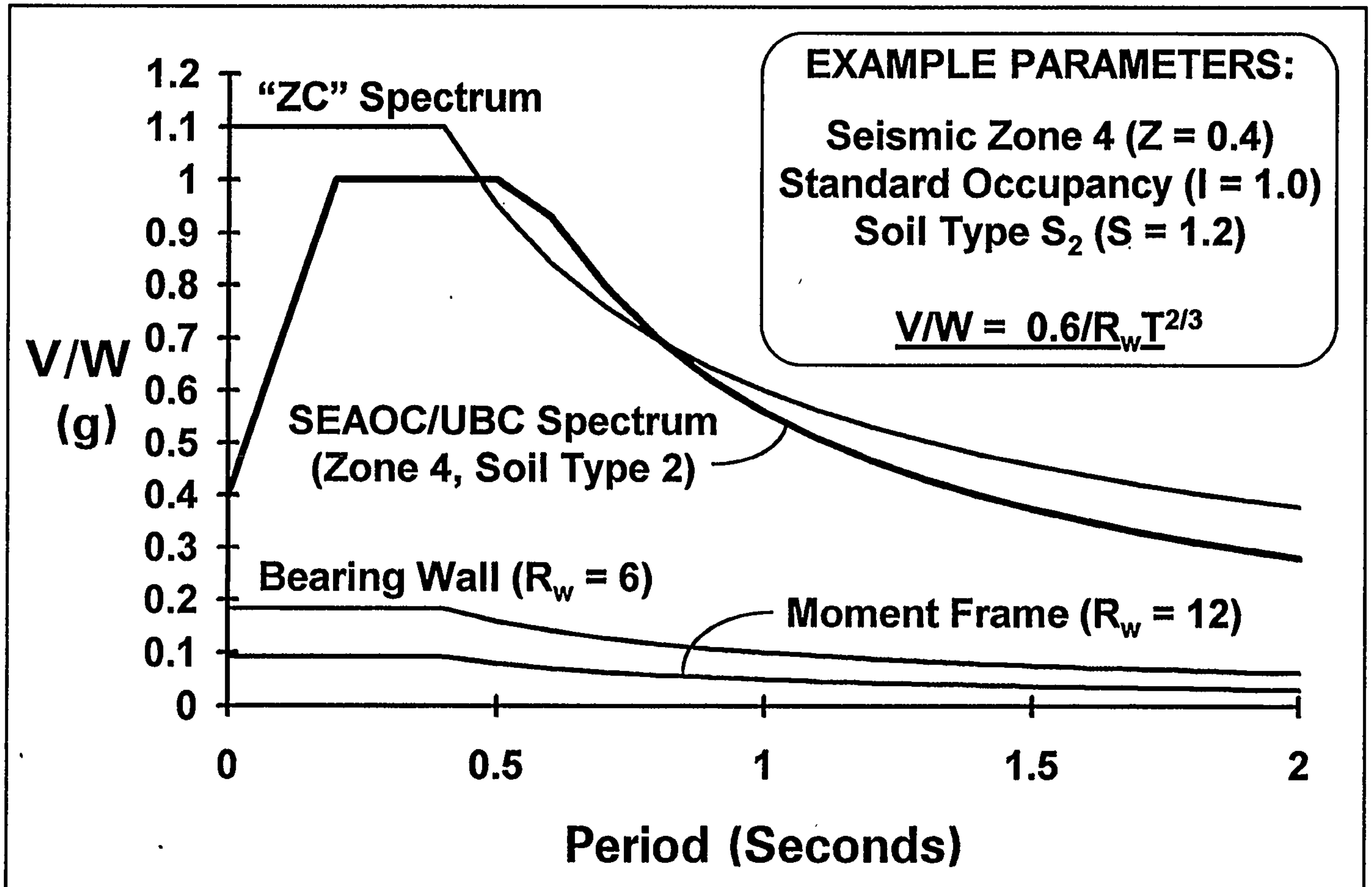
<u>OVERVIEW OF SEISMIC CODE PROVISIONS</u>

SEAOC/UBC SEISMIC DESIGN PHILOSOPHY

- ♦ PROTECT LIFE-SAFETY
  - Provide strength required to resist major earthquake ground shaking without collapse or life-threatening failure
  - Permit signifcant inelastic response of the lateral force resisting system
- ♦ BEND (NOT BREAK) APPROACH
  - Reduce theoretical elastic response by a factor (R_w) of up to 12 for working stress design of lateral force resisting elements
  - Design elements and connections to remain "ductile" for inelastic response taken as $3R_w/8$ times the "working stress" design level
  - Check structure stability for response taken as $3R_w/8$ times the "working stress" design level
- ♦ DAMAGE CONTROL
  - "Damage to and repairability of the structure was not a direct consideration in the development of the R_w factors. Thus it may be appropriate for the Engineer to consider these concerns and increase the level of loading, stiffness, and detailing requirements it it is desired to control the amount of damage and/or decrease the cost of repairing the damage." - 1990 SEAOC Blue Book Commentary

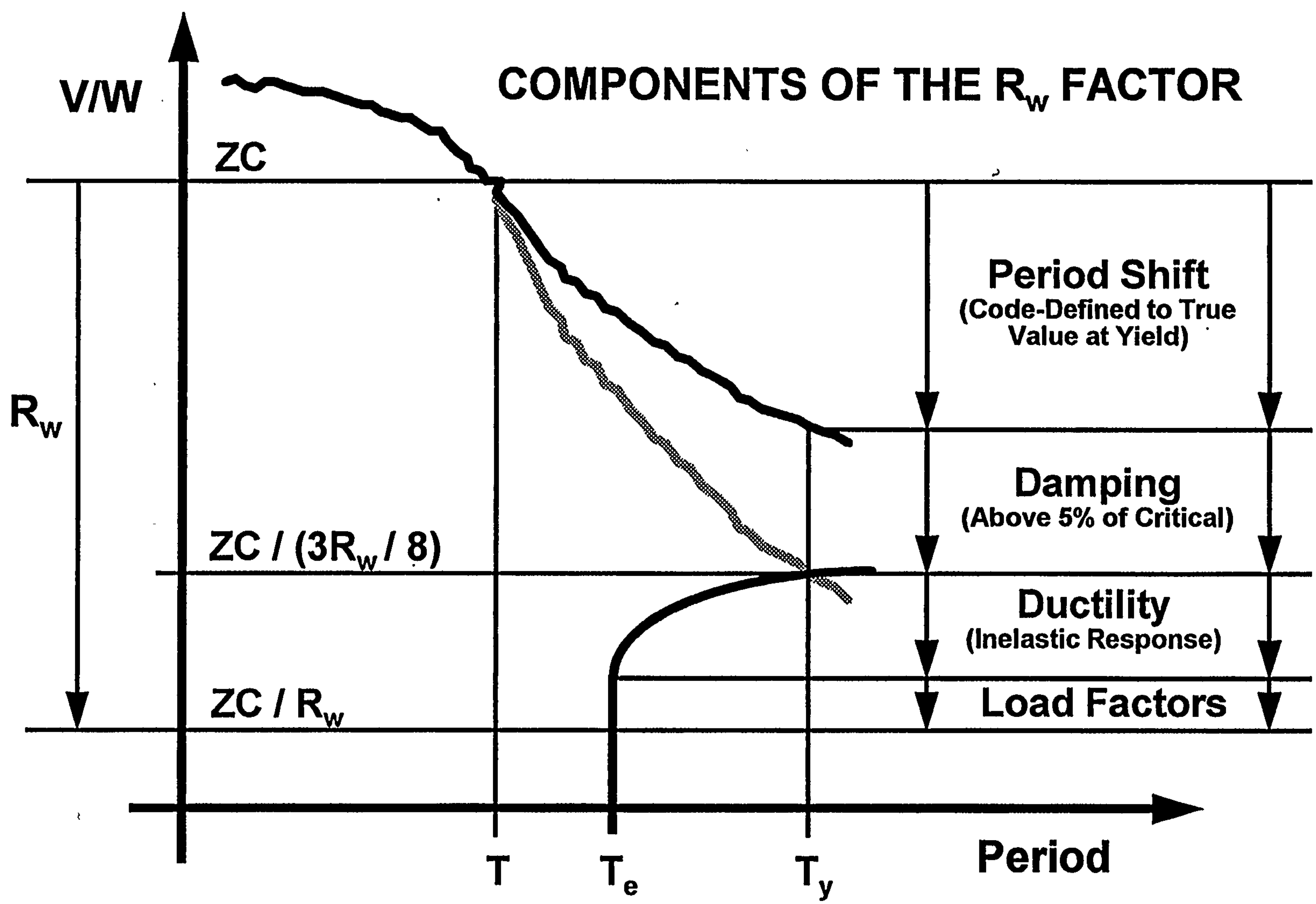
OVERVIEW OF SEISMIC CODE PROVISIONS

EXAMPLE COMPARISON OF EARTHQUAKE SPECTRA AND LATERAL FORCE REQUIRED FOR DESIGN OF BEARING WALL AND MOMENT FRAME SYSTEMS



Charles Kircher & Associates
Consulting Engineers

OVERVIEW OF SEISMIC CODE PROVISIONS



LLNL/DOE Short Course on Seismic Isolation

CURRENT BUILDING CODE PROVISIONS - SEAOC/UBC

by

Charles A. Kircher, Ph. D., P.E.

August 11, 1992

PRESENTATION AGENDA

OVERVIEW OF SEISMIC ISOLATION CODES

STATE AND LOCAL CODES

- Seismology Committee, Structural Engineers Association of California (SEAOC), *Recommended Lateral Force Requirements and Commentary*, ("Blue Book"), "Tentative General Requirements for the Design and Construction of Seismic-Isolated Structures," Appendix 1L, Fifth Edition, 1990

- Building Safety Board (BSB), California Office of Statewide Health Planning and Development, "An Acceptable Method for Design and Review of Hospital Buildings Utilizing Base Isolation," April 1989 (revised January 1992) - based on guidelines developed by SEAOC

- State of California, *California Administrative Code, Title 24, Building Standards, Part 2, Basic Regulations*, "Regulation for Seismic-Isolated Structures," - proposed changes incorporating BSB guidelines for hospitals into Title 24 (1994)

OVERVIEW OF SEISMIC ISOLATION CODES

FEDERAL AND NATIONAL CODES

- International Conference of Building Officials, *Uniform Building Code*, "Earthquake Regulations for Seismic-Isolated Structures," Division III of Chapter 23, 1991 Edition - based on Appendix 1L of the SEAOC Blue Book

- Building Seismic Safety Council (BSSC) of the National Institute of Building Sciences, *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings* - Technical Subcommittee 12: Base Isolation/Energy Dissipation tasked with developing requirements for the 1994 NEHRP provisions update

- United States General Services Administration, *Facilities Standards for the Public Buildings Service*, PBS P 3430.1A , "Base Isolated Structures," (Appendix 12-D of Chapter 12, "Seismic Design Guidelines") - references 1986 SEAONC "Yellow Book"

- American Association of State Highway and Transportation Officials, *Guide Specifications for Seismic Isolation Design*, June 1991 - based on static analysis formulas of SEAOC/UBC

BACKGROUND AND PHILOSOPHY OF SEAOC/UBC

WHY?
(NEED FOR SEAOC/UBC PROVISIONS)

- Building codes and regulations provide seismic requirements for fixed-base buildings that are not directly applicable to isolated structures
- Isolated buildings and structures are being designed and constructed in California and other states (e.g., first isolated building: Foothill County Law and Justice Center was designed and constructed in the mid-1980's)

BACKGROUND AND PHILOSOPHY OF CODE

SEAOC ISOLATED-STRUCTURE CODE EFFORTS

- Base isolation committees have been active in the northern and southern sections of the Structural Engineers Association of California (SEAOC) since mid-1980's
- Northern section of SEAOC published isolated structure design requirements (SEAONC "Yellow Book") in September 1986
- State Seismology Committee formed Ad Hoc Base Isolation Subcommittee in 1988 to develop "consensus" document (Appendix 1L of the 1990 SEAOC "Blue Book")
- International Conference of Building Officials (ICBO) adopted SEAOC provisions as non-mandatory appendix of the Uniform Building Code (Division III of Chapter 23 of the 1991 UBC)

BACKGROUND AND PHILOSOPHY OF SEAOC/UBC

SCOPE AND LIMITATIONS

- SEAOC/UBC provisions are intended primarily for design of new buildings and other ground-supported structures
- SEAOC/UBC provisions do not preclude, but may not fully prescribe design requirements for:

  (1) vertical isolation of buildings and structures

  (2) isolation retrofit of existing buildings and structures

  (3) isolation of equipment and other secondary systems located in a building or structure

- SEAOC/UBC provisions may not allow certain isolation systems

BACKGROUND AND PHILOSOPHY OF SEAOC/UBC

ACCEPTABLE ISOLATION SYSTEMS

- In general, the SEAOC/UBC provisions will permit the use of an isolation system that can be shown to:

  (1) remain stable for the required design displacements

  (2) provide increasing resistance with increasing displacement

  (3) not degrade under repeated cyclic load

  (4) have quantifiable engineering parameters (e,g,, stiffness and damping)

BACKGROUND AND PHILOSOPHY OF SEAOC/UBC

DESIGN STRATEGY

- Trade-off: Use isolation to improve the seismic performance of the building and contents above that of a fixed-base building (comparable superstructure and cost)

  or

  Use isolation to achieve the same performance as a fixed-base building (less superstructure and cost)

- SEAOC/UBC provisions are intended primarily to improve the seismic performance of an isolated building and its contents over that of a comparable fixed-base building, rather than to effect design economy of the superstructure

BACKGROUND AND PHILOSOPHY OF SEAOC/UBC

SEISMIC CRITERIA

- SEAOC/UBC provisions define two levels of earthquake ground motion:

  DESIGN-BASIS EARTHQUAKE (DBE) - the level of ground shaking that has a 10 percent probability of being exceeded in a 50-year period (same earthquake definition as that specified by the SEAOC/UBC provisions fixed-base building design)

  MAXIMUM CREDIBLE EARTHQUAKE (MCE) - the maximum level of ground shaking that may ever be expected at the building site within the known geological framework, may be taken as the level of ground motion that has a 10 percent probability of being exceeded in 250 years

BACKGROUND AND PHILOSOPHY OF SEAOC/UBC

BASIC DESIGN APPROACH

- SEAOC/UBC provisions require the structure above the isolation system to remain "essentially elastic" for the design basis earthquake (DBE)
- SEAOC/UBC provisions require the isolation system to be designed and tested for the full effects of the maximum credible earthquake (MCE)

BACKGROUND AND PHILOSOPHY OF SEAOC/UBC

PROTECTION PROVIDED BY FIXED-BASE BUILDINGS

RISK CATEGORY	EARTHQUAKE GROUND SHAKING LEVEL		
	Minor	Moderate	Major
Life-Safety	✓	✓	✓
Structural Damage	✓	✓	
Nonstructural and Contents Damage	✓		

BACKGROUND AND PHILOSOPHY OF SEAOC/UBC

PROTECTION PROVIDED BY ISOLATED BUILDINGS

RISK CATEGORY	EARTHQUAKE GROUND SHAKING LEVEL		
	Minor	Moderate	Major
Life-Safety	✓	✓	✓
Structural Damage	✓	✓	(✓)
Nonstructural and Contents Damage	✓	✓	(✓)

BACKGROUND AND PHILOSOPHY OF SEAOC/UBC

SOURCES OF UNCERTAINTY IN THE RESPONSE OF SEISMICALLY ISOLATED BUILDINGS

- SEISMIC LOAD (i.e., level, frequency, content and duration of ground motion)
- ISOLATION SYSTEM (i.e., design properties and long-term reliability of isolators)
- DESIGN METHOD (i.e, equivalent static, response spectrum or time history analysis)

CRITERIA SELECTION

DESIGN METHODS

- STATIC ANALYSIS

  (1) Establishes minimum design displacements and forces

  (2) Useful for preliminary design and design review

- DYNAMIC ANALYSIS

  (1) Establishes site-specific ground motion criteria

  (2) Specifies response spectrum analysis methods

  (3) Specifies time history analysis methods

CRITERIA SELECTION

SITE-SPECIFIC SPECTRA REQUIRED

- Isolated structure is located on a soft soil (i.e., Soil Type S_3, or S_4)
- Isolated structure is located near an active fault (i.e., within 15 km)
- Isolated structure period is very long (i.e., greater than 3 seconds)
- Isolated structure is located in zones of low or moderate seismicity (i.e., Seismic Zone 1, 2a or 2b)

CRITERIA SELECTION

RESPONSE SPECTRUM ANALYSIS REQUIRED

- When site-specific spectra are required
- Superstructure is "irregular" (e.g., structure above the isolation system has a "soft" or "weak" story)
- Superstructure is tall (i.e., structure above the isolation system exceeds 4 stories or 65 feet, in height)
- Isolation system is, or may be "coupled" dynamically with the superstructure (i.e., the isolated-structure period is less than or equal to 3 times the elastic, fixed-base period of the structure above the isolation system)

CRITERIA SELECTION

TIME HISTORY ANALYSIS REQUIRED*

- Isolated structure has a "nonlinear" isolation system (e.g., sliding systems, highly-damped systems)
- Isolated structure has a "nonlinear" superstructure (i.e, superstructure response exceeds elastic analysis limits)
- Isolated structure is located on very soft soil and may be subjected to long-period, long-duration shaking (i.e., Soil Type S_4)

* Time history analysis may be used in lieu of response spectrum analysis

STATIC ANALYSIS

PRESCRIPTIVE FORMULAS

- Formulas directly applicable to design of isolated buildings with "stiff" and "regular" superstructures, and "linear" isolation systems
- Formulas also applicable to preliminary design of other types of isolated buildings
- Formulas appropriate for design adequacy checking by peer reviewers and building officials

Charles Kircher & Associates
Consulting Engineers

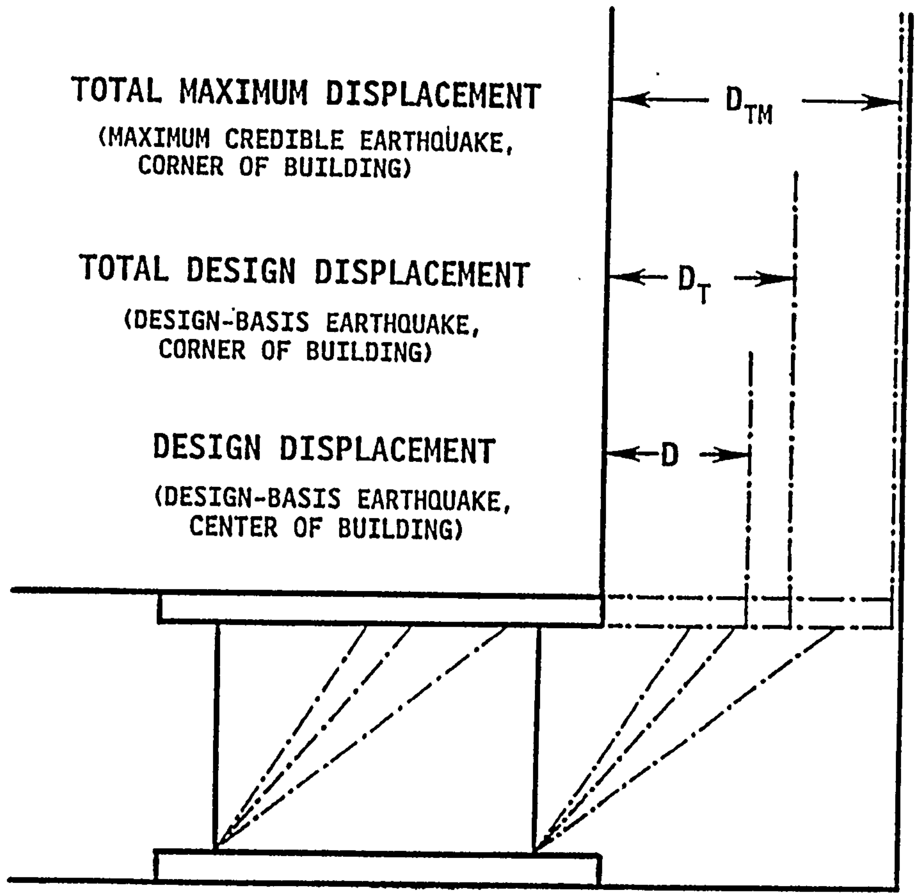
STATIC ANALYSIS

DISPLACEMENT TERMINOLOGY

TOTAL MAXIMUM DISPLACEMENT
(MAXIMUM CREDIBLE EARTHQUAKE, CORNER OF BUILDING)

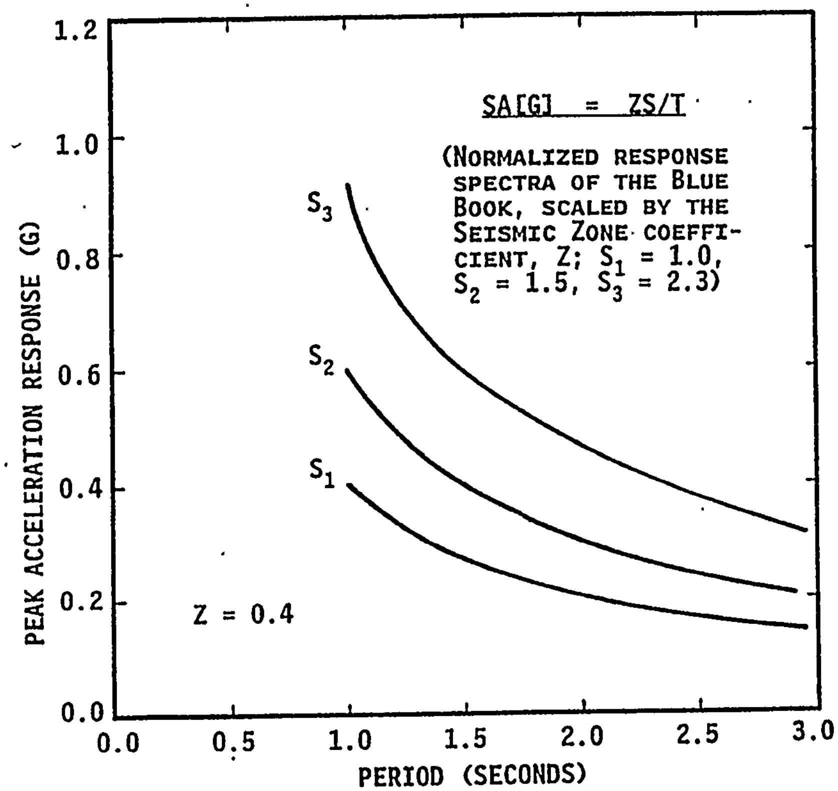
TOTAL DESIGN DISPLACEMENT
(DESIGN-BASIS EARTHQUAKE, CORNER OF BUILDING)

DESIGN DISPLACEMENT
(DESIGN-BASIS EARTHQUAKE, CENTER OF BUILDING)



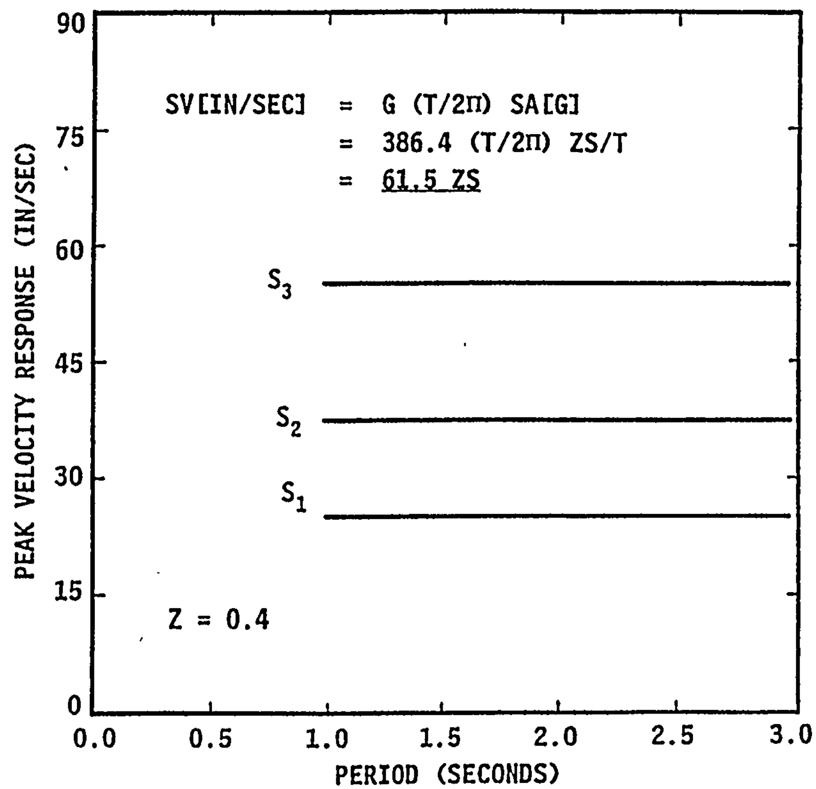
STATIC ANALYSIS

LONG-PERIOD SPECTRAL ACCELERATION (5%-DAMPED RESPONSE)



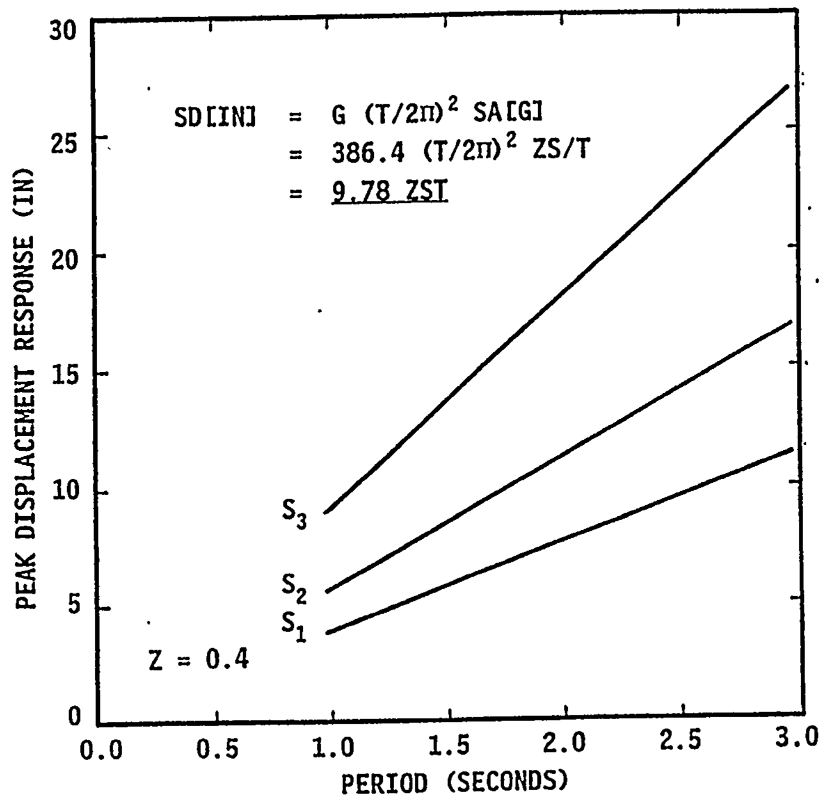
STATIC ANALYSIS

LONG-PERIOD SPECTRAL VELOCITY
(5%-DAMPED RESPONSE)



STATIC ANALYSIS

LONG-PERIOD SPECTRAL DISPLACEMENT (5%-DAMPED RESPONSE)



STATIC ANALYSIS

DESIGN DISPLACEMENT, FORMULA (E-1)*

The isolation system shall be designed to withstand a peak lateral design displacement, D, in inches, as follows:

$$D = \frac{10(ZNS_I)\,T_I}{B}$$

where:
- Z = seismic zone coefficient
- N = near-field coefficient
- S_I = site-soil coefficient
- T_I = effective period of the isolated building (seconds)
- B = damping coefficient

* Formula numbers refer to the SEAOC provisions, UBC formulas are identical but have a different numbering scheme

STATIC ANALYSIS

EFFECTIVE PERIOD, FORMULA (E-2)

The effective period, T_I, of the isolated building shall be determined using the deformational characteristics of the isolation system as follows:

$$T_I = 2\pi \sqrt{\frac{W\,g}{k_{min}}}$$

where:
- W = total seismic dead load above the isolation interface
- g = gravity constant
- k_{min} = minimum effective stiffness of the isolation system at design displacement, D, as determined by prototype testing

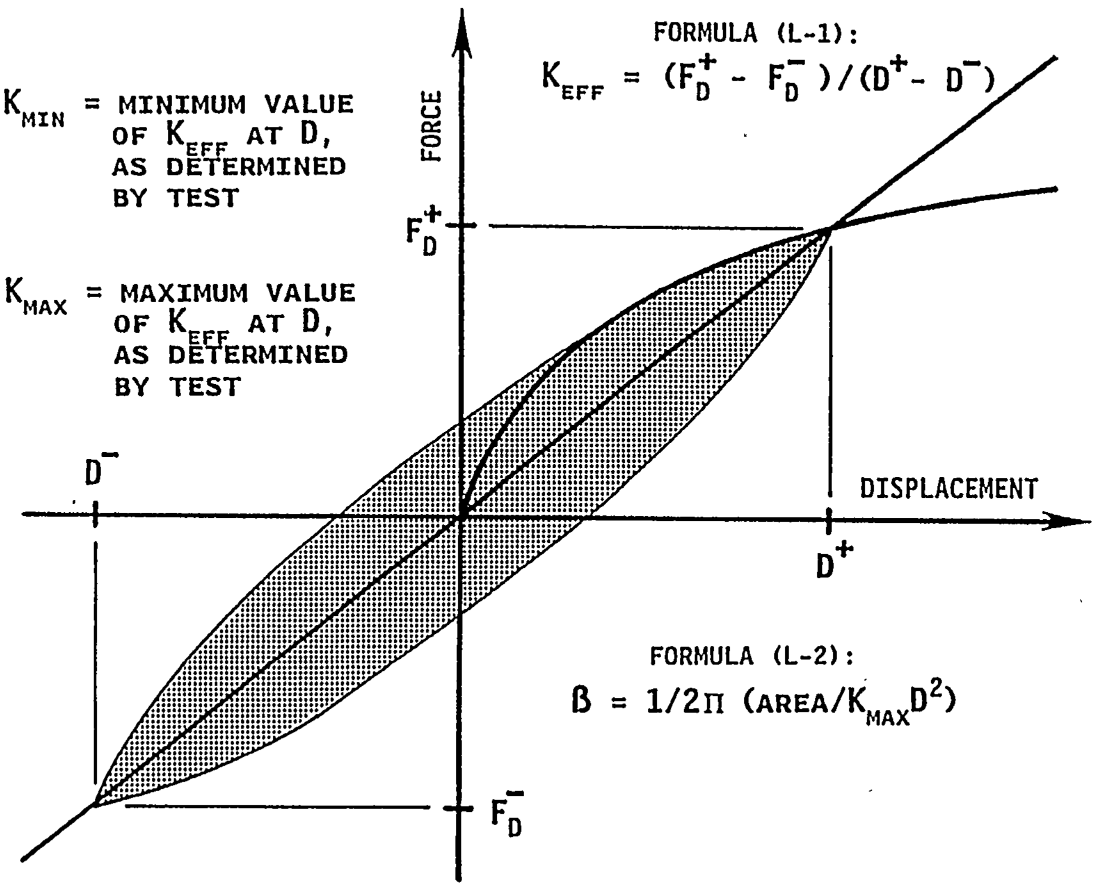
Charles Kircher & Associates
Consulting Engineers

STATIC ANALYSIS

EFFECTIVE STIFFNESS (k_{eff}) AND DAMPING (β)

K_{MIN} = MINIMUM VALUE OF K_{EFF} AT D, AS DETERMINED BY TEST

K_{MAX} = MAXIMUM VALUE OF K_{EFF} AT D, AS DETERMINED BY TEST



FORMULA (L-1):

$$K_{EFF} = (F_D^+ - F_D^-)/(D^+ - D^-)$$

FORMULA (L-2):

$$\beta = 1/2\pi\ (AREA/K_{MAX}D^2)$$

STATIC ANALYSIS

TOTAL DESIGN DISPLACEMENT (INCLUDING TORSION)

- Calculation to be based on actual plus 5% accidental eccentricity
- If the stiffness of the isolation system is uniformly distributed, the total design displacement, D_T, is specified by Formula (E-3), as follows:

$$D_T = D\left[1 + y\frac{12e}{b^2+d^2}\right]$$

$$\approx 1.15D - 1.3D$$

- The minimum value of the total design displacement is specified as follows:

$$D_T \geq 1.1D$$

STATIC ANALYSIS

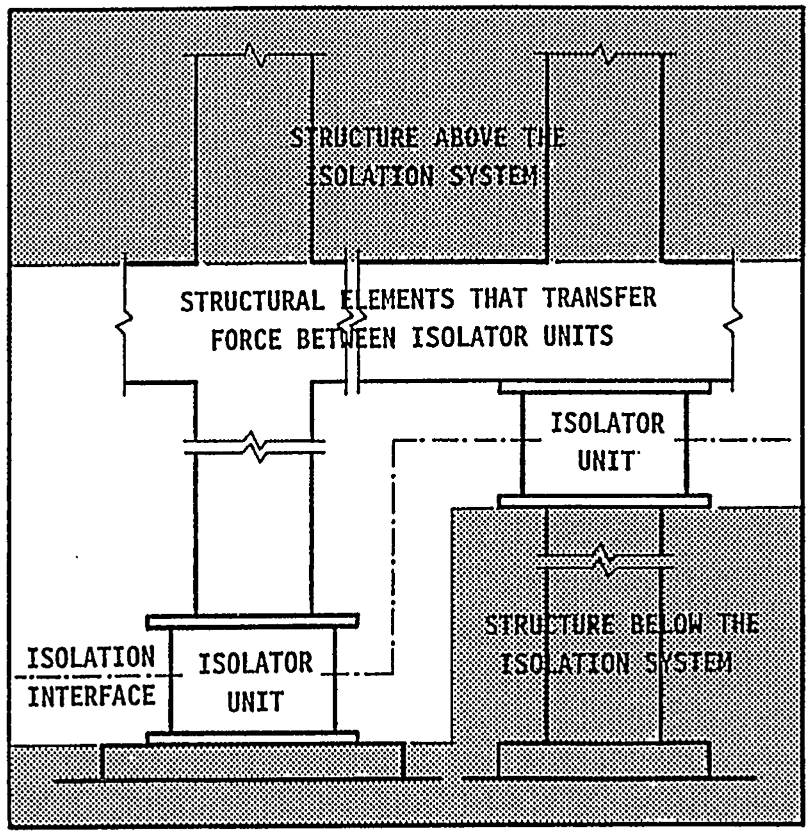
TOTAL MAXIMUM DISPLACEMENT, FORMULA (E-4)

The minimum value of the total maximum displacement, D_{TM}, required for verification of isolation system stability is prescribed as follows:

$$D_{TM} = 1.5 D_T$$

STATIC ANALYSIS

ISOLATION SYSTEM TERMINOLOGY



STATIC ANALYSIS

ISOLATION SYSTEM AND SUBSTRUCTURE DESIGN, FORMULA (E-5)

The lateral force or shear, V_B, required for working-stress design of the isolation system, and structural elements below the isolation system, is prescribed as follows:

$$V_B = \frac{k_{max} D}{1.5}$$

where: k_{max} = maximum effective stiffness of isolation system at the design displacement, D, as determined by prototype testing

STATIC ANALYSIS

SUPERSTRUCTURE DESIGN, FORMULA (E-6)

The lateral force or shear, V_s, required for working-stress design of the structure above the isolation system is prescribed as follows:

$$V_S = \frac{k_{max} D}{R_{wI}}$$

where: R_{wI} = design-force reduction coefficient, as specified for various types of lateral-force-resisting systems

STATIC ANALYSIS

COMPARISON OF LATERAL DESIGN FORCE FORMULAS: FIXED-BASE AND ISOLATED BUILDINGS

- **Fixed-Base Buildings:**

$$V = \frac{ZICW}{R_w} \qquad (1-1)$$

where:

$$C = \frac{1.25S}{T^{2/3}} \qquad (1-2)$$

- **Isolated Buildings:**

$$V_s = \frac{k_{max} D}{R_{wI}} \qquad (E-6)$$

where:

$$D = \frac{10ZNS_I T_I}{B} \qquad (E-1)$$

$$k_{max} = \frac{4\pi^2 W}{g T_I^2} \qquad (k_{max} = k_{min})$$

$$V = \frac{ZI(1.25)SW}{T^{2/3} R_w} \quad \longleftrightarrow \quad V_s = \frac{ZNS_I W}{T_I B R_{wI}}$$

STATIC ANALYSIS

COMPARISON OF LATERAL DESIGN FORCE FORMULA TERMS: FIXED-BASE AND ISOLATED BUILDINGS

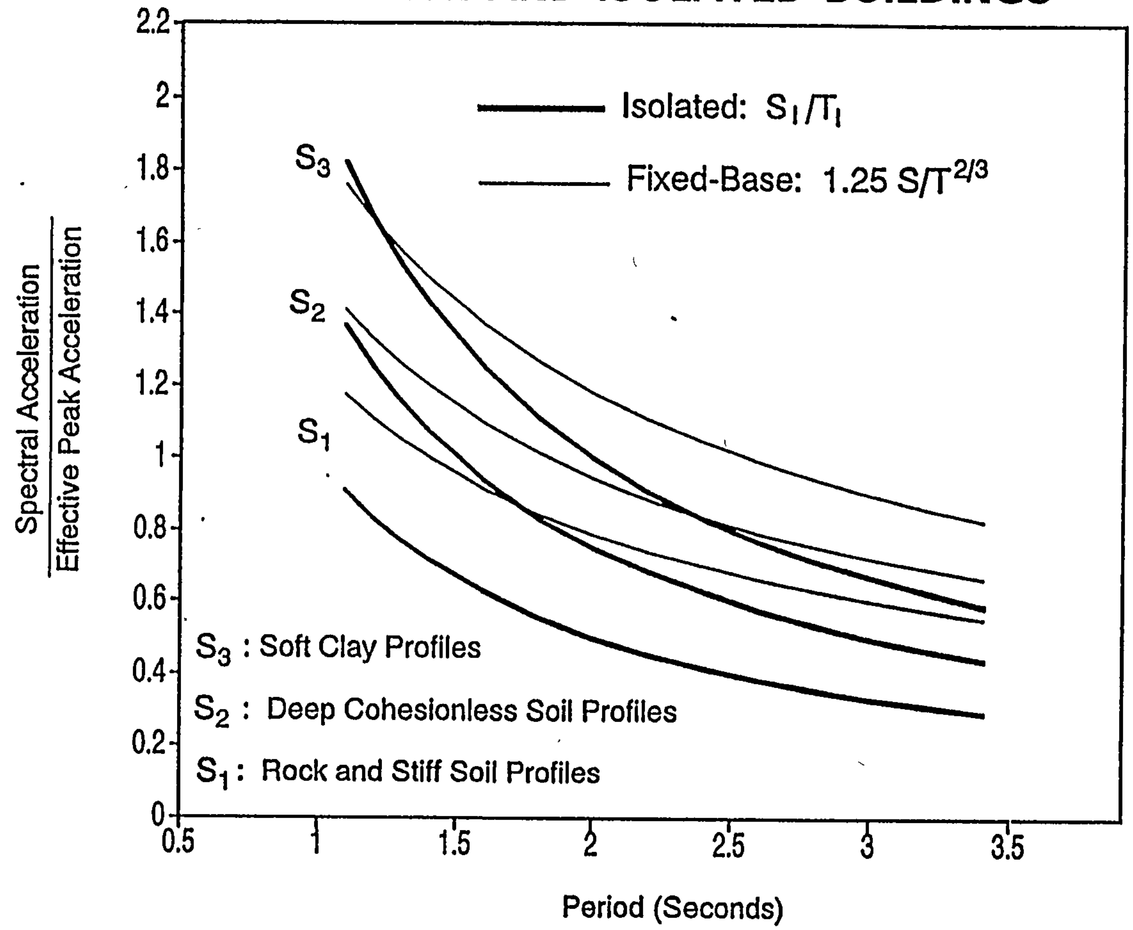
Lateral Design Force Formula Term	Fixed-Base Buildings	Isolated Buildings
Seismic Zone Factor	Z	Z
Importance Factor	I	(1.0)
Near-Fault Coefficient	(1.0)	N
Building Weight	W	W[1]
Lateral Force Coefficient (Spectral Shape Factor)	$1.25S/T^{2/3}$	S_I/T_I
Response Reduction Coefficient(s) (Knock-Down Factor)	$1/R_W$	$1/BR_{WI}$

1 Above the Isolation Interface

Charles Kircher & Associates
Consulting Engineers

STATIC ANALYSIS

COMPARISON OF DESIGN LATERAL FORCE COEFFICIENTS: FIXED-BASE AND ISOLATED BUILDINGS



STATIC ANALYSIS

COMPARISON OF RESPONSE REDUCTION COEFFICIENTS: FIXED-BASE AND ISOLATED BUILDINGS

STRUCTURAL SYSTEM	R_{wI} ISOLATED BUILDINGS	R_w FIXED-BASE BUILDINGS
Special Moment-Resisting Frame	3.0	12
Shear Wall	2.6	6
Braced Frame (Concentric)	2.2	8
Ordinary Moment-Resisting Frame	1.8	6

STATIC ANALYSIS

SUPERSTRUCTURE DESIGN LIMITS

- In all cases, the value of V_S shall not be less than any one of the following:

  1. The lateral seismic force required by the SEAOC/UBC provisions for a fixed-base structure of period, T_I

  2. The base shear corresponding to the design wind load

  3. The lateral force required to fully activate the isolation system

STATIC ANALYSIS

DISTRIBUTION OF FORCE, FORMULA (E-7)

The total force shall be distributed over the height of the structure above the isolation interface in accordance with the following formula:

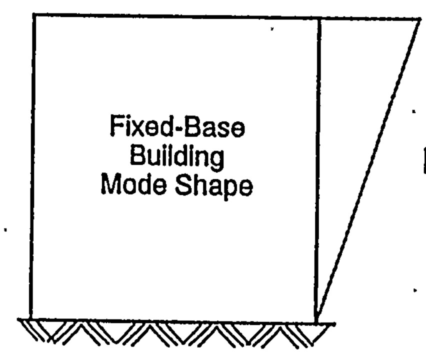
$$F_x = V_s \left[\frac{w_x}{\sum w_i} \right]$$

where: F_x = lateral force applied to level x

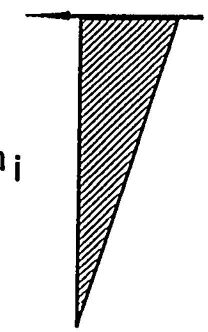
w_x, w_i = that portion of the total seismic dead load, W, assigned to level x or i, respectively

STATIC ANALYSIS

COMPARISON OF DESIGN LATERAL FORCE DISTRIBUTIONS: FIXED-BASE AND ISOLATED BUILDINGS

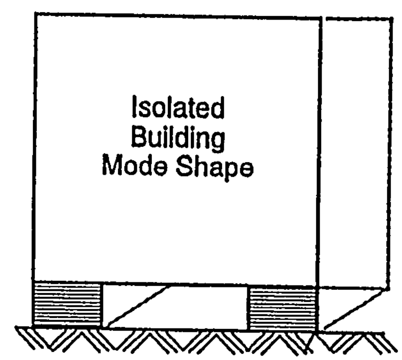


$F_t = 0.07TV$



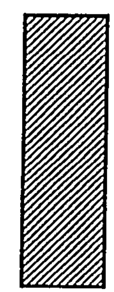
$$F_x = (V - F_t)w_x h_x / \sum_{i=1}^{n} w_i h_i$$

$$V = ZI(1.25)SW/T^{2/3} R_w$$



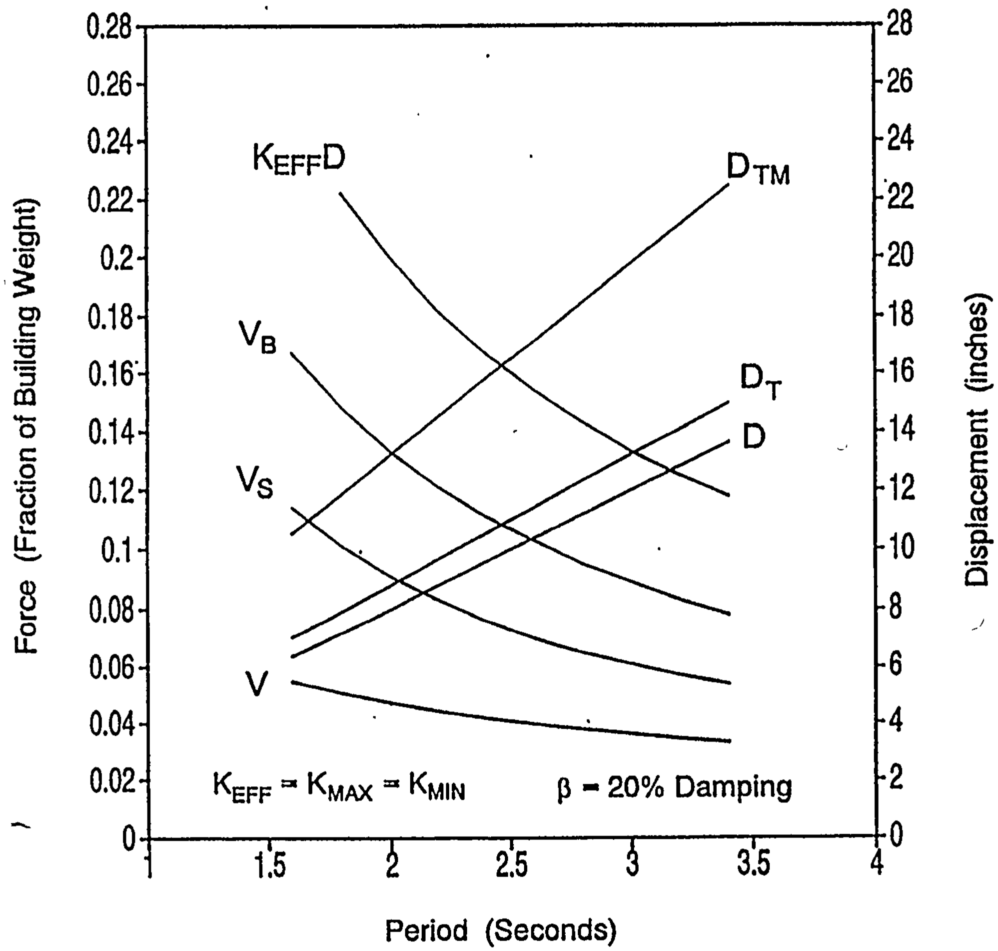
$$F_x = V_s w_x / \sum_{i=1}^{n} w_i$$

$$V_S = ZNS_I W/BT_I R_{WI}$$



STATIC ANALYSIS

DESIGN EXAMPLE: MINIMUM LATERAL FORCE AND DISPLACEMENT



Notes:

(1) Building assumed to be located in Seismic Zone 4, on Soil Type S_2, 15 km from active faults (ZNS = 0.6).

(2) Lateral force resisting system assumed to be a steel, concentric-braced frame.

DYNAMIC ANALYSIS

DESIGN SPECTRA

DESIGN LEVEL	WITH SITE-SPECIFIC HAZARD ANALYSIS	WITHOUT SITE-SPECIFIC HAZARD ANALYSIS
Design-Basis Earthquake (DBE)	≥ 0.8E	≥ 1.0E
Maximum Credible Earthquake	≥ 1.25DBE	≥ 1.25DBE

where: E is the normalized response spectrum of the SEAONC/UBC provisions, scaled by the appropriate soil type coefficient, S_I, and seismic zone coefficient, Z

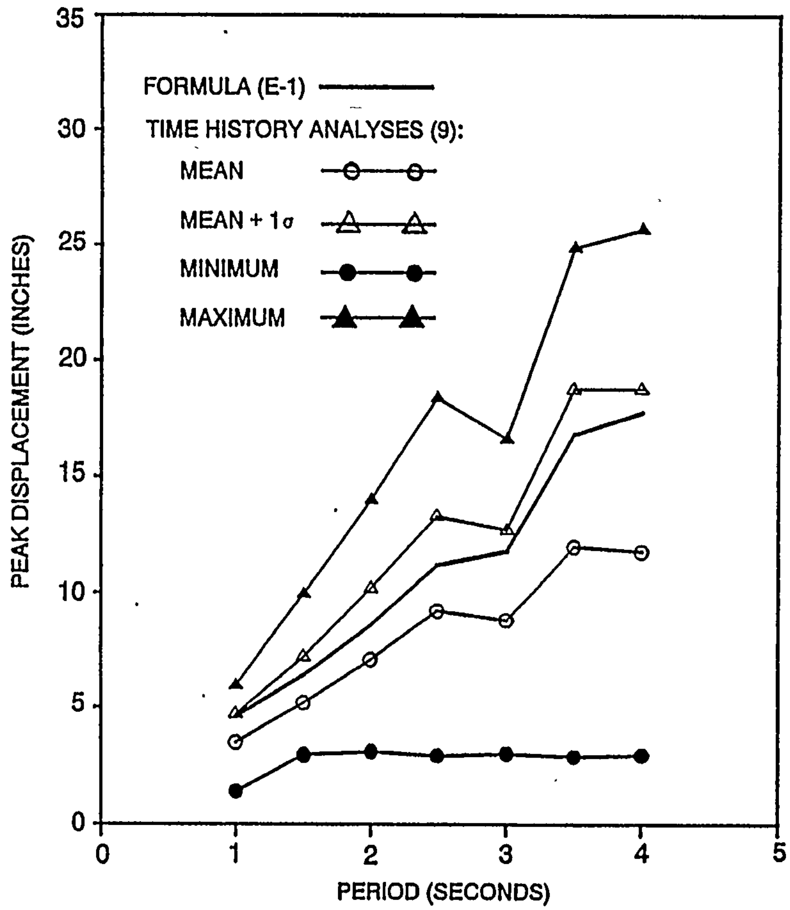
DYNAMIC ANALYSIS

DESIGN TIME HISTORIES

- At least three (3) pairs of horizontal components to be selected from recorded events
- Each pair of time history components to be scaled to match the design spectrum
- Time histories to be consistent with the duration of the design earthquake
- Time histories to incorporate near-fault phenomena for sites within 15 km of a major active fault

DYNAMIC ANALYSIS

COMPARISON OF PEAK DISPLACEMENT: STATIC FORMULA (E-1) AND TIME HISTORY ANALYSIS



Notes:

(1) Building assumed to be located in Seismic Zone 4, on Soil Type S_2, 15 km from active faults (ZNS = 0.6).

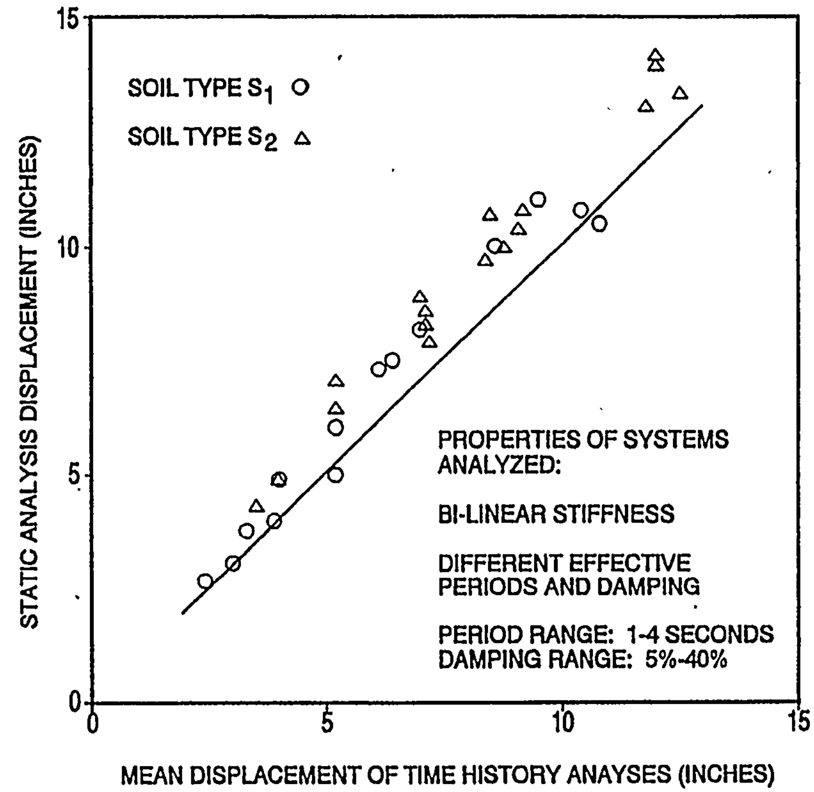
(2) Nine (9) pairs of recorded time history components were time - domain scaled such that the average value of their 5%-damped response spectra was approximately equal to assumed site conditions at long periods (i.e., 0.6/T).

(3) Isolation systems were modeled as bi-linear, hysteretic elements, 10% - 20% effective damping.

Charles Kircher & Associates
Consulting Engineers

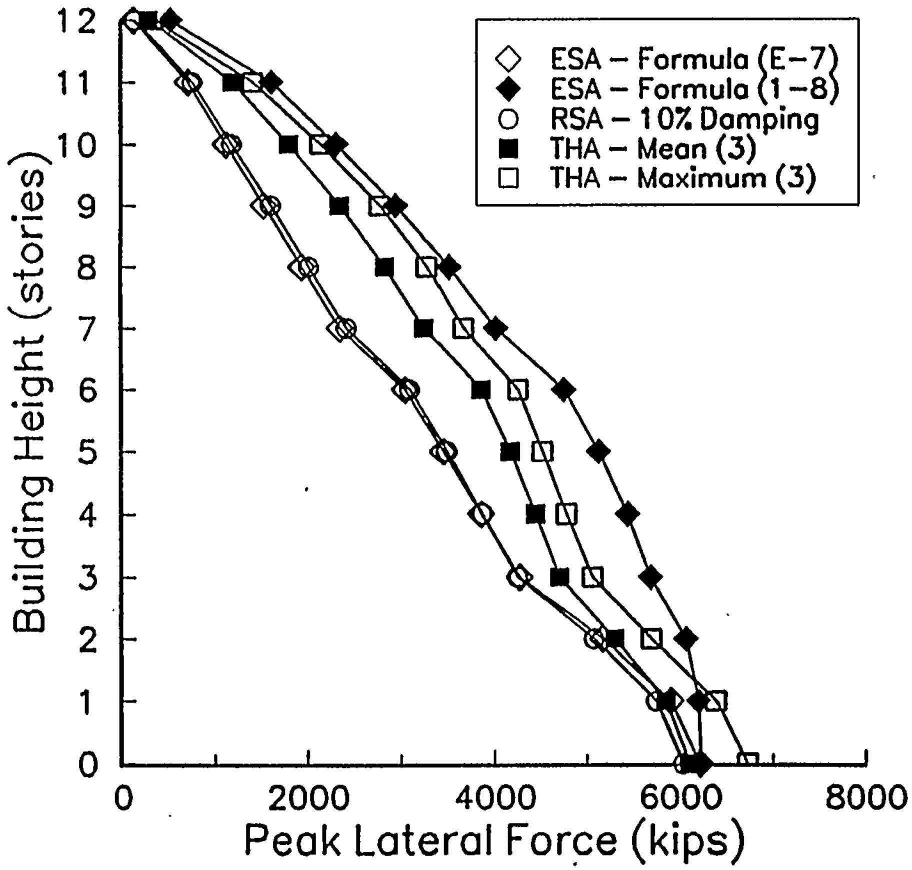
DYNAMIC ANALYSIS

COMPARISON OF PEAK DISPLACEMENT: STATIC AND TIME HISTORY ANALYSIS



DYNAMIC ANALYSIS

COMPARISON OF PEAK STORY SHEAR: STATIC (ESA), RESPONSE SPECTRUM (RSA) AND TIME HISTORY ANALYSIS (THA)



Notes

(1) Building assumed to be located in Seismic Zone 4, on Soil Type S_2, 15 km from active faults (ZNS = 0.6).

(2) Three (3) time histories were used for THA, each time-domain scaled to match ZNS/T = 0.6/T at long periods.

(3) Superstructure is an 11-story, reinforced-concrete shear wall system, approximately 25,000 kips total weight, with a fixed-base period of about 0.6 seconds.

(4) Effective period and damping of the isolated building are about 2.0 seconds and 10%, respectively.

DYNAMIC ANALYSIS

DESIGN LIMITS

Design Parameter	Static Analysis	Dynamic Analysis	
		Response Spectrum	Time History
Design Displacement	$D = 10ZNST/B$	-	-
Total Design Displacement	$D_T \geq 1.1\ D$	$\geq 0.9\ D_T$	$\geq 0.9\ D_T$
Total Maximum Displacement	$D_{TM} = 1.5\ D$	$\geq 0.8\ D_{TM}$	$\geq 0.8\ D_{TM}$
Design Shear (At or Below Isolation System)	$V_B = K_{MAX}D/1.5$	$\geq 0.9\ V_B$	$\geq 0.9\ V_B$
Design Shear ("Regular" Superstructure)	$V_S = K_{MAX}D/R_{WI}$	$\geq 0.8\ V_S$	$\geq 0.6\ V_S$
Design Shear ("Irregular" Superstructure)	$V_S = K_{MAX}D/R_{WI}$	$\geq 1.0\ V_S$	$\geq 0.8\ V_S$
Drift	$0.010/R_{WI}$	$0.015/R_{WI}$	$0.020/R_{WI}$

NONSTRUCTURAL COMPONENTS

NONSTRUCTURAL DESIGN REQUIREMENTS

- ABOVE THE ISOLATION INTERFACE - Design for calculated peak response reduced by 1.5 (or design using SEAOC/UBC requirements for fixed-base building components)
- ACROSS THE ISOLATION INTERFACE - Design for the total maximum displacement, D_{TM}
- BELOW THE INTERFACE - Design using SEAOC/UBC requirements for fixed-base building components

DETAILED SYSTEM REQUIREMENTS

DETAILED ISOLATION SYSTEM REQUIREMENTS

- ENVIRONMENTAL - Isolation system is to be designed considering the effects of aging, creep, fatigue, operating temperature, moisture and damaging substances
- WIND FORCES - Isolation system drift due to wind is limited to that permitted between floors of the superstructure
- FIRE RESISTANCE - Isolation system is to meet fire resistance requirements of the building's structure
- LATERAL RESTORING FORCE - Isolation system is to have a specified level of restoring force or to be designed for the greater of either 3.0 times the total design displacement, D_T, or 36ZNS

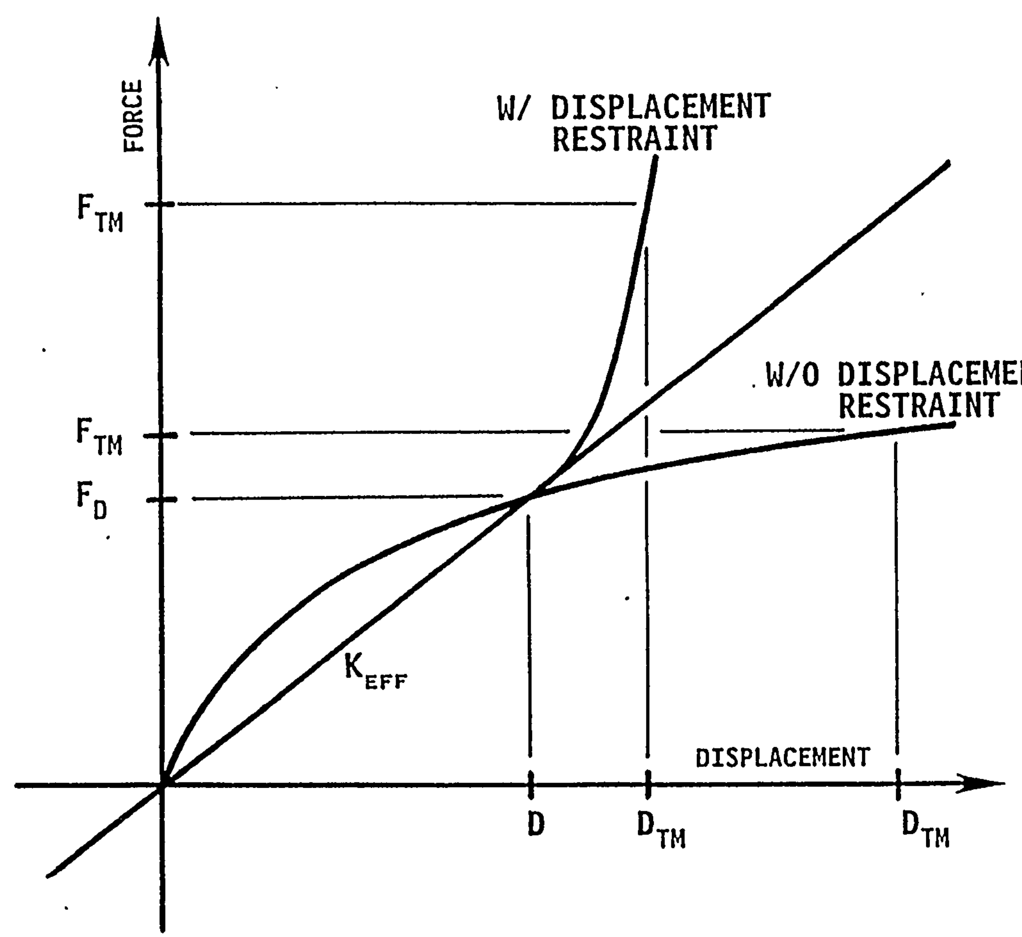
DETAILED SYSTEM REQUIREMENTS

DETAILED ISOLATION SYSTEM REQUIREMENTS (CONTINUED)

- DISPLACEMENT RESTRAINT - If displacement of the isolation system is "restrained" to less than 1.5 times the total design displacement, then the isolation system and superstructure are to be evaluated for loads corresponding to the maximum credible earthquake (MCE)
- INSPECTION AND REPLACEMENT - Access is to be provided for inspection and replacement of all isolation system components
- QUALITY CONTROL - A quality control testing program is required for fabrication of isolator units

DETAILED SYSTEM REQUIREMENTS

ISOLATION SYSTEM DISPLACEMENT RESTRAINT CONCEPT



DETAILED SYSTEM REQUIREMENTS

DETAILED STRUCTURAL SYSTEM REQUIREMENTS

- HORIZONTAL DISTRIBUTION OF FORCE - A diaphragm (or other structural elements) is required above the isolation interface to connect vertical-load-carrying elements and to provide rigidity at the base of the superstructure
- BUILDING SEPARATIONS - the minimum separation between an isolated building and other structures is to be not less than the total maximum displacement

DESIGN AND CONSTRUCTION REVIEW

PEER REVIEW

- Design review of the isolation system and related test programs is required for all isolation projects
- The review team is to include persons licensed in the appropriate disciplines and experienced in isolation theory and application
- Isolation system review topics include:

  (1) Site-specific seismic criteria

  (2) Preliminary design

  (3) Prototype testing

  (4) Final design and supporting analyses

  (5) Quality-control testing

ISOLATION SYSTEM TESTS

PROTOTYPE TESTS

- Prototype tests are required to confirm the force-deflection properties used for design and to verify the overall adequacy of the isolation system

- Prototype tests are to include two, full-size specimens of each type and size of isolator unit, as well as specimens of the wind restraint system, if such is used in the design

ISOLATION SYSTEM TESTS

PROTOTYPE TESTS (CONTINUED)

- Cyclic-Load tests are to be performed (with vertical load equal to typical DL) as follows:

  (1) 20 full cycles at the wind design force,

  (2) 3 full cycles at each increment: 0.25, 0.50, 0.75, and 1.0 times the total design displacement (D_T),

  (3) $15S_I/B$, but not less than 10, full cycles at 1.0 times the total design displacement (D_T)

- If isolator units support vertical load, the cyclic-load tests of Item (2) above are to be repeated for two additional load cases:

  (1) $1.2\ DL + 0.5\ LL + |E|$

  (2) $0.8\ DL - |E|$

ISOLATION SYSTEM TESTS

PROTOTYPE TESTS (CONTINUED)

- Additional cyclic-load tests may be required to quantify effects of rate of loading, direction of load, etc., on isolator properties
- Static-load tests are to be performed at the total maximum displacement (D_{TM}) of isolators for the following two load cases:

(1) $1.2\ DL + 1.0\ LL + |E|$

(2) $0.8\ DL - |E|$

TESTS OF THE ISOLATION SYSTEM

ISOLATION SYSTEM ACCEPTANCE CRITERIA

- Force-deflection plots show a positive incremental force-carrying capacity for all prototype tests
- Effective stiffness does not vary appreciably for isolator units of a common type and size
- Effective stiffness does not degrade appreciable during the 10+ cycles of load at 1.0 times the total design displacement (D_T)
- All specimens remain stable at the total maximum displacement (D_{TM}) under full vertical load

LLNL/DOE SHORT COURSE OF SEISMIC (BASE) ISOLATION

DESIGN PROCESS FOR ISOLATED STRUCTURES

BY

Charles A. Kircher Ph.D., P.E.

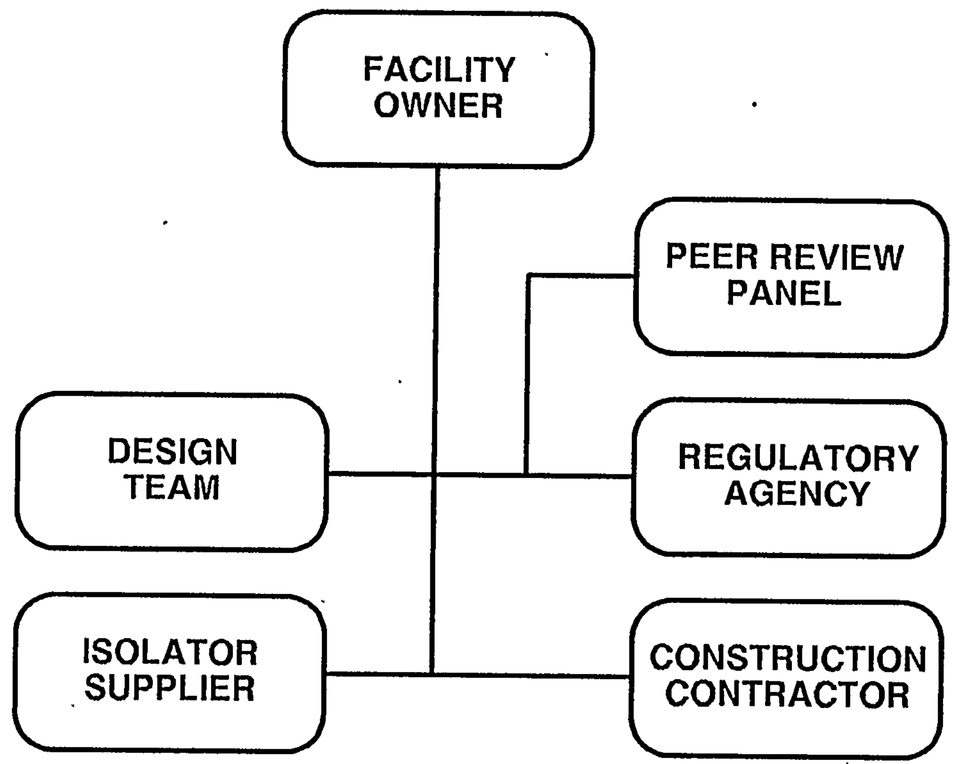
August 11, 1992

<u>Design Process for Isolated Structures</u>

PRESENTATION TOPICS

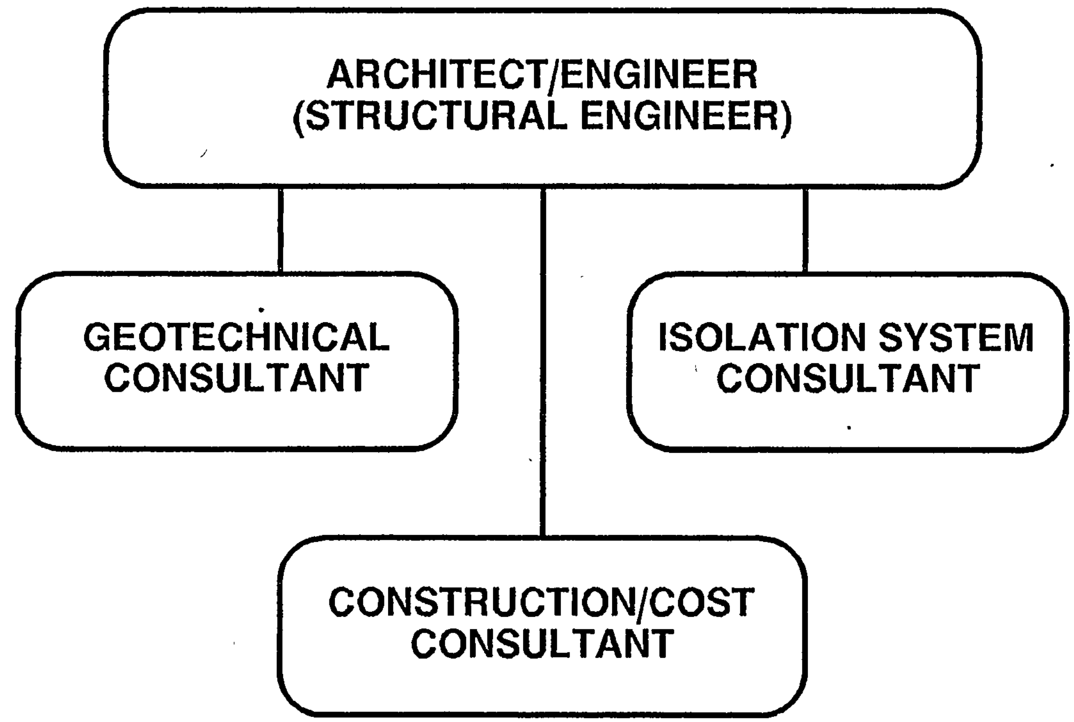
Design Process for Isolated Structures

OVERALL PROJECT ORGANIZATION



Design Process for Isolated Structures

KEY DESIGN TEAM MEMBERS



Design Process for Isolated Structures

ELEMENTS OF THE BUILDING DESIGN PROCESS

- SPECIAL OR CONCEPT STUDIES
- DESIGN PHASES
  - Programming (Pre-Design) Phase - Establish configuration and layout of building
  - Schematic (Concept Design) Phase - Develop design scheme(s) and alternatives
  - Design Development (Tentative Design) Phase - Develop preliminary design and prepare preliminary design drawings
  - Construction (Bid Document) Document Phase - Revise preliminary design and prepare final drawings and specifications for bid
- DESIGN REVIEW ACTIVITIES
  - Owner Review and Approval - At end of each design phase plus additional approval points during Design Development
  - Peer Review/Value Engineering, etc.

OVERVIEW OF ISOLATION DESIGN PROCESS
(Competitive Source Selection - GSA Approach[1])

- ♦ SCHEMATIC PHASE
  - Establish seismic/structural design criteria (with site hazard input from a geotechnical study)
  - Identify feasible isolation system alternatives (i.e., 3 to 4 isolation systems with established properties and prior use on at least two jobs)
  - Develop structural design concept for each isolation system alternative
- ♦ DESIGN DEVELOPMENT
  - Develop tentative design and prepare drawings for each structural concept (i.e., for each isolation system alternative)
- ♦ CONSTRUCTION DOCUMENTS PHASE
  - Prepare "Contract Bid Set" drawings, specifications, etc., for tentative design approved by GSA
- ♦ SOURCE SELECTION
  - Develop criteria (including isolation system specification) and assist with selection of vendor during schematic and design development phases

1. General Services Administration: (1) U.S. Court of Appeals, 7th and Mission, San Francisco, and (2) Federal Building, 50 United Nations Plaza, San Francisco.

<u>Design Process for Isolated Structures</u>

OVERVIEW OF ISOLATION DESIGN PROCESS
(Competitive Bid - SBC Approach[1])

- ♦ SCHEMATIC PHASE
  - Establish/adopt preliminary seismic/structural design criteria (with tentative site hazard input from a geotechnical study)
  - Develop preliminary structural/isolation system design
  - Develop isolation bid document:
    - isolator sizes based on performance specified for preliminary design
    - lump-sum bid based on unit prices and preliminary isolator sizes
    - vendor commits to unit prices
- ♦ DESIGN DEVELOPMENT
  - Select isolation system vendor
  - Finalize design criteria (after State approval of site hazard study)
  - Revise preliminary designs and drawings based on final criteria
- ♦ CONSTRUCTION DOCUMENTS PHASE
  - Prepare final design drawings, specifications, etc.

1. San Bernardino County Medical Center Replacement Project

Design Process for Isolated Structures

OVERVIEW OF ISOLATION DESIGN PROCESS
(Competitive Source Selection - OSA Approach[1])

- SCHEMATIC PHASE
  - Establish seismic/structural design criteria (with site-specific hazard)
  - Develop structural/isolation system concept
  - Develop isolation vendor request for qualifications (RFQ)
- DESIGN DEVELOPMENT
  - Develop preliminary isolation system designs and structural details based on information provided with vendor RFQ responses
  - Analyze isolation system alternatives and present findings (i.e., Pros/Cons, costs, etc.) to State Selection committee
  - Prepare isolation system contract documents for State negotiations with top-ranked vendor
- CONSTRUCTION DOCUMENTS PHASE
  - Revise preliminary design/analyses
  - Prepare final design drawings, specifications, etc.

1. Office of the Architect, State of California Justice Building, 350 McAllister Street, San Francisco.

STATE OF CALIFORNIA JUSTICE BUILDING

Project Organization

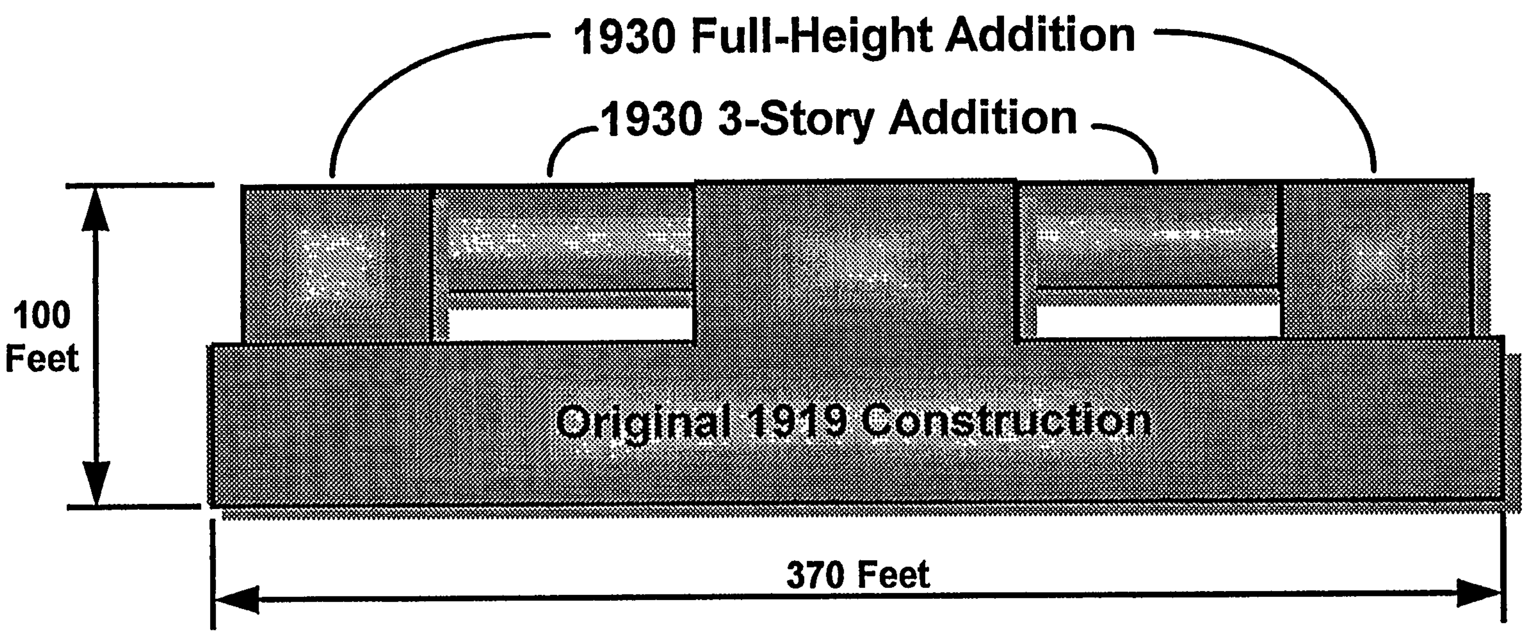
- OWNER:
  - OSA Project Manager - Kathryn O'Shea
  - OSA Consulting Manager - William Richardson
- DESIGN TEAM MEMBERS:
  - Architect - Esherick Homsey Dodge and Davis
  - Structural Engineer - Rutherford & Chekene
  - Geotechnical Consultant (Hazard) - Dames & Moore
  - Cost Estimating - Adamson Associates
  - Isolation Consultant - Kircher & Associates
- Peer Review:
  - H. Patrick Campbell (Chairman) - Chief Structural Engineer, OSA

STATE OF CALIFORNIA JUSTICE BUILDING

- BACKGROUND
  - The SCJB provides office and support space for the State Supreme Court, the First District Court of Appeal, State Police and State officials
  - The SCJB was closed following the Loma Prieta earthquake of October 1989 and will be fully renovated before being returned to service
  - In accordance with Senate Bill 920 (Rogers, Seismic Safety Building Design), the State Architect selected the SCJB to be the first state-owned building to be seismic retrofitted with base isolation technology
  - Preliminary design is complete, prospective isolation system suppliers have been evaluated and ranked; final design is pending notice to proceed by the State
- DESCRIPTION OF BUILDING
  - The building, located at 350 McAllister Street, is part of the Civic Center district of San Francisco and is of historical significance
  - The building was designed in 1915 by the architectural firm of Bliss and Faville and constructed in 1919 - 1920
  - In 1930, the building was enlarged by the addition of two full-height segments at each end of the building, making the building E-shaped in plan, and by partial 3-story infills between the legs of the "E"

Design Process for Isolated Structures

STATE OF CALIFORNIA JUSTICE BUILDING



PLAN VIEW - LOCATION OF 1919 AND 1930 CONSTRUCTION

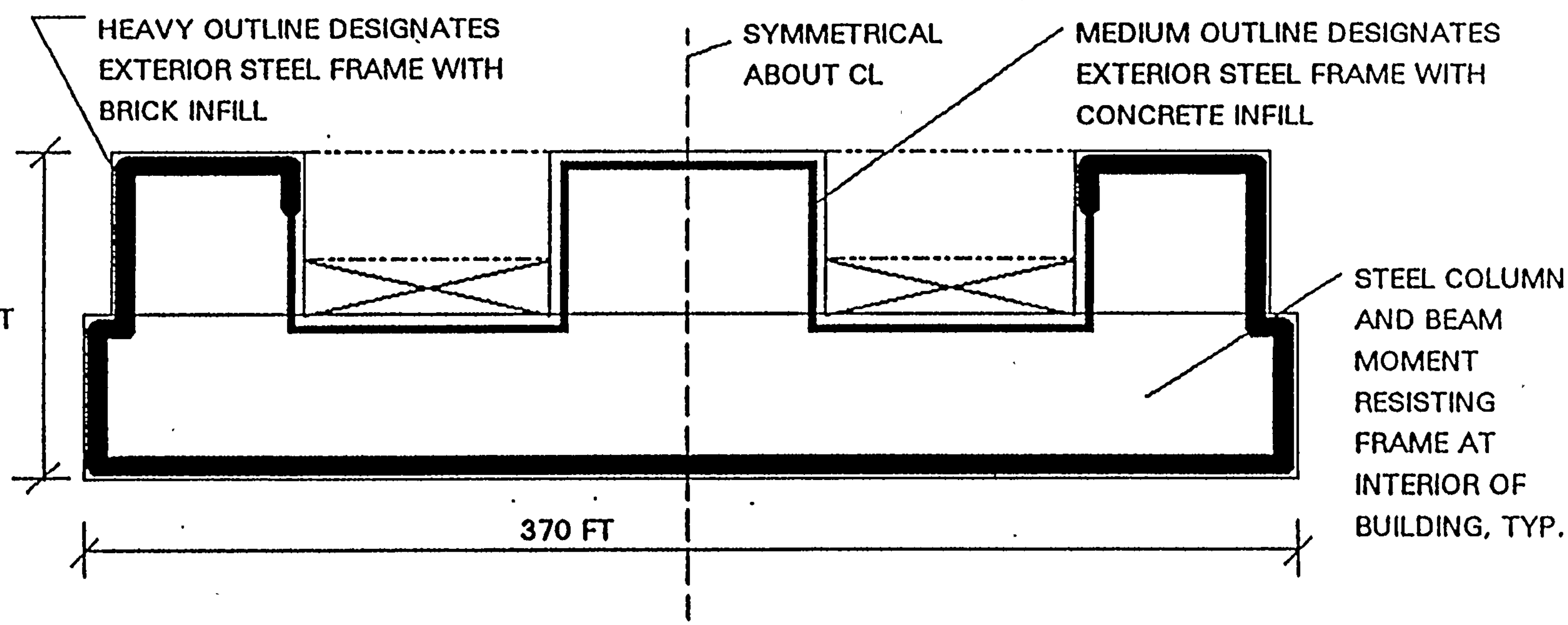
STATE OF CALIFORNIA JUSTICE BUILDING

- ♦ DESCRIPTION OF STRUCTURE
  - Structure is five stories in height with a full basement, approximately 100 feet by 370 feet in plan
  - Structural system consists of a vertical-load-supporting steel frame with unreinforced-masonry and concrete infill walls that resist lateral force
- ♦ DESCRIPTION OF RENOVATION WORK
  - Partial, 3-story segments of building will be demolished and full-height additions constructed to form an essentially rectangular building in plan
  - HVAC, fire and other nonstructural systems will be fully upgraded
- ♦ DESCRIPTION OF SEISMIC RETROFIT
  - Existing foundations will be strengthened and new retaining walls constructed around the building's basement
  - Existing basement columns and walls will be cut and seismic isolators will be installed just above the foundation
  - Existing basement elements will be strengthened and a new basement floor will be constructed
  - Superstructure of building will be strengthened by the addition of new reinforced-concrete shear walls at perimeter and interior locations

Charles Kircher & Associates
Consulting Engineers

Design Process for Isolated Structures

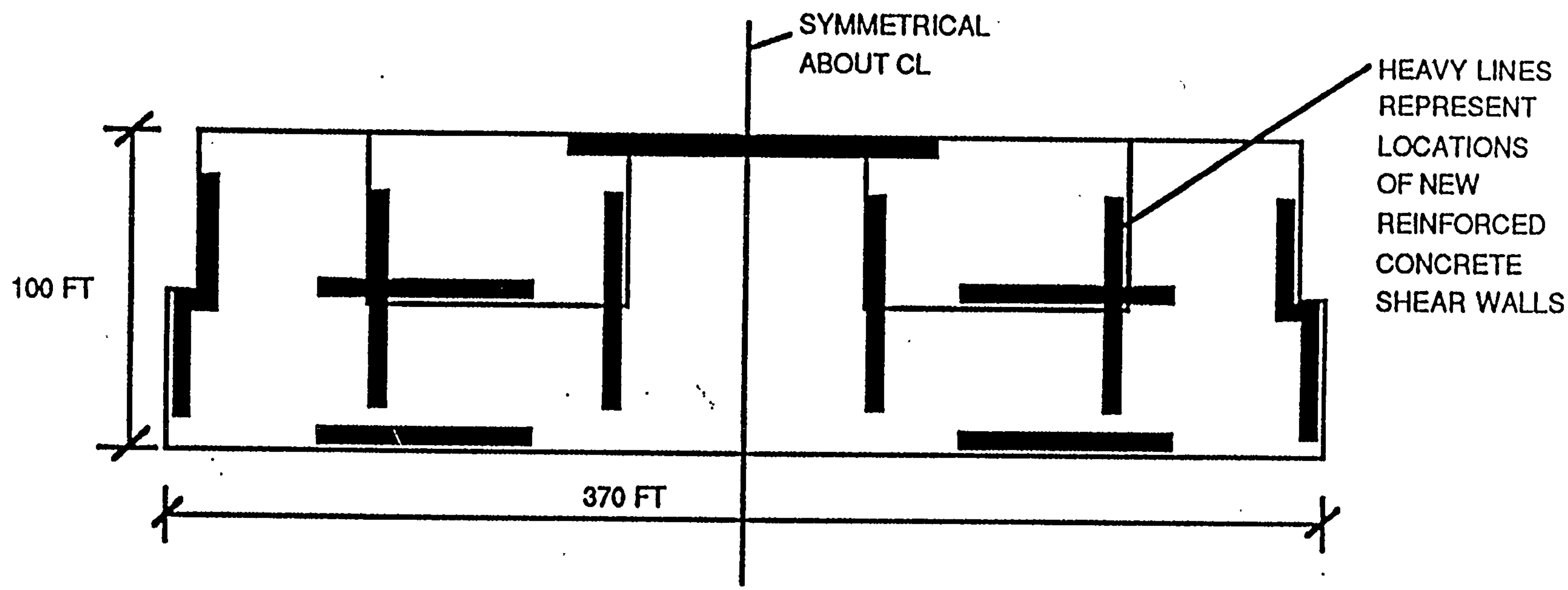
STATE OF CALIFORNIA BUILDING



PLAN VIEW - LOCATION OF URM AND CONCRETE INFILL WALLS

Design Process for Isolated Structures

STATE OF CALIFORNIA BUILDING



PLAN VIEW - LOCATION OF NEW REINFORCED-CONCRETE SHEAR WALLS

Design Process for Isolated Structures

STATE OF CALIFORNIA JUSTICE BUILDING

Performance Objectives

Performance Objective	Probable Earthquake (50% in 50 years)	Design Basis Earthquake (10% in 50 years)	Maximum Credible Earthquake (M 8.0, San Andreas)
Life-Safety Protection	FULL	FULL	SUBSTANTIAL, WILL NOT SIGNIFICANTLY JEOPARDIZE LIFE
Functionality	IMMEDIATE OCCUPANCY; POST-EARTHQUAKE FUNCTION RESUMED WITHIN HOURS	POST EARTHQUAKE FUNCTION RESUMED WITHIN DAYS TO MONTHS	BUILDING MAY BE CLOSED INDEFINITELY
Damage Protection	NO STRUCTURAL DAMAGE, SOME NONSTRUCTURAL CLEAN-UP REQUIRED	REPAIRABLE DAMAGE, MINOR STRUCTURAL AND NONSTRUCTURAL DAMAGE	SIGNIFICANT STRUCTURAL AND NONSTRUCTURAL DAMAGE, MAY NOT BE REPAIRABLE

<u>Design Process for Isolated Structures</u>

STATE OF CALIFORNIA JUSTICE BUILDING

Key Design Parameters

- Rubber Isolators (e.g., HDR or LR):

  (1) Effective stiffness, expressed as a function of lateral displacement (or rubber strain)
  (2) Effective damping, expressed as a function of lateral displacement (or rubber strain)
  (3) Vertical-load capacity, expressed as a function of lateral displacement

- Sliding Isolators (e.g, FPS):

  (1) Dynamic (sliding) coefficient of friction, expressed as a function of vertical load
  (2) Static (breakaway) coefficient of friction, expressed as a function of vertical load
  (3) Restoring force stiffness (i.e., radius of curvature of FPS sliding surface).

Design Process for Isolated Structures

STATE OF CALIFORNIA JUSTICE BUILDING

Preliminary Performance Specification

- Performance vs. Material/Fabrication Requirements
- Minimum Performance Requirements:

  (1) Basic Configuration and Dimensions

  (2) Minimum Vertical-Load Capacity (at Design Displacement)

  (3) Acceptable Range of Effective Stiffness

  (4) Acceptable Range of Effective Damping

- Prototype Testing Requirements

Design Process for Isolated Structures

STATE OF CALIFORNIA JUSTICE BUILDING

Isolator Suppliers/Fabricators[1]

ISOLATOR TYPE	SUPPLIER/FABRICATOR
High Damping Rubber (HDR) Isolators	LTV Energy Products Company Oil States Industries Division Arlington, Texas FURON - Structural Bearing Division Athens, Texas Dynamic Rubber, Inc. Athens, Texas Bridgestone Engineered Products, Inc. Huntington Beach, California
Lead Rubber (LR) Isolators	Dynamic Isolation Systems, Inc. Berkeley, California
Friction Pendulum System (FPS) Isolators	Earthquake Protection Systems, Inc. San Francisco, California/ VSL Corporation, San Jose, California

1. Experienced isolator suppliers/fabricators with offices in the United States.

Design Process for Isolated Structures

STATE OF CALIFORNIA JUSTICE BUILDING

Isolator Supplier/Fabricator Evaluation Topics

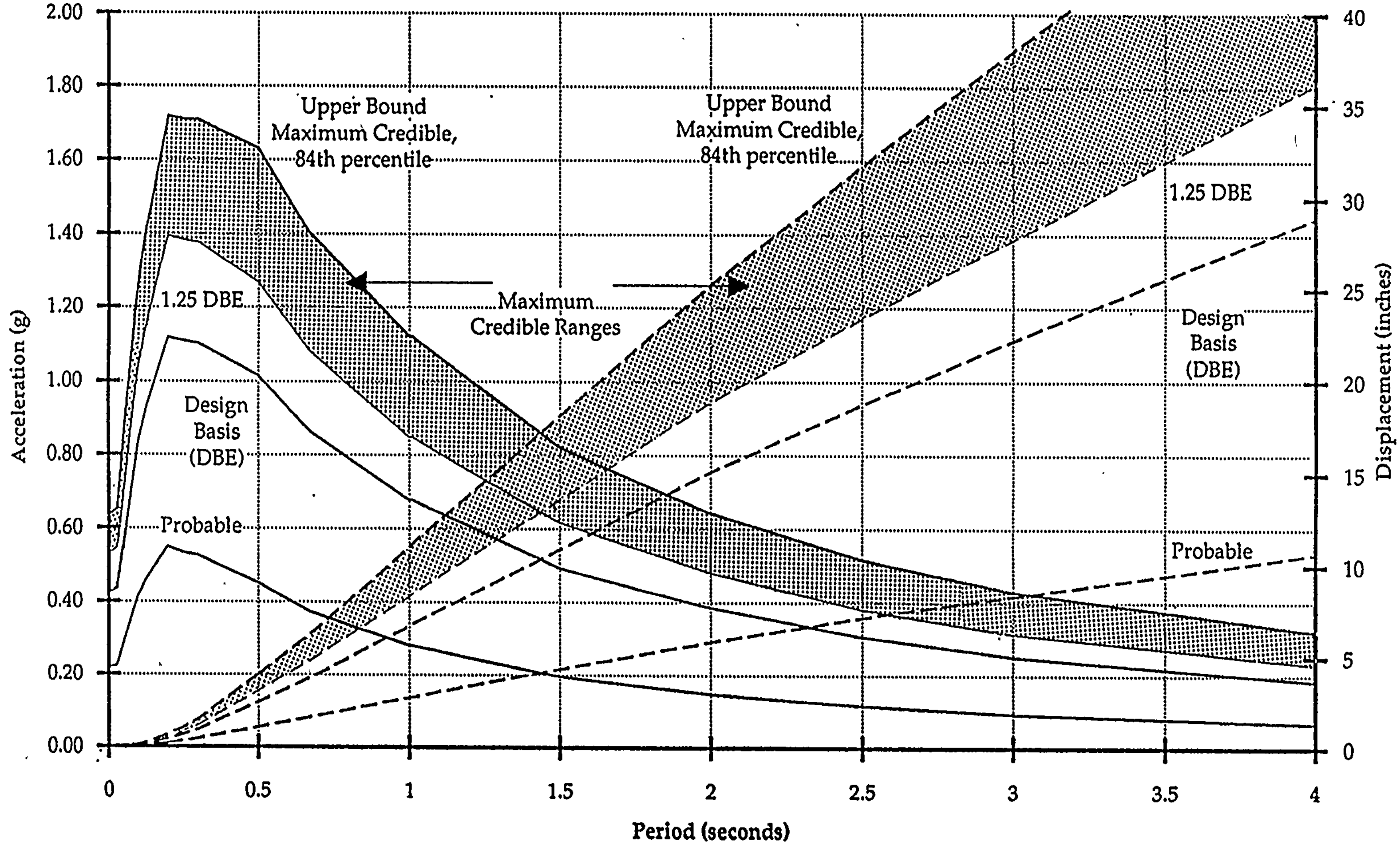
AREA	TOPIC
Technical	Isolated Structure Performance Isolator Properties - Design Basis Isolator Properties - Long Term
Project Personnel	Isolator Designer/Consultant Isolator Fabricator
Company (Fabricator)	Overall Manufacturing Capability Isolator Production Capacity Quality Control/Product Warranty

Charles Kircher & Associates
Consulting Engineers

Design Process for Isolated Structures

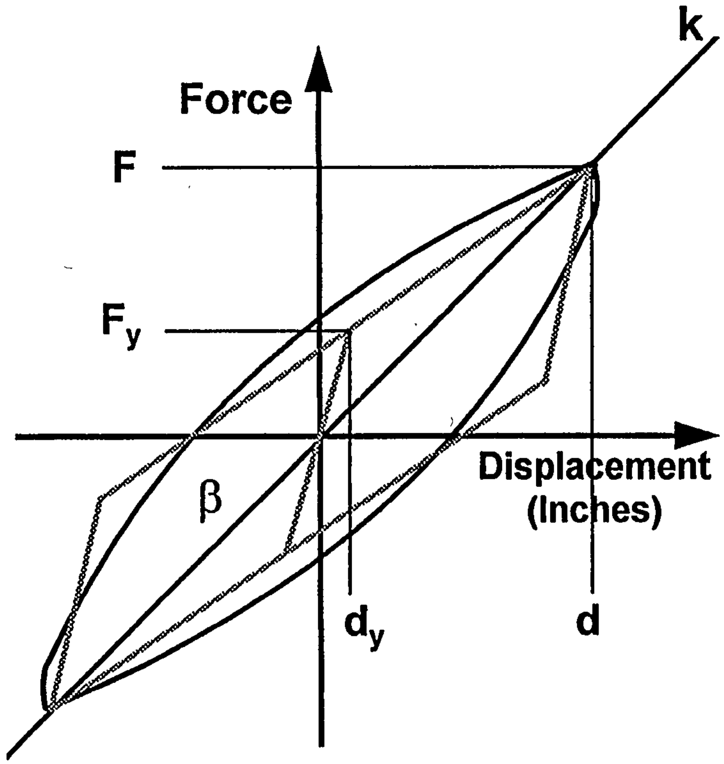
STATE OF CALIFORNIA JUSTICE BUILDING

Design Response Spectra - 5% Damping



<u>Design Process for Isolated Structures</u>

BI-LINEAR CHARACTERIZATION OF FORCE-DEFLECTION CURVE



NONLINEAR SYSTEM
(Viscous Plus Hysteretic Damping)

Effective Period:

$$T = 0.32\sqrt{\frac{d}{a}}$$

Effective Damping:

$$\beta = \frac{0.637(a_y d - d_y a)}{ad}$$

$$a_y = \frac{F_y}{W} \qquad a = \frac{F}{W}$$

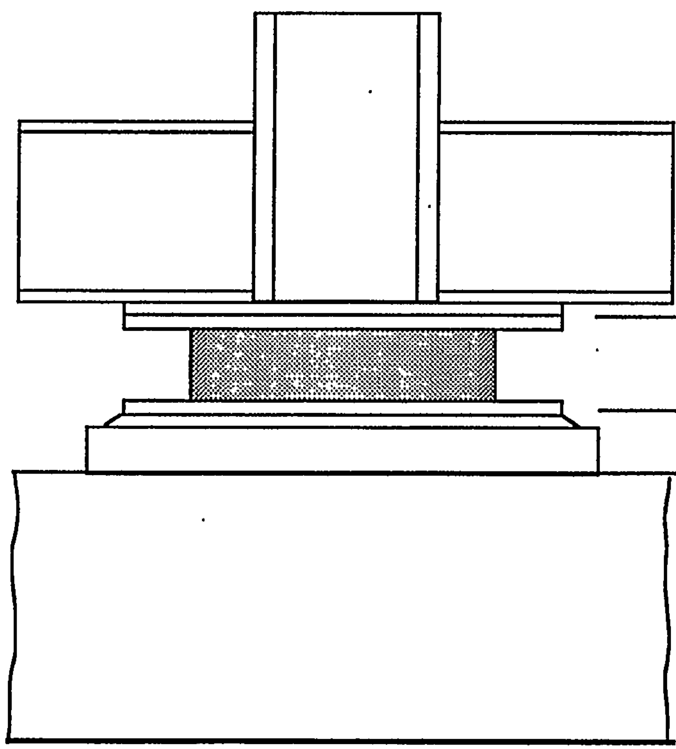
Design Process for Isolated Structures

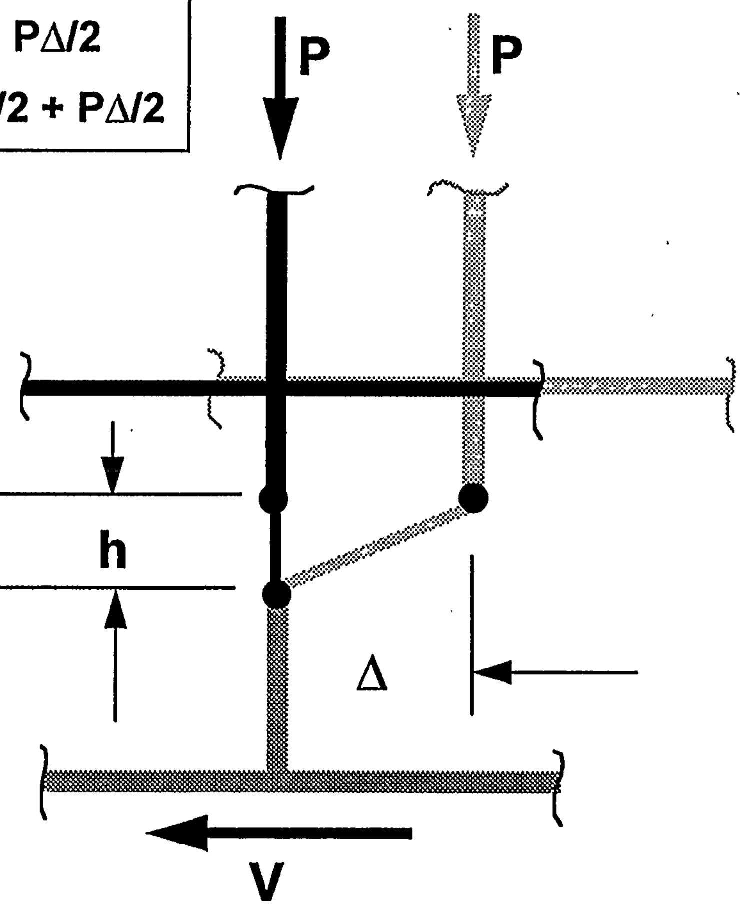
STATE OF CALIFORNIA JUSTICE BUILDING

Connection Details - HDR or LR Bearings Bolted to Structure

MOMENT AT TOP OF BEARING: $Vh/2 + P\Delta/2$

MOMENT AT BOTTOM OF BEARING: $Vh/2 + P\Delta/2$





Charles Kircher & Associates
Consulting Engineers

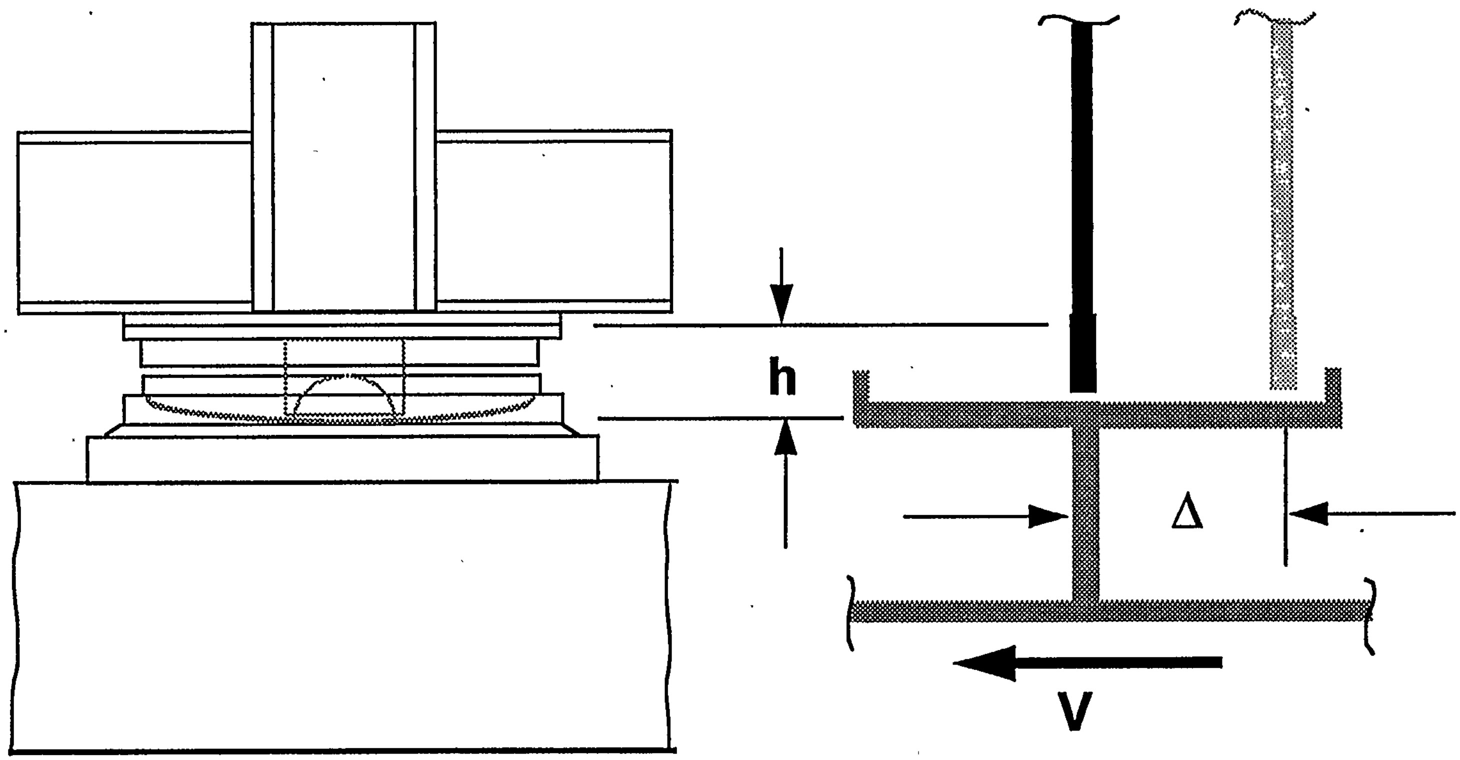
Design Process for Isolated Structures

STATE OF CALIFORNIA JUSTICE BUILDING

Connection Details - FPS Bearings

MOMENT AT TOP OF BEARING: Vh

MOMENT AT BOTTOM OF BEARING: PΔ



Design Process for Isolated Structures

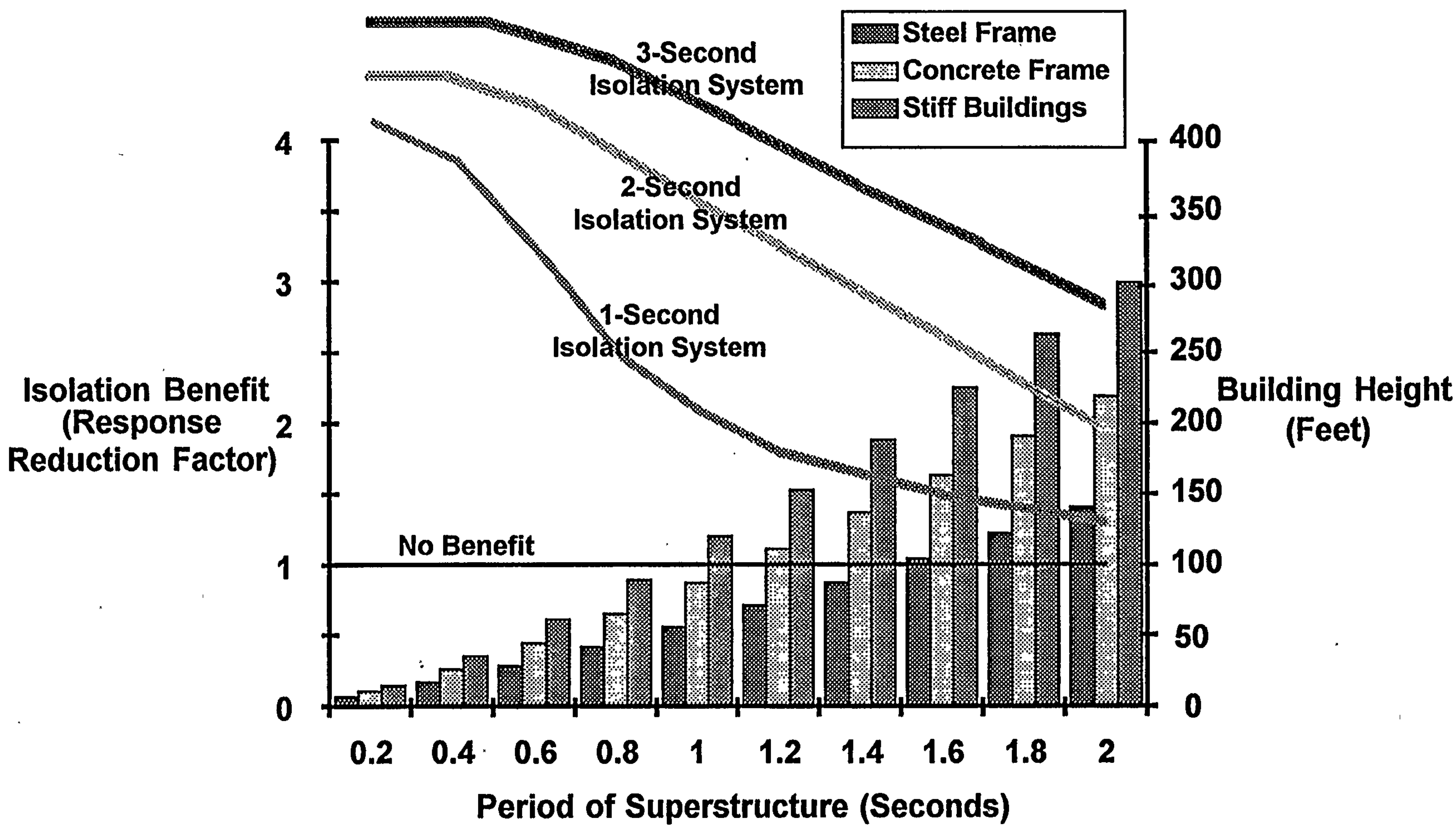
STATE OF CALIFORNIA JUSTICE BUILDING

Technical Performance Issues

ISSUE	HDR Isolators	LR Isolators	FPS Isolators
Design-Basis Period (seconds)	1.5 - 3.5	1.5 - 2.5	1.5 - 3.5
Design-Basis Damping (% of critical)	10% - 15%	10% - 25%	15% - 30%
Higher-Mode Effects	Best	OK	OK
Torsional Effects (additional response at edge of structure)	10% - 30%	10% - 30%	0% - 10%
Stability (factor of safety)	≥ 3.0 ($D = 0$) ≈ 1.0 (D_{TM})	≥ 3.0 ($D = 0$) ≈ 1.0 (D_{TM})	≈ 2.0
Isolator Clearance (height, h, inches)	8 - 24	12 - 24	6 - 12
Localized Load on Superstructure	OK	OK	Best
Localized Load on Foundation	OK	OK	OK

Design Process for Isolated Structures

APPROXIMATE RELATIONSHIP BETWEEN ISOLATION BENEFIT AND SUPERSTRUCTURE PERIOD



LECTURE 6

Experimental Work on Base Isolation

by

James M. Kelly
Professor of Civil Engineering
Earthquake Engineering Research Center
University of California
Berkeley, California 94720

Short Course on Seismic Base Isolation
Department of Energy

August 10 - August 14, 1992

J.M. Kelly

August 10-14, 1992

LECTURE 6

Experimental Work on Base Isolation

EARTHQUAKE SIMULATOR STUDIES

Introduction

The concept of base isolation is now well accepted and the number of buildings using this earthquake resistant technique is increasing rapidly. It is now generally accepted that a base isolated building will perform better than a conventional fixed base building in moderate or strong earthquakes. Several isolated buildings have experienced moderate earthquakes and have performed well. The major benefit of isolation, in the buildings in which it has been used so far, has been to reduce the effects of seismic forces on contents and internal equipment. In structures such as nuclear power plants, hospitals, emergency preparedness centers, computer manufacturing facilities and data centers, the reduction of damage to equipment is of sufficient importance to justify the increased cost of isolated construction. There are several examples of these types of base-isolated buildings in the United States, including both new and rehabilitated buildings and many more are in the design or construction stage. Additionally, the number of base isolated buildings worldwide is increasing with Japan in the forefront. The number of base isolated buildings in Japan either completed, under construction or approved for construction by the Ministry of Construction now exceeds sixty-five.

Whereas the concept of base isolating structures from the damaging effects of earthquake motions is not new, the implementation of this technique is relatively new. This has mainly been due to the need for several important developments in materials science and experimental and analytical modeling before base isolation could evolve into a practical approach for seismic design.

The objective of experimental research in base isolation at the Earthquake Engineering Research Center (EERC) in recent years has been to verify the performance, reliability and repeatability of behavior of components for isolation systems; to develop methods for modeling system behavior for design purposes; to investigate unusual aspects of behavior of various systems (especially during extreme conditions of loading), and from the wealth of data available, to examine the suitability of proposed code provisions to accurately predict behavior for design.

One of these developments has been the ability to test large scale isolation systems using simulated seismic loads. Performance evaluation studies using the earthquake simulator to test large-scale model isolated structures have been carried out for a variety of isolation systems and structures. Uplift studies of slender base-isolated buildings and the investigation of the behavior of base-isolated skew bridge decks has also been studied.

To date, applications of base isolation as a seismic design strategy for buildings have been restricted to low aspect-ratio structures for which there is no chance of overturning occurring during extreme ground shaking. In most instances, this limitation does not affect designers; generally, the taller the structure, the less practical it is to use base isolation. Building slenderness has some relation to period but, more important, it plays a large part in determining whether a building will uplift off its base during extreme lateral loadings and this is not something that is accommodated by usual base isolation devices.

An extensive research program was undertaken to address the question of uplift of base-isolated medium-rise structures. The work was conducted in two phases: the first involved shaking table studies of a medium-rise structure with a base isolation system allowing free uplift and a detailed evaluation of the behavior of the structure during uplift; the second phase of the program was to consider approaches which prevent uplifting of the superstructure from the isolation system.

The promising results of the previous test program prompted a series of tests of individual bearings at EERC with the purpose of showing that with properly designed and manufactured isolators it is possible to ensure that the isolation system will not fail before the structure of the isolated building above it fails. Thus, the ultimate state of the overall structure will be the failure of the structural frame (as in a conventional structure), but the input level of earthquake motion that will produce this failure will be very much larger than that needed to fail the same structural frame if it were conventionally based.

Several aspects of the ultimate capacity of an isolation system were examined. The results show that if an isolation system uses high strength elastomers (such as natural rubber or polychloroprene rubber) and high quality bonding techniques, the ultimate capacity can be accurately predicted and very substantial safety margins can be established. These results and other related work on isolators conducted under this program should lead to confidence in their use by designers of all types of structures, including nuclear facilities.

An extensive research program was undertaken to address the question of uplift of base-isolated medium-rise structures. The work was conducted in two phases: the first involved shaking table studies of a medium-rise structure with a base isolation system allowing free uplift and a detailed evaluation of the behavior of the structure during uplift and the second phase of the program was to consider approaches to avoiding the occurrence of uplift of the superstructure from the isolation system.

Uplift of Medium Rise Structures

Phase I

Phase I of the uplift investigation program involved the earthquake simulator testing of a 1/5-scale 7-story reinforced concrete structure shown in Fig. 1 [Griffith, Kelly, Coveney & Koh 1988]. The model was tested with 3 different types of filled natural rubber bearing isolation systems. Throughout testing, the model was permitted to uplift freely from the bearings. An extensive testing program was performed. First of all, static rig tests of the isolation bearings were carried out to assess the performance characteristics of each type of bearing. The dynamic testing of the model consisted of free-vibration tests, harmonic excitation tests, white-noise tests, and earthquake simulation tests using 8 different earthquake motions representing a wide variety of ground motions. The input motions ranged from the predominantly high-frequency San Francisco Earthquake of 1957 to the long-period shaking of the 1985 Mexico City and 1977 Romanian Earthquakes. To satisfy model similitude requirements, the records were time-scaled by a factor of $\sqrt{5}$. The model was subjected to the time-scaled signals at three levels of peak-table acceleration (typically 0.2g, 0.5g and 0.8g) allowing the effect of increasing shear strain in the rubber and the consequent change in structural response of the system to be studied. The effectiveness of the isolation systems was studied as a function of the magnitude of the input motion.

For all input motions (and levels of input), the response was observed to be essentially rigid-body translation, an aspect that is important for the basis of simplified code design procedures. Previous tests of the model in a fixed base configuration allowed some comparisons of response to be made, and it was seen that the accelerations, interstory displacements and story shears were considerably reduced over those for a fixed-base structure.

A comparison of the SEAONC "Tentative Seismic Isolation Design Requirements" [SEAONC, 1986] with the observed responses of the structure was undertaken to evaluate the suitability of the design formulae for this particular structural system. In particular, consideration was given to the suitability of the SEAONC formula for displacement.

In evaluating the proposed code formula one of the objectives was to determine the best coefficient for "ZN" in the design equation, with consideration given to the peak-table acceleration (PGA), and the coefficients A_a and A_v as defined by ATC3-06 [ATC 1978], derived for each of the time-scaled earthquake signals. It was found that:

- The velocity-based coefficient, A_v, was the best measure of the earthquake motion for use in the formula.

- As is acknowledged, the results showed that the design procedures are not applicable to low-frequency motions.

Phase II

The concrete model used in Phase I of the uplift study had suffered substantial damage when previously tested as a fixed base structure. At the time the isolation system was added, the model had been repaired at its base and all large cracks in the beam-column joints and the shear wall were injection grouted. The favorable test results from Phase I indicated that the rehabilitation work was successful, but for the more severe testing planned it was decided to use another model structure for Phase II [Griffith, Aiken & Kelly 1988]. This structure was a 1/4-scale 9-story braced steel frame (Fig. 2) which, with a height-to-width ratio of 1.59, was more suited to developing high overturning forces during severe shaking.

The model was tested using a number of isolation systems (described in subsequent sections). The isolation system for the uplift study consisted of four lead-rubber bearings under the internal columns and four natural rubber bearings under the corner columns of the model. With this combination of bearings column uplift was achieved at moderate levels of shaking.

For a series of tests, maximum model acceleration was plotted against maximum table acceleration and it was found that the structure acceleration to cause corner column uplift was about 0.44g. Bearing-column separation displacements up to 0.75 in. were observed. The hysteresis loops for the bearings which experienced column uplift exhibited unstable behavior. The shear force hysteresis loops showed dramatic changes in slope and flattening at the extreme displacements, while the axial force hysteresis loops had a large increase in displacement (corresponding to times of column uplift) with no change in axial load (Fig. 3). This behavior was felt to be undesirable, especially since the possibility of complete disengagement of the building from the bearing could cause major damage to the structure in that region.

Uplift Restraint Device

After conducting the series of free-to-uplift tests, bearings containing a device capable of resisting uplift forces [Kelly, Griffith & Aiken 1987] were placed under the corner columns of the model and the test program was repeated. The system performed well for all earthquake inputs, and the restrainer device prevented any occurrence of column uplift during severe motions. The nature of the restrainer device was such that it also served to limit horizontal displacements, and as such, the responses of the structure were somewhat higher than for the free-to-uplift condition but the behavior was still very favorable. The hysteresis loops for a bearing containing the restrainer device (Fig. 4) showed stable behavior. The increase in the higher mode contributions to the overall response of the structure (due to the action of the devices) was slight.

Performance Evaluation of Isolation Systems

Concurrent with the uplift investigation program, an extensive series of shaking table tests was conducted to evaluate the performance of a number of different base isolation systems. These systems and some of the notable aspects of the tests are described in the following sections.

Filled Natural Rubber Bearings

In conjunction with Phase I of the uplift study, a series of tests on 3 types of filled natural rubber bearing were performed [Griffith, Kelly, Coveney & Koh 1988]. Each type of bearings were 6 in. x 6 in. in plan but with varying internal configurations. The Type-1 bearings had 16 layers of 0.213 in. thick rubber and a shear stiffness giving the 1/5-scale reinforced concrete model an isolated frequency (at about 50% shear strain) of 1.1Hz. The Type-2 bearings had 18 layers of 0.198 in. thick rubber and a shear stiffness providing an isolated frequency of 0.72 Hz at 50% shear strain. The Type-3 bearings were of a special lightweight design that was the same in all respects as the Type-2 bearings except that the internal shims were only 0.012 in. compared with 0.037 in. for the Type-2 bearings. Filled natural rubber bearings possess a significantly nonlinear effective stiffness versus shear strain relationship and this provides the isolation system with the required high stiffness at small amplitudes of motion to resist wind loads and low-level shaking, while still allowing optimum isolation performance for severe earthquake shaking.

The Type-2 bearings were designed to investigate the feasibility of using base isolation in situations of low frequency ground motion and also the feasibility of using isolation for small buildings (≈ 50-200 tons). The objective of the Type-3 bearing tests was to evaluate the performance of less expensive lightweight bearings that might be suitable for application to low-cost housing in developing countries.

All three bearing designs performed well for a large range of earthquake motions and magnitudes of input. In all cases but the predominantly long-period Mexico City signal, the isolation systems offered significant reductions in structure acceleration over the peak input acceleration, and interstory displacements (and hence story shears) were reduced considerably from those for a fixed-base structure. Small amplifications of acceleration response were observed for the Mexico City tests. It was concluded that the bearings performed extremely well and further analytical and experimental investigation in the area of lightweight bearings is warranted.

Neoprene Bearings

A neoprene bearing isolation system has been studied using the 9-story steel test structure used in Phase II of the uplift program [Kelly, Griffith & Aiken 1988]. The bearings were 6 in. x6 in. in plan and consisted of 6 layers of 3/8-in. neoprene separated by 5 1/8-in. steel reinforcing shims. The neoprene bearings possessed a significant nonlinearity of shear stiffness with shear strain. At 50% shear strain, the bearing shear stiffness was about 2.1 kip/inch and at 2% shear strain, the shear stiffness was about two times this value. This nonlinearity made direct comparisons of results from different tests difficult, but assuming a simple proportionality relationship led to the approximate damping value of ξ = 10% at a frequency of 1.5 Hz and 10% shear strain (or f = 1.1 Hz at 100% shear strain).

The performance of the neoprene bearing system was studied for the 8 earthquakes as used in previous test programs up to shear strains of 114% and was felt to be excellent. The bearings showed no instability tendencies and the shear force hysteresis loops exhibited stable behavior and no change in stiffness properties even for a large number of displacement cycles exceeding 50% shear strain. The suitability of the SEAONC design formulae for this isolation system was assessed [Griffith, Aiken & Kelly 1988] and found to give good conservative results using the A_v coefficient.

Lead-Rubber Bearings

Following the neoprene bearing tests, the 9-story structure was base isolated with a system of lead-rubber bearings and subjected to another series of simulated earthquakes. The lead-rubber bearings were geometrically similar to the neoprene bearings, but with the addition of 1.25 in. diameter lead plug inserts. The inherent damping of the natural rubber of these bearings was only about 5-7%, but with the lead plug added, the equivalent viscous damping ratio for a bearing at 50% shear strain was in the range of 20-25%. Each lead-rubber bearing had a shear stiffness of about 3.2 kip/in. (the stiffness of the bearing without the lead plug was 1.6 kip/in.). To achieve a reasonable overall system stiffness, only four of the eight bearings of the isolation system were provided with lead plugs.

Because of the inherently higher shear stiffness of the lead-rubber bearings, the degree to which the isolated building responded in its first mode was of particular interest. Test results showed that the first mode response did indeed dominate, thus confirming that the simplified code design approach is applicable to this type of structure and isolation system.

Combination Slider-Elastomeric Bearing System

This isolation system was tested using the 9-story steel structure and consisted of uplift-restrainer bearings under the four corner columns and teflon-stainless steel slider bearings situated under the interior columns of the model [Kelly & Chalhoub 1988]. The system provided significant reductions in story acceleration and base shear from the levels expected in a similar structure with a fixed-base support. The uplift restrainer bearings allowed no column uplift during the tests and the slider bearings reduced displacements from those seen for other isolation systems. This behavior is in contrasted that of a purely sliding system in that the elastomeric bearings ensured that the isolation system as a whole did not suffer any significant permanent displacement offset after shaking (the largest offsets were of the order of only 0.1 in.).

The combination isolation system offers four important features:

- resistance to wind loads and low-level shaking is implicit in the behavior of the slider bearings;
- a restoring effect is provided by the elastomeric bearings to eliminate displacement offsets;
- control of overturning and extreme displacements is provided;
- the slider bearings represent a fail-safe backup to the elastomeric bearings for cases of extreme loading.

Bridge Deck Tests

Base isolation has been implemented extensively in the seismic design of bridges. In an effort to answer some of the questions on the response of base-isolated bridge decks, an experimental program [Kelly, Buckle & Tsai 1985] was initiated to address a number of issues, including the effect of the type of isolation system on the deck behavior and the response of skew-isolated bridge decks. A 20 ft long steel deck, simulating one simple span of a bridge, was tested using two different isolation systems. The first consisted of filled multilayer natural rubber bearings (8 in. x 8 in. x 7.8 in. high) with two shear dowels in each end plate and the second used natural rubber bearings of the same design but additionally incorporating 1 1/2-in. lead plug inserts.

A parameter identification routine designed to provide an equivalent linearization of the dynamic response of the nonlinear isolation system was developed (that gave accurate displacement and acceleration maxima), to provide elementary design rules for the preliminary design of base-isolated buildings and bridges. The lead-rubber bearings were effective in reducing deck displacements 25-50% over the filled bearings, and for the real-time earthquake signals these displacement reductions were not achieved at the expense of increased accelerations. In bridge decks, the higher modes are generally very much higher than the fundamental mode and unlike the case of building structures, will not be excited by the lead-rubber bearings. For this reason, lead-rubber bearings are highly effective for bridge structures. Other results of the test series illustrated that a very simple formula can be used to predict the roll-out displacement of a bearing system. This is important as roll-out governs the displacement capacity of the isolation system and hence the system limit-state for an ultimate event.

SECOND TEST SERIES: High-Damping Natural Rubber Isolators

Following the success of the previous testing program, four different isolation bearing manufacturing companies provided isolation bearings for the testing and evaluation of the mechanical and failure characteristics of high-damping natural rubber isolators. Twenty eight natural rubber bearings were obtained from LTV Energy Products Co. from the United States, four high-damping natural rubber bearings were obtained from Bridgestone Corp. from Japan, one high-damping, low modulus natural rubber bearing was obtained from Rubber Consultants from the United Kingdom, and four polychloroprene rubber bearings were obtained from Dynamic Rubber Products from the United States. The description of the bearing designs and rubber compounds used in the elastomers is also presented. The results from the tests show that it is possible to compound natural rubber isolators with high damping and high strength, and by using high quality bonding techniques, the isolators can sustain shear forces six times greater than the force at 200% strain and they can reach strains greater than 400%. In some cases, the isolators could not, because of the displacement limit of the actuator, be tested to failure; in some tests, the displacement exceeded the diameter of the isolator while it continued to carry the vertical load.

LTV Energy Products Company Isolation Bearings

In this test series, four different types of isolators, identified here as Types-1, -2, -5 and -6, featured two different rubber compounds and two different moderate shape factors (i.e., 9 and 18). The shape factor S, is defined for a circular bearing by

$$S = \frac{\Phi}{4t}$$

where Φ is the diameter and t is the thickness of an individual layer. Each type of bearing was subjected to seven different types of tests, for a total of 28 bearings tested. Two fatigue tests for each bearing type were carried out at different shear strain levels, one at 100% and the other one at 150%. In addition, each type of bearing was subjected to five different failure tests in which the axial and the shear loads was changed in order to obtain an axial-shear load interaction diagram.

The effective stiffness and the equivalent viscous damping are the characteristics of most interest to be determined from the dynamic tests. The effective stiffness is computed from the secant measured from peak to peak in each hysteresis loop. The equivalent viscous damping is computed using the formula given in Appendix A.

Bearing Design and Rubber Compounds

Bearing Types-1 and -2 had the same rubber compound (259-62) and diameter (shim diameter of 14 in. with a 1 in. cover for a total diameter of 16 in.). However, bearing Type-1 had a shape factor of 9 with 10 layers of 3/8 in. thick rubber and 9 steel shims which were 1/16 in. thick, while Type-2 had a shape factor of 18 with 20 layers of 3/16 in. thick rubber and 19 steel shims which were 1/16 in. thick.

Bearings Types-5 and -6 had the same rubber compound (257-71) and diameter (shim diameter of 12 in. with a 1 in. cover for a total diameter of 14 in.). Type-5 had a shape factor of 9 with 12 thin layers of 3/15 in. thick rubber and 11 steel shims which were 1/16 in. thick, while Type-6 had a shape factor of 18 with 24 thin layers of 5.32 in. thick rubber and 23 steel shims which were 1/16 in. thick.

The total rubber thickness for all four types of bearings was 3.75 in. The bearings had oversized end plates to permit them to be bolted to the test machine. A summary of the dimensions and properties of the isolators is given in Table I.

The compounds used in the bearings were designated 259-62 for the Types-1 and -2) with nominal shear modulus of 120 psi at 100%, and 257-71 for the Types-5 and -6) with shear modulus of 150 psi evaluated at 100% shear strain. The nominal vertical load of 62.5 tons corresponds to 812 psi vertical pressure for bearing Types-1 and -2, and 1105 psi vertical pressure for bearing Types-5 and -6. These isolators were designed to provide a horizontal frequency of 0.50 Hz at 100%.

Test Results

There were a total of five fatigue tests, one test at each strain level for bearings Type-1 and Type-5, and just one test at 100% shear strain for bearing Type-6. During the five tests, only the two bearings subjected to cycles at 150% shear strain failed. Both bearings failed due to the heat generated by the cyclic loading which modified the properties of the rubber causing the isolator to break apart. Both fatigue tests were characterized by a drastic reduction in stiffness accompanied by an increase of energy dissipation as they neared failure. Fig. 5 shows these characteristics during the fatigue failure of isolator Type-5. The bearings subjected to cycles at 100% shear strain performed very well (i.e., bearing Type-5 resisted a total of 2880 cycles spread over several days).

The compression tests revealed that the cause of failure was the yielding and tearing of the reinforcing steel plates. The plates are loaded by surface shear stresses as they act to prevent barrelling of the bearing under the vertical load. It was demonstrated that the elastomer and the rubber steel bond were able to sustain shear stresses that were large enough to yield the steel in a type of all around tension. The failure of the steel reinforced plates of the bearing due to vertical pressure depended on the shape factor, but was in every case so much larger than the design vertical pressure or possible overload induced by seismic or other events, as to be beyond significance. A local failure of the foundation or attachment details to the superstructure would precede bearing failure in this failure mode. Table II gives the results of the compression tests and Fig. 6 shows the plot of one compression failure loop.

The predominant failure mechanism of the test program was deficient bonding between the top steel plate to the rubber bearing. The bond between the rubber and steel is a critical component of the performance of an isolator. The results show a considerable degree of variability in the bond strength. The objective should be to have failure due to tearing of the rubber rather than breaking of the bond. The program has shown that an adequate bond can be achieved if correct quality assurance procedures are followed during the molding and vulcanizing of the bearings. Most of the bearings tested under zero or tension load failed due to the debonding of the top plate. The bearings tested in shear under design vertical load or greater, performed well even though the failure mechanism was the same. Figures 7 and 8 show selected hysteresis and failure loops from the test program.

A valuable property of the isolators revealed by the testing program was that the mechanical behavior of a bearing with bond failure is almost exactly the same as before failure as long as it continues to carry the design vertical load.

Bridgestone Corp. Isolation Bearings

Four identical bearings were tested and the results confirmed that the bearings have excellent damping characteristics and that they are able to sustain very large shear deformations before failure occurs. The failure mechanism in all four bearings was tearing of the rubber itself with no failure of the rubber-steel bond. The bearings were tested at four different levels of vertical pressure 0, 500, 1000, 1500 psi. The failure strain was only slightly affected by the pressure but a significant increase in damping resulted from the increase in pressure. The damping factor was fairly constant over the range of strain up to 200% and at the highest pressure of 1500 psi averaged 15% as opposed to around 11% at zero pressure.

Bearing Design and Rubber Compound

The four bearings had 22 thin layers of 079 in. thick rubber and 21 steel shims which were 0.031 in. thick. The shim diameter was 9.45 in. and there was 0.098 in. of cover for a total diameter of 9.65 in. The bearings had oversize end plates that permitted them to be bolted to the test machine. Figure 9 shows the detailed bearing design.

The compound used in the bearings is designated KL401 and this identifies it as the high stiffness, high damping rubber which is a blend of natural and synthetic rubber with around 31% carbon filler. With a nominal vertical load of 66.2 kips corresponding to 945 psi vertical pressure, these bearings are represented by the manufacturer as providing a horizontal frequency of 0.85 Hz at 100% and a vertical frequency of 29.5 Hz. This very high vertical frequency is a consequence of the extremely thin rubber layers.

Test Results

The dynamic tests were carried out at four levels of vertical pressure, 0, 500, 1000, and 1500 psi. The bearing stiffness at the various peak strains were computed using peak-to-peak measurements in the resulting hysteresis loops. The stiffness, equivalent viscous damping, energy dissipated and shear modulus of the first and fifth cycle for bearing Types-1 and -4 are given in Table III. Figure 8 shows the variation of the effective horizontal stiffness with shear strain at different pressure levels.

The force-displacement plots for shear strains from 5% to 350% with 500 psi vertical pressure is shown in Fig. 11, and with 1500 psi vertical pressure is shown in Fig. 12. The change in the appearance of the hysteresis loops as the strains increase is clear from these plots. The loops change from being quite elliptical to being elongated with parallel sides and strong hardening.

The reduction in the damping ratios in the higher strain cycles is a consequence of the strong hardening of the material when the strain exceeds 200%. The important aspect of the bearing behavior is that the energy dissipated and this energy dissipation continued to increase while the strong hardening of the elastomer eliminated the possibility of resonant response.

For bearings of the Bridgestone type which have very large shape factors and a squat aspect ratio, the buckling load is so much larger than the design vertical load that the influence of vertical load on the horizontal characteristics through the stability of the bearing is negligible. However, the vertical pressure does play a role in the horizontal response and in these bearings the effect must be through an interaction between pressure and shear in the elastomer. This is a reasonable conjecture since for such large shape factors it has been shown [Chalhoub & Kelly 1990] that the normal assumption of incompressible material behavior does not hold. Thus, the vertical load on the bearing produces a volume change in the material and this could interact with the shear behavior to modify the horizontal response.

At smaller strain levels of less than 200%, there is no identifiable effect of pressure on stiffness; above 200% the stiffness is smaller at the higher pressure but the effect is small and can be ignored. The pressure, however, has a very definite effect on the damping. It is obvious from the hysteresis loops that their area for fixed strain increases with increasing pressure, leading to much larger damping factors.

The damping factors for each pressure level and each peak strain level were computed using the formula developed in Appendix A. Figure 13 shows the variation of the equivalent viscous damping with shear strain at different pressure levels. Furthermore, the fact that the higher pressure can generate higher damping factors with no detrimental effects would encourage the use of higher bearing pressures.

Since the horizontal natural frequency of an elastomeric isolation system is governed by the ratio of pressure to shear modulus, using a higher pressure could allow the use of a stiffer elastomer. Since it is generally easier to increase the damping in the rubber by increasing the stiffness, the result is an increase in damping both from the increased pressure and from the increased stiffness. The alternative to using a stiffer rubber is of course to use a smaller bearing. This may be advantageous since the level of damping is already so high that very little further benefit may be possible by any further increase.

After the shear diagnostic tests, each bearing was loaded monotonically to failure at a rate of 2.5 in/min. The force-displacement curves for the four levels of pressure are superimposed in Fig. 14. In the bearing sustaining the heaviest load, the-force displacement curve reached a maximum and began to drop before the rubber failed. The peak load in the bearing developed at 507% strain in the case of the bearing with 1000 psi pressure and the rubber failed at 516% strain. For the bearing with 1500 psi, the peak load developed at 473% strain and the rubber failed at 513% strain. In the bearing with zero pressure, the failure was instantaneous, in the others it was more gradual. It is interesting to note that the displacement at failure exceeded the diameter of the bearing in the case of the bearings with 0 and 500 psi vertical pressure.

The test program confirmed the extremely high quality of the Bridgestone isolators. They were shown to be capable of extremely large shear strains before failure even under high levels of vertical pressure. The fact that the failure mechanism was only slightly affected by pressure, and that the stiffness was unaffected by pressure while the damping was increased by pressure can be used in the design process to lead to smaller isolators than those currently used.

Rubber Consultants Isolation Bearing (Sendai Bearing)

Since 1986, a test facility at Tohoku University in Sendai in northern Japan has been used to study the response of base isolation systems to earthquake action [Kelly 1988]. The test facility has two buildings, one of which is base isolated and the other which is conventionally founded. The buildings are full-sized 3-story reinforced concrete structures and the dimensions and construction of the superstructures were identical. The construction of the test facility was carried out and funded by the Shimizu Corporation.

A number of different isolation systems have been installed in the isolated building in the test facility. The first was a system using low-damping natural rubber bearings provided by Bridgestone Rubber Co. combined with hydraulic dampers. The second system consisted of a high-damping rubber bearing system also provided by Bridgestone Rubber Co. but with no additional dampers. The test results for the first two systems were reported in several publications [Yamahara and Izumi 1987] [Izumi, Tobita, Kurosawa & Miyazawa 1990]. The third system consisted of a high-damping natural rubber system manufactured by Flourocarbon Corporation (now Furon) of the United States. These latter bearings were installed as part of a joint Argonne National Laboratory (ANL)/Shimizu Corp. project partially funded by the National Science Foundation (NSF). These bearings have now been replaced by a set of low modulus, high-damping natural rubber bearings manufactured by Rubber Consultants of the United Kingdom, using a new compound developed by the Malaysian Rubber Producers Research Association (MRPRA). The manufacture and installation of these isolators was also part of the joint ANL-Shimizu Corp. project and partially supported by NSF.

The new compound has a shear modulus that is about half that of the shear modulus of the high-damping rubber that has been used for most base isolated buildings used in the United States. In comparison with the previously used compound, the damping in the new rubber compound is less dependent on strain and exceeds that in the previous compound at strains of 100% and above. The modulus is dependent on strain but less so than the earlier compound.

The bearing was subjected to two sequences of horizontal displacement cycles. Each sequence included three cycles of displacement at each level of strain as follows: ±5%, ±10%, ±25%, ±50%, ±75%, ±100% and the second sequence included strains ±125%, ±150%, ±175%and ±200%.

In addition, the bearing was subjected to five cycles of vertical loading centered around 500, 625, 1000 and 1500 psi vertical pressure. The variation in the vertical load in each of these tests corresponded ±100 psi . This particular test series is close to the limits of the test system capability.

Sequence 1 horizontal tests were carried out at frequencies of 0.1 Hz. and 0.5 Hz. After the dynamic test sequence was completed the bearing was loaded monotonically at a rate of 2.5 in/min. It was not possible to fail the isolator before reaching the maximum displacement limit of the actuator.

Bearing Design and Rubber Compound

The bearing had 12 layers of .394 in. thick rubber and 11 steel shims which were 0.128 in. thick. The shim diameter was 13.78 in. and there were 0.98 in. of cover for a total diameter of 14.76 in. An additional oversize end plate was used to bolt the bearing to the test machine. With a nominal vertical load of 92.61 kips corresponding to 621 psi vertical pressure, these bearings provide a horizontal frequency of 0.40 Hz at 100%. The shape factor for this bearing was 8.75, which is considered moderate shape factor.

Test Results

The force-displacement plots for Sequences I at frequency of 0.5 Hz. with zero, and 625 psi vertical pressure are shown respectively in Figs. 15 and 16. Bearings with low shape factors and the range of height to width ratios that result from the selection of a low pressure, tend to be somewhat sensitive to vertical load due to the fact that the vertical load is a significant fraction of the bearing buckling load.

The dynamic tests were carried out at five levels of vertical pressure, 0, 500, 625, 1000, and 1500 psi. The bearing stiffness at the various peak strains were computed using peak-to-peak measurements in the resulting hysteresis loops. For this isolator, the pressure had a very definite effect on both the stiffness and the damping. It is obvious from the hysteresis loops that their area for fixed strain increases with increasing pressure, and the effective stiffness decreases, both of these factors leading to much larger damping factors.

The effect of the pressure is primarily an interaction between the horizontal stiffness and the vertical load produced by the buckling behavior of the bearing. This phenomenon has been described by Koh and Kelly [1989]. Using the approach outlined by Koh and Kelly [1989], the buckling pressure for this isolator is around 3500 psi, so that the modification of the stiffness and damping at 1500 psi should be expected.

The bearing was loaded monotonically at a rate of 2.5 in/min. No failure was achieved in the three trials. The force displacement curve for the design pressure is shown in Fig. 17. The maximum shear strain obtained was 415% with a maximum shear stress of 355 psi. This is equivalent to a maximum deflection of 19.6 in. with a maximum shear load of 52.9 Kips. It is interesting to note that the maximum displacement exceeds the diameter of the bearing.

The most important result of the failure test of these isolators is the fact that the bearing was able to sustain shear displacements under design vertical load that exceeded the diameter of the bearing. The maximum shear strain developed at the limit of the actuator displacement (412%) is, of course, very large and would be at least a factor of two beyond the most ambitious design level.

Dynamic Rubber Products Isolation Bearings

The four polychloroprene bearings were manufactured by Dynamic Rubber Products of Athens, Texas using a high-damping compound developed by Du Pont de Nemours, Inc. The design of the bearings was identical to that of the bearing provided by Rubber Consultants.

The test program included diagnostic tests at zero, 500, 1000, and 1500 psi, respectively, over a range of maximum shear to ±200. A series of vertical tests was carried out consisting of cycles of ± 100 psi centered at 500, 1000, and 1500 psi, respectively. Each bearing was then tested under monotonic loading either to failure or to the existing displacement (around 20 in.) of the actuator.

The force-displacement curves under 500 psi of vertical pressure and 1500 psi of vertical pressure are shown in Figs. 18 and 19, respectively. The force displacement curves of the four shear failure tests are shown in Fig. 16. The results demonstrate that the bearings can sustain very high shear strains and displacements that exceed the bearing diameter.

CONCLUSION

The earthquake simulator provides the ability to perform studies of the behavior of isolation systems incorporated in large-scale structures subjected to a wide range of dynamic loadings. The studies have shown the effectiveness of base isolation systems at reducing structure responses to earthquake motions. It has been shown that base isolation is suitable for medium-rise structures and a system to overcome the potential problem of structure uplift has been developed. Other studies have shown the particular suitability of base isolation to bridge decks and evaluated the effectiveness of the proposed SEAONC code regulations to predict the response of a range of different isolation systems for design purposes.

The ability to test base isolation systems and individual isolation components and to demonstrate the performance of such systems to earthquake loadings is a very important step toward achieving general acceptance of the technique of base isolation as a feasible and practical seismic design strategy. The test programs detailed here confirm the high quality of elastomeric bearings. They have shown to be capable of extremely large shear strains before failure even under high levels of vertical pressure. The fact that the failure mechanism is only slightly affected by pressure and that the stiffness is unaffected by pressure while the damping is increased by pressure can be used in the design process to lead to smaller isolators than those currently used.

The entire search to find damping mechanisms to accompany isolation systems has been a misplaced effort. Much more important for the design of practical isolation systems is the fact that the elastomer hardens very strongly after a level of shear strain has been exceeded. In the case of the Bridgestone bearings, the stiffness increases by a factor of six beyond 250%

shear strain. Thus, if 200% shear strain is taken as the nominal design level, at which level of base shear the superstructure is just at the yield point, then the base shear must increase by at least a factor of six before the isolation system fails. This means that the failure mechanism for the entire structure will be in the superstructure and not in the isolators. Conceptually, collapse of an isolated structure is no different from that of a conventional structure with, however, the proviso that the level of earthquake impact that produces the failure must be much greater for the isolated structure due to the large displacement capacity of the isolation system. The strong hardening response of the system beyond the design level means that if the system exceeds this level, the period shortens significantly and the system becomes non-resonant.

The results of these programs should give the designer confidence that base isolated buildings can be designed and built and that their performance in moderate and strong earthquakes will be superior to conventional buildings and that the isolation system will provide damage control to the structure and to internal equipment and contents. The performance in very strong earthquakes much beyond those assumed for design will also be superior to conventional structures but will have the same type of collapse mechanisms. These results should lead to increased confidence in the use of this promising new technology for buildings of all types and particularly for buildings housing sensitive internal equipment such as data centers, computer manufacturing facilities, telephone exchanges, and buildings that must be able to continue operation after strong earthquakes such as emergency control centers and hospitals. The possibility of the use of isolation for nuclear power facilities is also enhanced by the results of this test program.

REFERENCES

Applied Technology Council 1978. "Tentative Provisions for the Development of Seismic Regulations for Buildings," *ATC Publication No. ATC3-06.*

Chalhoub, M.S. & J.M. Kelly 1990. "Effect of Bulk Compressibility on the Stiffness of Cylindrical Base Isolation Bearings," *International J. Solids and Structures*, 26(7):743-760.

Griffith, M.C., I.D. Aiken & J.M. Kelly 1990. "Comparison of Earthquake Simulator Test Results to the SEAONC Tentative Seismic Isolation Design Requirements", *Earthquake Spectra*, 6(2):403-417.

Griffith, M.C., J.M. Kelly, V.A. Coveney & C-G. Koh 1988. "Experimental Evaluation of Seismic Isolation of Medium-Rise Structures Subject to Uplift," *Report No. UCB/EERC-88/02,* Earthquake Engineering Research Center, Univ. of California, Berkeley, California.

Kelly, J.M. 1988. "Base Isolation In Japan, 1988," *Report No. UCB/EERC-88/20,* Earthquake Engineering Research Center, Univ. of California, Berkeley, California.

Kelly, J.M., I.G. Buckle & H-C. Tsai 1985. "Earthquake Simulator Testing of a Base-Isolated Bridge Deck,: *Report No. UCB/EERC-85/09,* Earthquake Engineering Research Center, Univ. of California, Berkeley, California.

Kelly, J.M. & M.C. Chalhoub 1988. "Earthquake Engineering Research at Berkeley—1988: Combining Sliders and Tension Controlled Elastomeric Bearings for Base Isolation," *Report No. UCB/EERC-88/17,* Earthquake Engineering Research Center, Univ. of California, Berkeley, California.

Kelly, J.M., M.C. Griffith & I.D. Aiken 1987. "A Displacement Control and Uplift Restraint Device for Base Isolated Structures", *Report No. UCB/EERC-87/03,* Earthquake Engineering Research Center, Univ. of California, Berkeley, California.

Kelly, J.M., M.C. Griffith & I.D. Aiken 1988. "Earthquake Simulator Tests of a Neoprene Base Isolation System for Medium-Rise Structures," *Second Symposium on Seismic, Shock and Vibration Isolation, ASME PVP-147*, pp. 35-43, American Society of Mechanical Engineers Pressure Vessel and Piping Conference, Pittsburgh, Pennsylvania.

Koh, C-G. & J.M. Kelly 1989. "Viscoelastic Stability Model for Elastomeric Isolation Bearings," *J. Structural Engineering*, 115(2):285-302.

Izumi M., J. Tobita, I. Kurosawa, F. Miyazawa, & K. Kubota 1990. "Earthquake Response Characteristics of Base-Isolated Buildings by Direct Comparison with a Non-Isolated One," Architectural Reports of the Tohku University, No. 29.

Structural Engineers Association of Northern California (SEOANC) 1986. "Tentative Isolation Design Requirements," Yellow Book.

Yamahara Y. & M. Izumi 1987. "Actual Proof Tests of the Base-Isolated Building Using Full-Sized Model," Shimizu Corp. and Tohoku University.

Table I

LTV Isolation Bearings - Dimensions and Compounds

Total Rubber Thickness: 3.75 in. Single Steel Shim Thickness: 1.16 in.							
Bearing	Steel Shim			Rubber		Shape	Compound
Type	Diameter	Area (in^2)	Layers	Thickness	Layers	Factor	Type
1	14 in.	153.94	9	3/8 in.	10	9	259-62
2	14 in.	153.94	19	3/16 in.	20	18	259-62
5	12 in.	113.10	11	5/16 in.	12	9	257-71
6	12 in.	113.10	23	5/32 in.	24	18	257-71

Table II

LTV Isolation Bearings - Compression Failure Test Results

Filename	Bearing	Test	Results	
	Type (S-N)	Number	Axial Load (kips)	Axial Stress (psi)
1.04	910403.08	13	1608.95	10452
2.10	910403.04	23	2686.55	17452
5.01	910403.06	14	1202.15	10629
6.05	910403.02	24	2590.75	22907

Table III

Mechanical Characteristics of Bridgestone Bearings at 1000 Psi and 1500 Psi Vertical Pressure

BRIDGESTONE BEARING ID: HR030-2

EERC ID: No. 1

VERTICAL PRESSURE: 1000 PSI

File No.	901023.04					901023.05				901023.06			
γ (±)	10%	25%	50%	75%	100%	100%	150%	200%	250%	200%	250%	300%	350%
$K_{(eff)}$ (1st cy)	12.66	9.61	7.36	5.89	5.08	4.87	4.58	4.53	4.83	3.31	3.91	4.65	4.75
Kips/in (5th cy)	12.71	8.6	6.32	5.41	4.72	4.55	3.9	3.77	3.84	3.13	3.54	3.7	3.71
β (1st cy)	13.71	12.99	12.48	13.24	14.01	15.45	13.86	12.24	10.57	15.68	10.99	9.11	8.29
(%) (5th cy)	13.73	11.08	14.07	15.1	14.91	15.03	14.66	13.02	11.31	15.25	11.74	10.18	9.46
W_D (1st cy)	0	1	4	7	11	13	24	38	55	50	45	64	78
Kips/in (5th cy)	0	1	3	7	11	12	22	34	48	31	43	57	70
G (1st cy)	313	237	182	145	125	120	113	112	119	82	97	115	117
Ksi (5th cy)	314	213	156	134	117/113	96	93	96	77	87	92	92	

BRIDGESTONE BEARING ID: HR030-5

EERC ID: No. 4

VERTICAL PRESSURE: 1500 PSI

File No.	901023.10					901023.11				901023.12			
γ (±)	10%	25%	50%	75%	100%	100%	150%	200%	250%	200%	250%	300%	350%
$K_{(eff)}$ (1st cy)	15.76/	1.04	8.56	6.13	5.09	4.66	4.38	4.28	4.33	2.97	3.52	4.15	4.2
Kips/in (5th cy)	11.31	8.45	6.32	5.08	4.48	4.36	3.74	3.46	3.44	2.77	3.14	3.23	3.19
β (1st cy)	12.71	14.52	14.34	14.84	16.39	17.13	15.77	14.06	12.81	18.58	13.24	10.85	9.92
(%) (5th cy)	14.64	14.75	14.52	15.97	16.52	16.16	16.96	15.46	13.67	17.61	14.11	12.4	11.59
W_D (1st cy)	0	1	4	8	13	13	26	41	60	37	50	70	85
Kips/in (5th cy)	0	1	4	7	12	12	24	36	50	33	48	62	75
G (1st cy)	389	273	211	151	126	115	108	106	107	73	87	103	104
Ksi (5th cy)	279	209	156	125	111	108	92	86	85	68	78	80	79

γ (±): Shear Strain (%)

β: Equivalent Viscous Damping (%)

G: Shear Modulus (psi)

$K_{(eff)}$: Effective Stiffness (Kips/in)

W_D: Energy Dissipation (Kips/in)

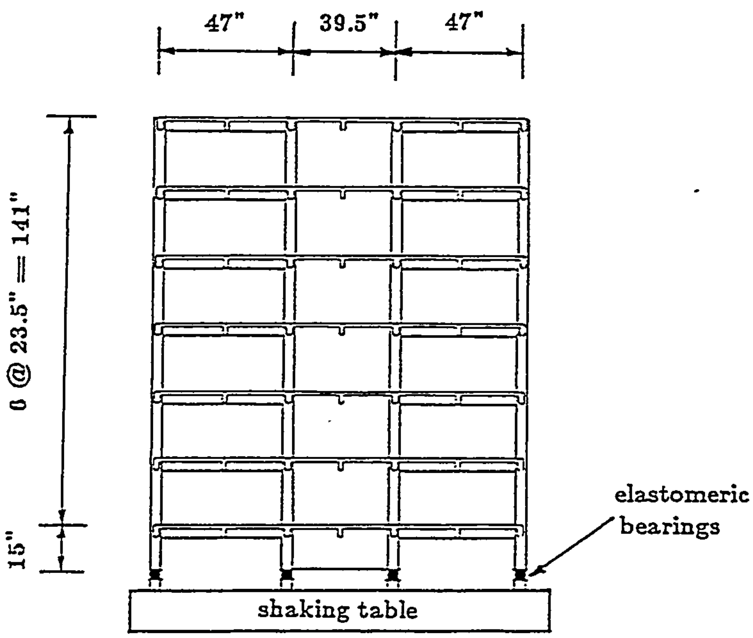


Fig. 1 1/5-Scale Reinforced Concrete Test Structure

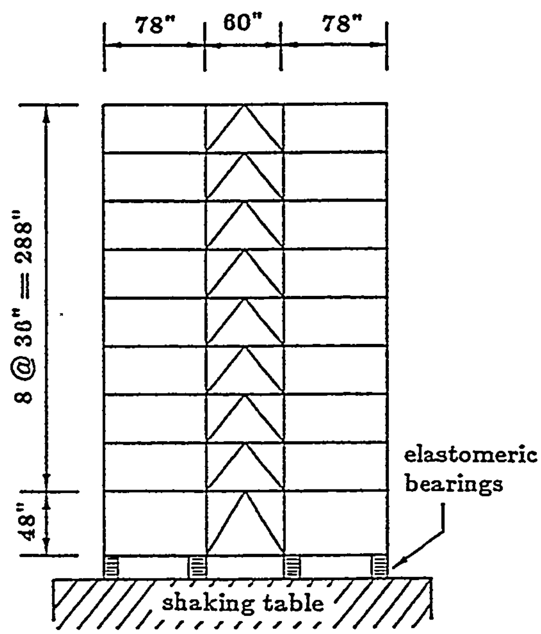
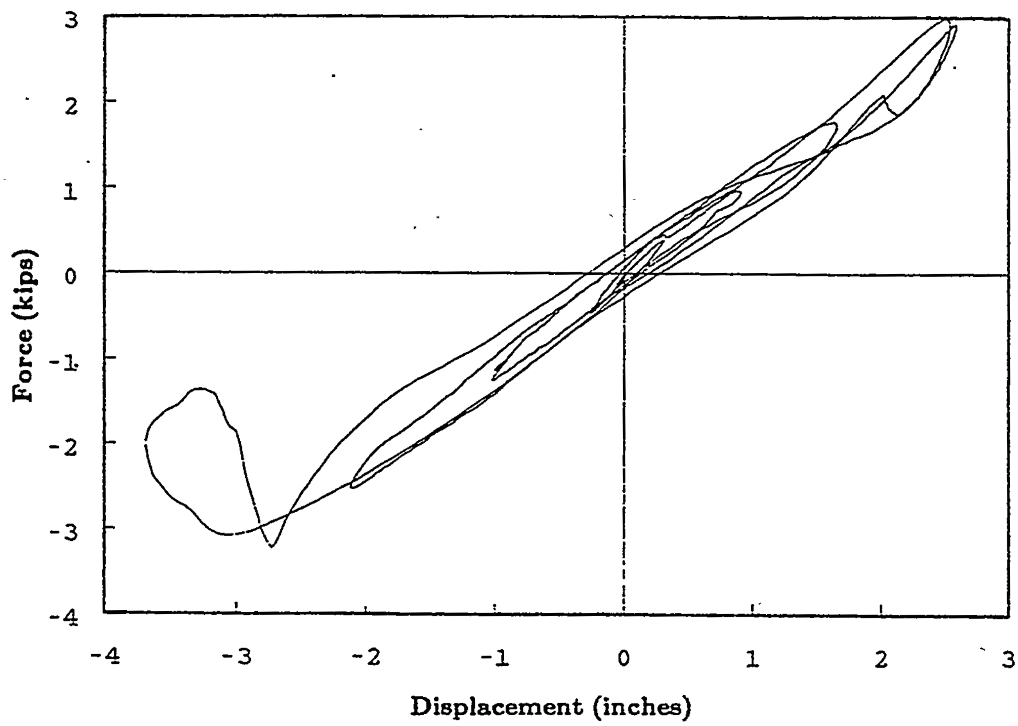
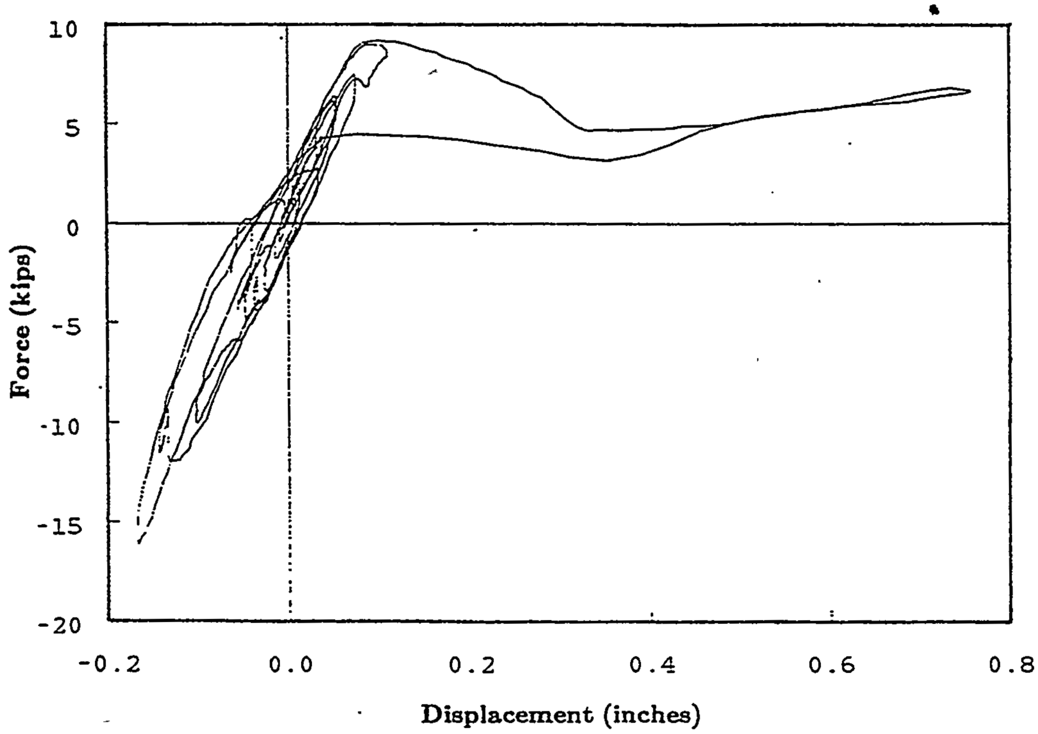


Fig. 2 1/4-Scale Steel Test Structure

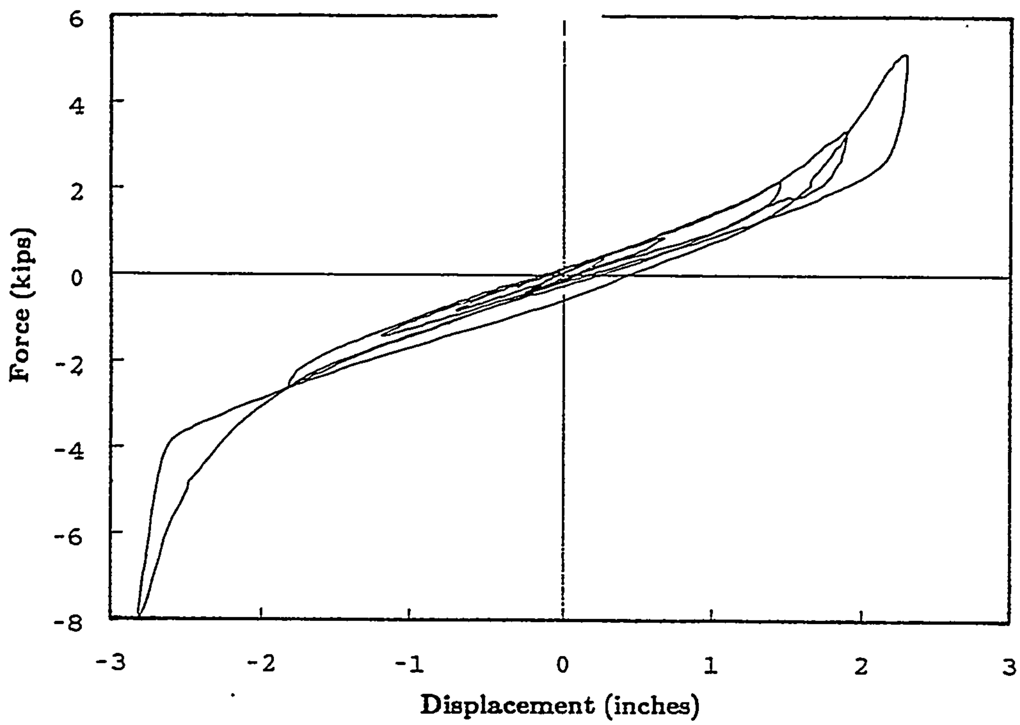


(a) Shear Force vs. Horizontal Displacement

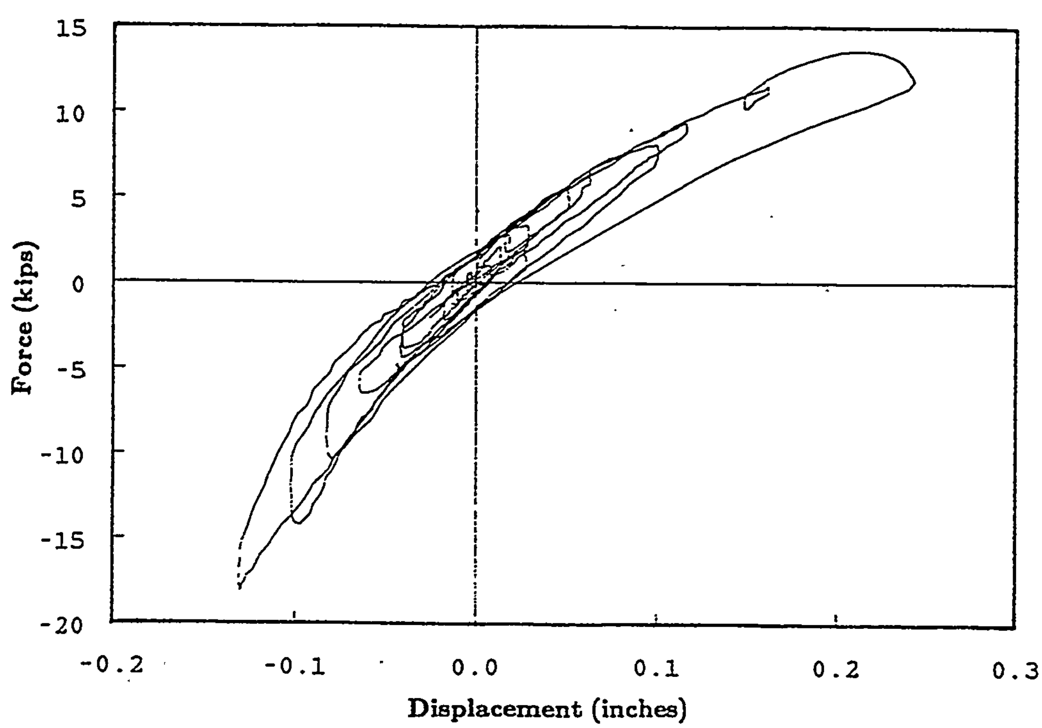


(b) Axial Force vs. Vertical Displacement

Fig. 3 Shear & Axial Force Behavior of Bearings in Free-to-Uplift Condition



(a) Shear Force vs. Horizontal Displacement



(b) Axial Force vs. Vertical Displacement

Fig. 4 Shear & Axial Force Behavior of Bearings in Uplift-Restrained Condition

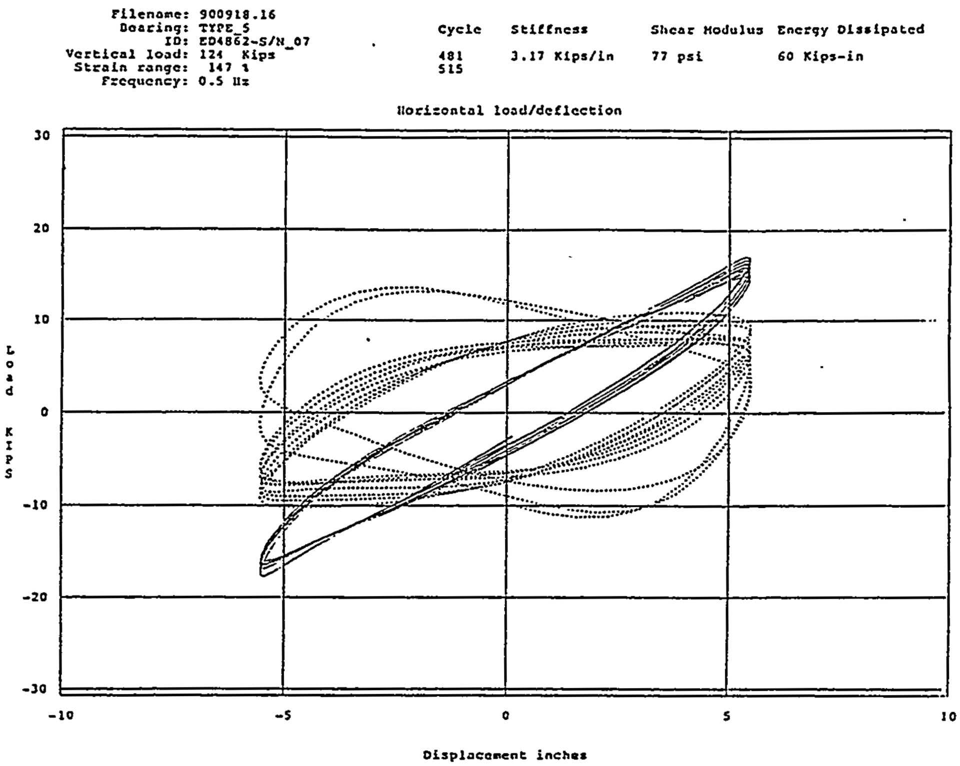


Fig. 5 Hysteresis Loops for the Fatigue Failure of LTV Bearing Type-5

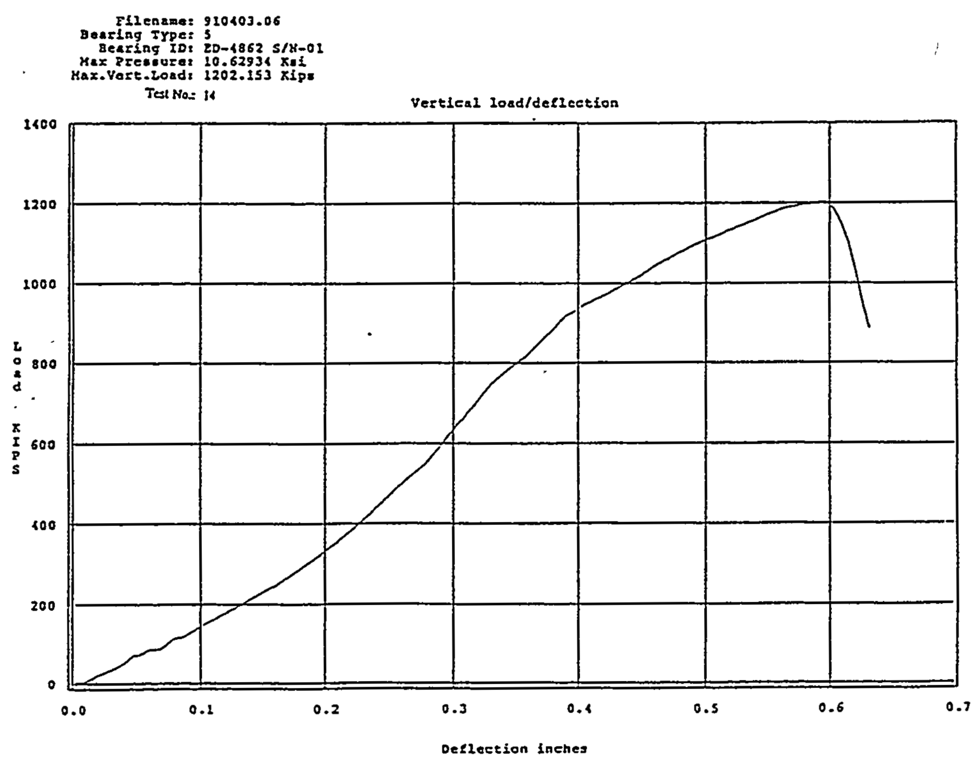


Fig. 6 Force-Deflection Plot for the Compression Failure of LTV Bearing Type-5

ARGONNE - LTV BEARING TEST
SHEAR DIAGNOSTIC OF FAILED BEARING 5%-100% STRAIN

		Strain	Stiffness	Shear Modulus	Damping
		5%	11.14 Kips/in	369 psi	12.02 %
FILENAME:	901019.07	10%	8.52	282	12.42
BEARING TYPE:	6	25%	6.85	227	10.09
BEARING ID.:	ED4863-S/N 04	50%	5.72	190	9.03
VERT PRESSURE:	1114 PSI	75%	5.05	167	8.75
TEST No.:	28	100%	4.94	164	8.17

Horizontal Load vs Deflection

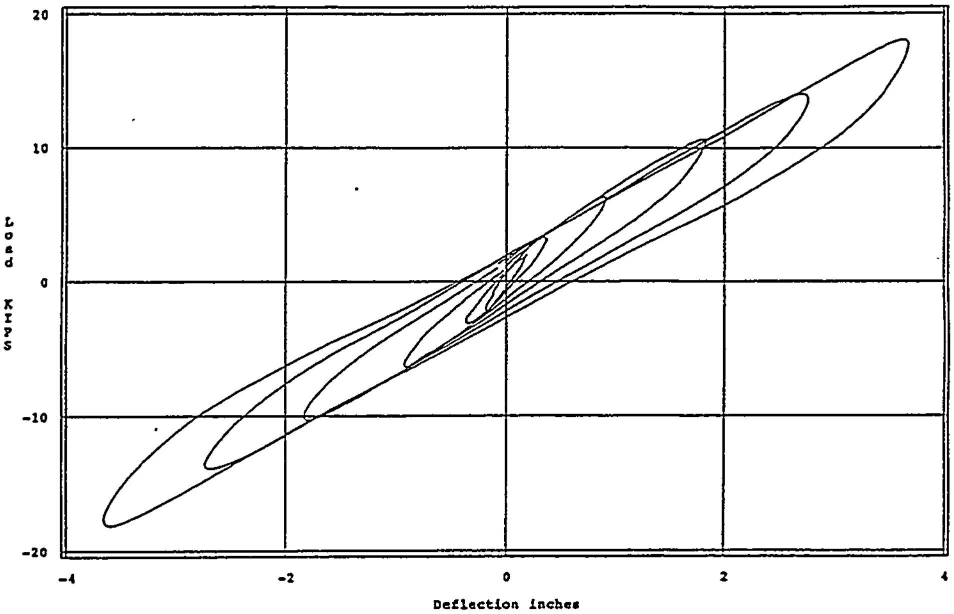


Fig. 7. Hysteresis Loops at Different Shear Strains for LTV Bearing Type-6

ARGONNE - LTV BEARING TEST
SHEAR FAILURE IN COMPRESSION P=1500 PSI

FILENAME: 901115.05
BEARING TYPE: 6
BEARING ID.: ED4863-S/N 03
VERT PRESSURE: 1512 PSI
TEST No.: 6

Horizontal Load vs Deflection

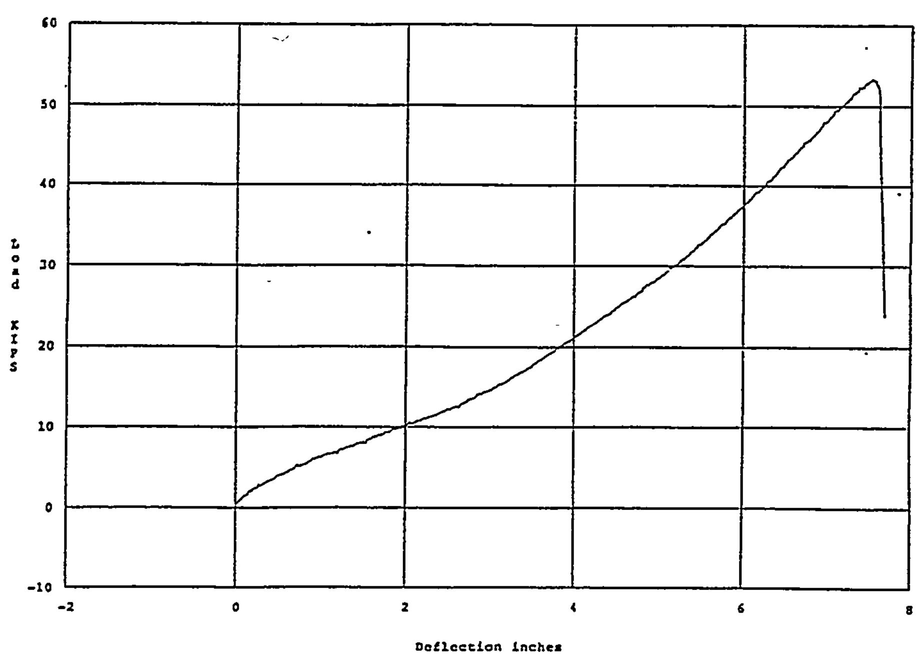


Fig. 8. Force-Deflection Plot for Shear Failure of LTV Bearing Type-6 under 1500 Psi Vertical Pressure

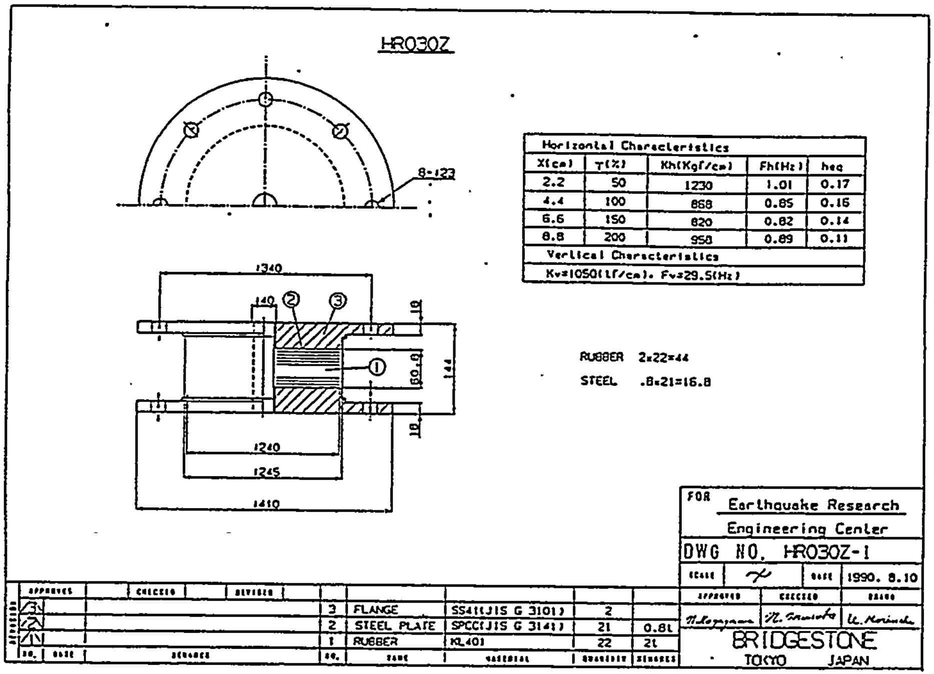


Fig. 9. Design Details of Bridgestone Test Bearings

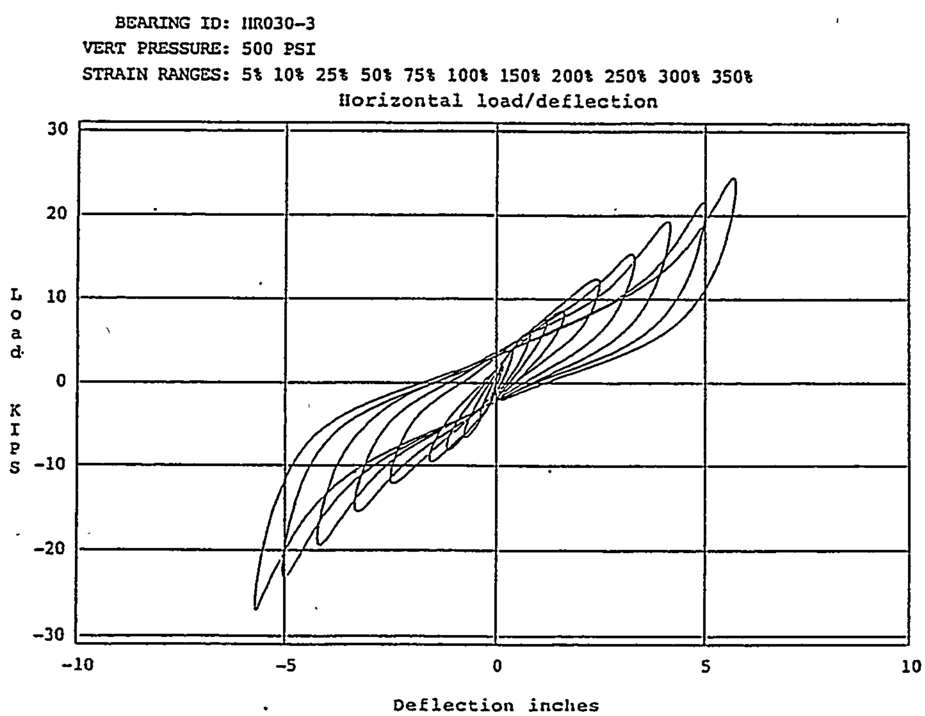


Fig. 10. Force-Displacement Plots at 500 Psi Vertical Pressure for Bridgestone Bearing

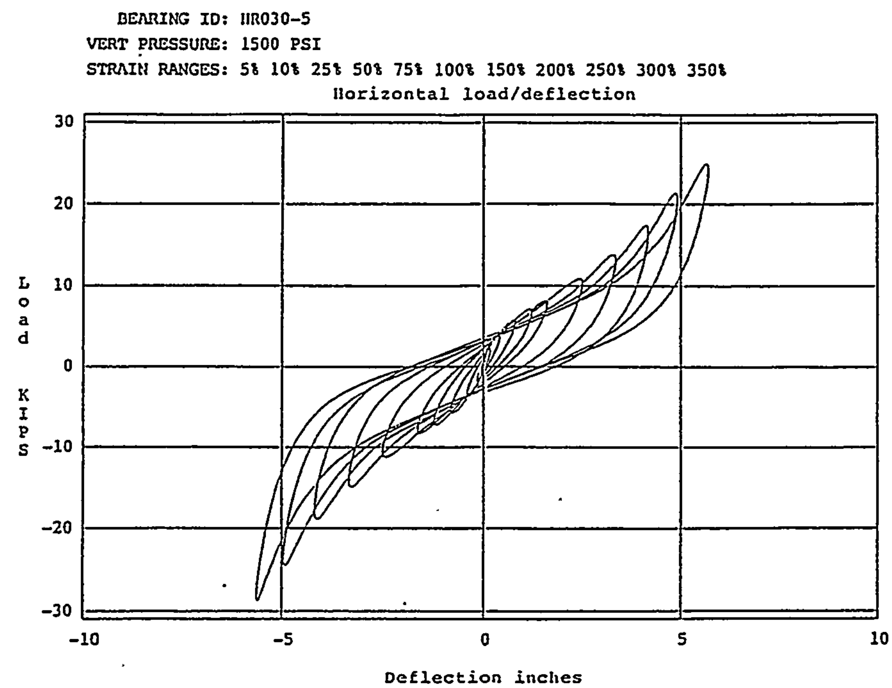


Fig. 11. Force-Displacement Plots at 1500 Psi Vertical Pressure for Bridgestone Bearing

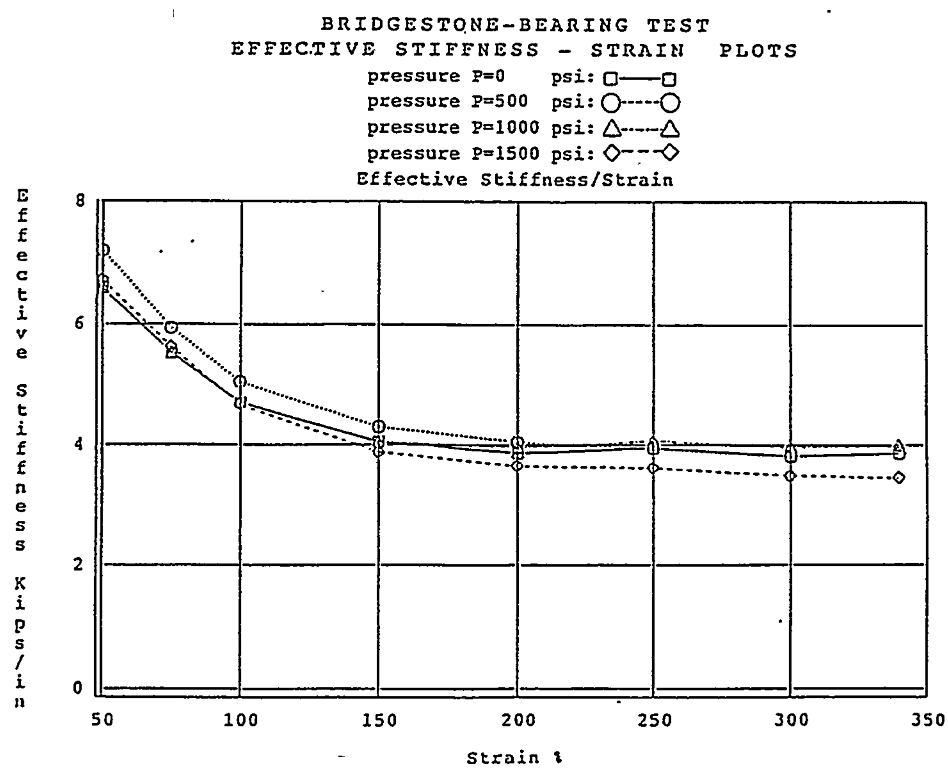


Fig. 12. Influence of Vertical Load and Strain on Bridgestone Bearing Stiffness

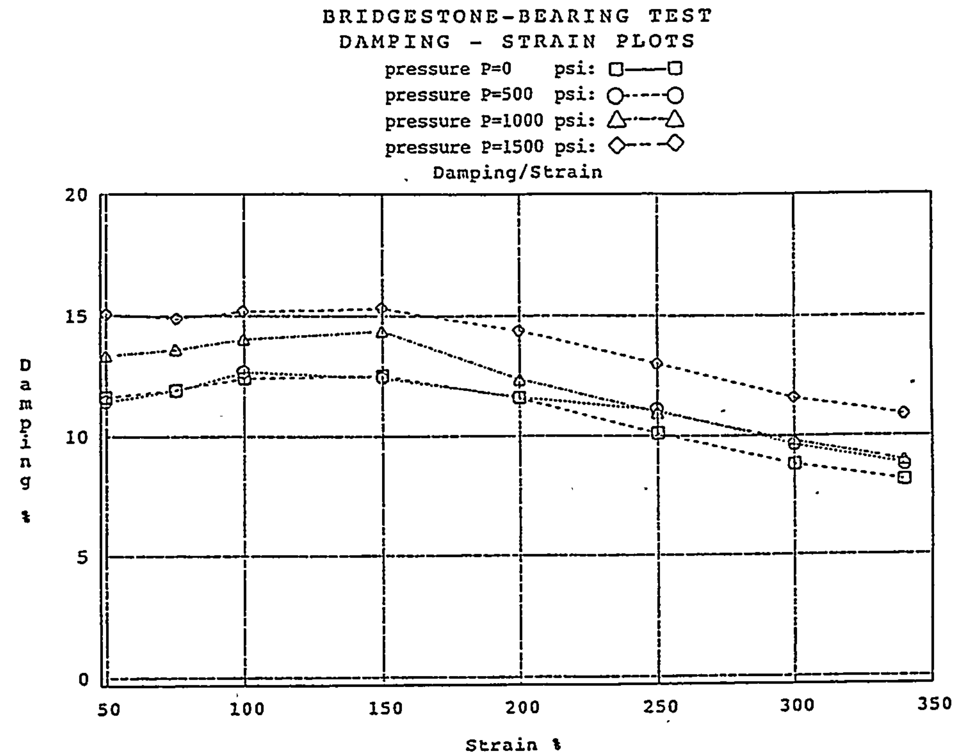


Fig. 13. Influence of Vertical Load and Strain on Bridgestone Bearing Damping

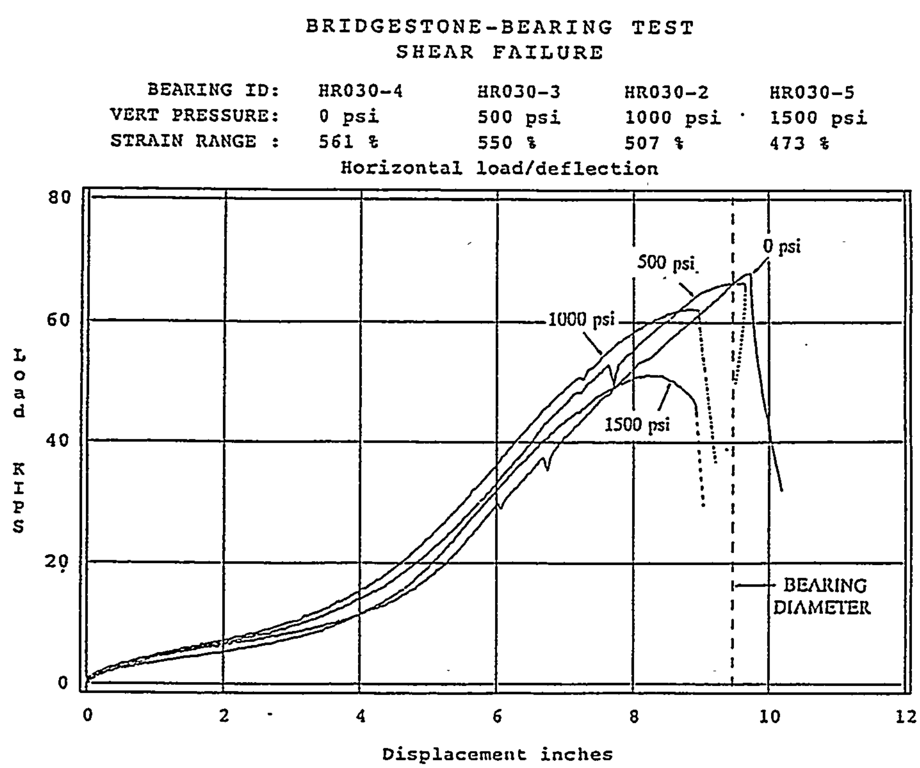


Fig. 14. Force-Displacement Curves for Bridgestone Bearings in Failure Tests

SENDAI-BEARING TEST
SHEAR DIAGNOSTIC TEST 5% - 100% STRAIN

FILENAME: 910325.10
VERT PRESSURE: 0 PSI
FREQUENCY: 0.5 HERTZ

Strain	Stiffness	Shear Modulus	Damping
5%	3.64 Kips/in	115 psi	13.69 %
10%	3.14	100	11.72
25%	2.61	83	9.66
50%	2.21	70	8.82
75%	1.97	62	8.27
100%	1.83	58	8.02

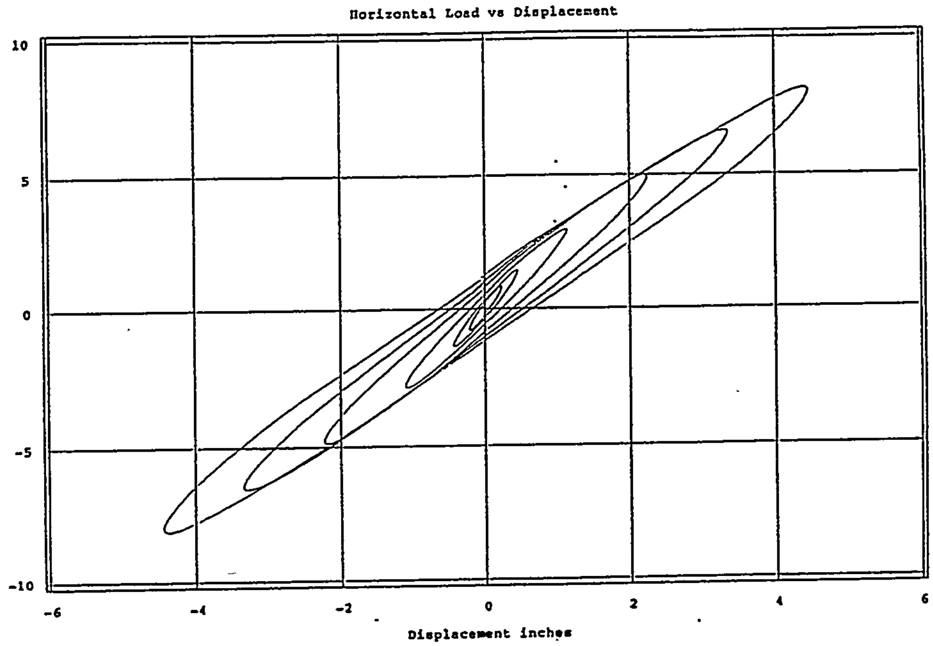


Fig. 15. Force-Displacement Plots for Sequence I - Zero Vertical Pressure at Frequency 0.5 Hz for Sendai Bearing

SENDAI-BEARING TEST
SHEAR DIAGNOSTIC TEST 5% - 100% STRAIN

FILENAME: 910325.12
VERT PRESSURE: 629 PSI
FREQUENCY: 0.5 HERTZ

Strain	Stiffness	Shear Modulus	Damping
5%	4.69 Kips/in	149 psi	18.31 %
10%	3.47	110	20.71
25%	2.41	76	19.27
50%	1.85	59	17.39
75%	1.58	50	16.17
100%	1.46	46	15.23

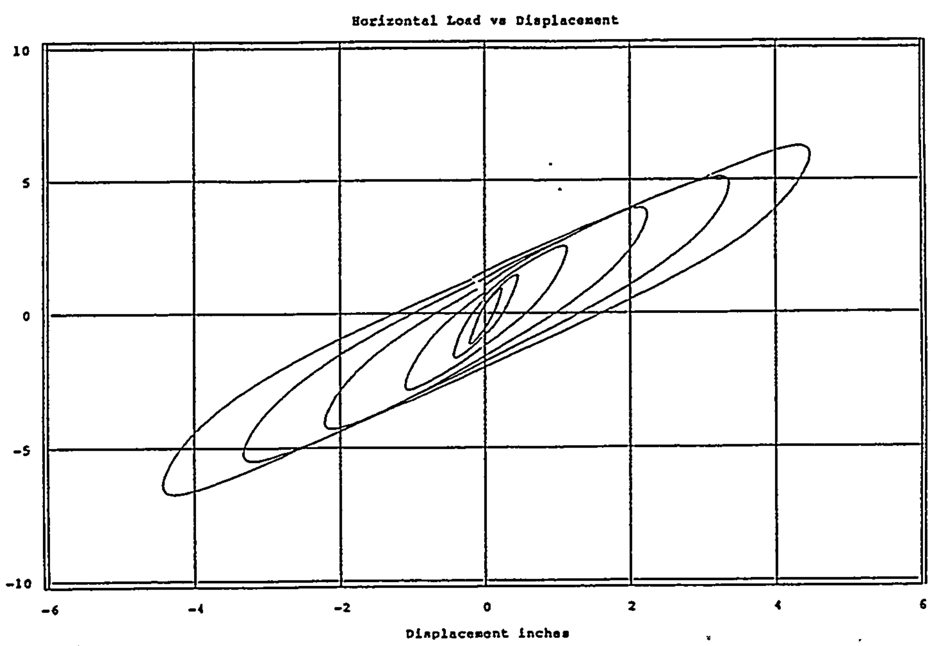


Fig. 16. Force-Displacement Plots for Sequence I - 625 psi Vertical Pressure at Frequency 0.5 Hz for Sendai Bearing

SENDAI - BEARING TEST
SHEAR FAILURE TEST - BEARING DID NOT FAIL

Filename: 910404.18

Max Strain: 415 %

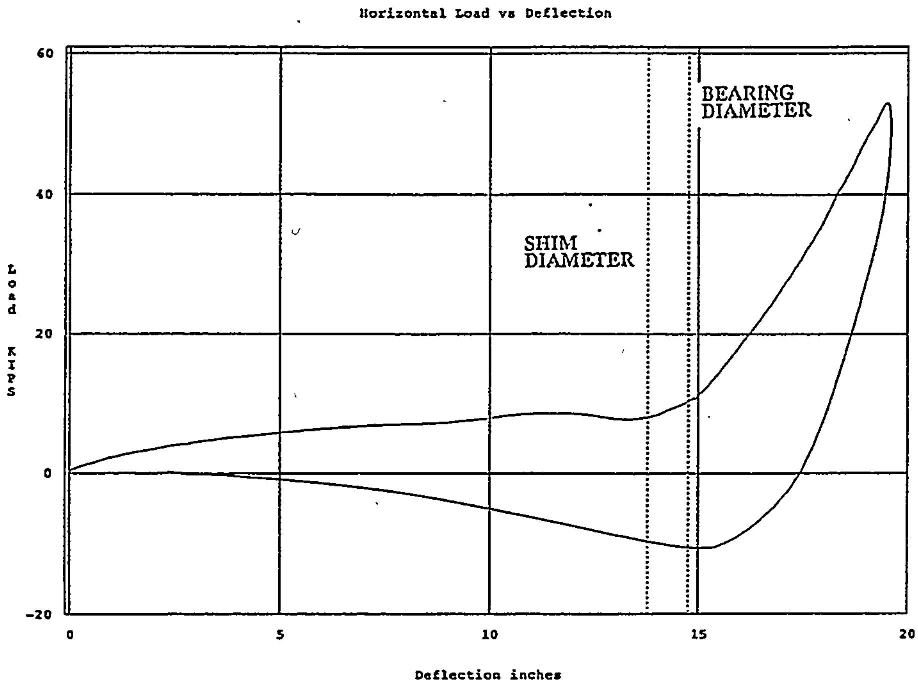


Fig. 17. Force-Displacement Plots for Sendai Bearing in Failure Test

DYNAMIC RUBBER PRODUCTS -BEARING TEST
SHEAR DIAGNOSTIC TEST 5% - 100% STRAIN

FILENAME: 910322.07
BEARING No.: 4
VERT PRESSURE: 500 PSI
FREQUENCY: 0.1 HERTZ

Strain	Stiffness	Shear Modulus	Damping
5%	14.36 Kips/in	455 psi	10.63 %
10%	12.03	381	10.27
25%	9.86	312	10.29
50%	8.14	258	10.38
75%	7.06	224	10.22
100%	6.56	208	10.31

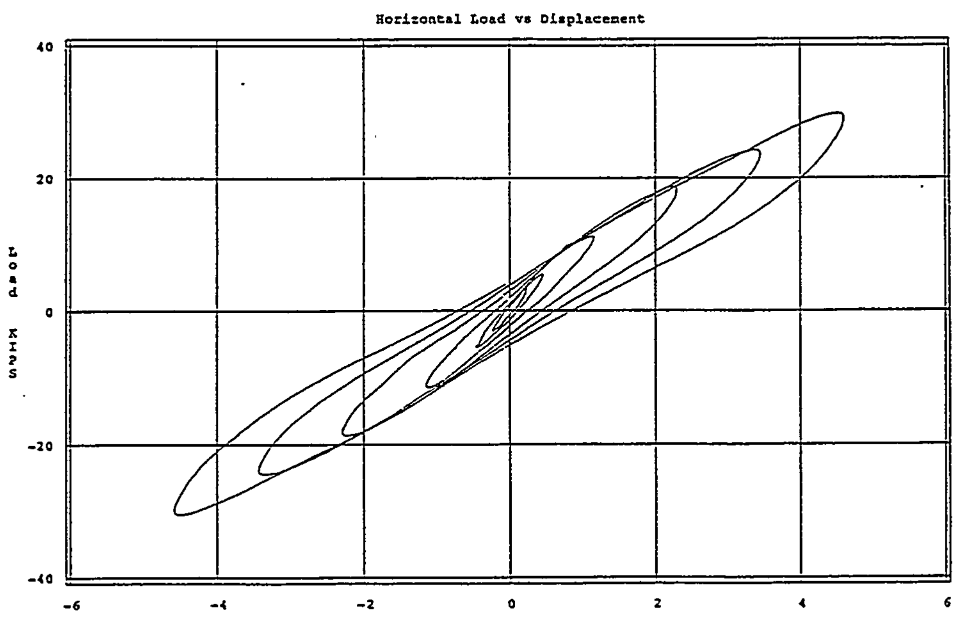


Fig. 18. Force-Displacement Plots at 500 Psi Vertical Pressure for Dynamic Rubber Products Bearing

DYNAMIC RUBBER PRODUCTS -BEARING TEST
SHEAR DIAGNOSTIC TEST 5% - 100% STRAIN

FILENAME: 910322.09
BEARING No.: 4
VERT PRESSURE: 1500 PSI
FREQUENCY: 0.1 HERTZ

Strain	Stiffness	Shear Modulus	Damping
5%	10.9 Kips/in	345 psi	13.44 %
10%	9.59	304	13.42
25%	7.72	245	13.82
50%	6.24	198	14.52
75%	5.65	179	13.98
100%	5.31	168	12.91

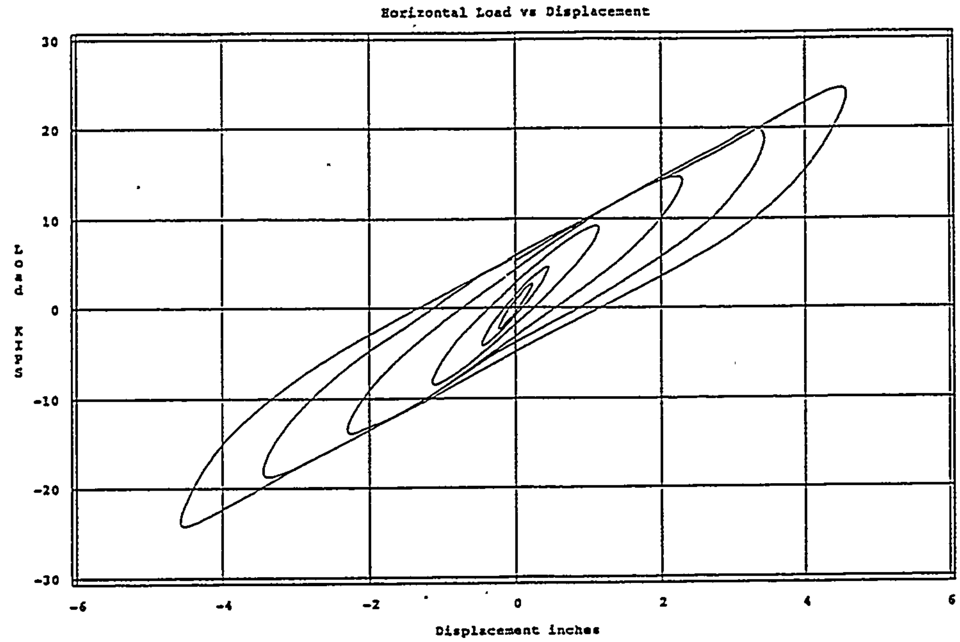


Fig. 19. Force-Displacement Plots at 1500 Psi Vertical Pressure for Dynamic Rubber Products Bearing

DYNAMIC RUBBER PRODUCTS - BEARING TESTS
SHEAR FAILURE TESTS OF 4 BEARINGS

FILENAME	BEARING No.	PRESSURE	MAX STRAIN	LINE TYPE
910410.09	1 TRIAL No. 1	0 PSI	409 %	
910415.06	2 TRIAL No. 1	1000 PSI	388 %	
910411.06	3 TRIAL No. 1	0 PSI	410 %	
910411.13	4 TRIAL No. 1	1000 PSI	363 %	

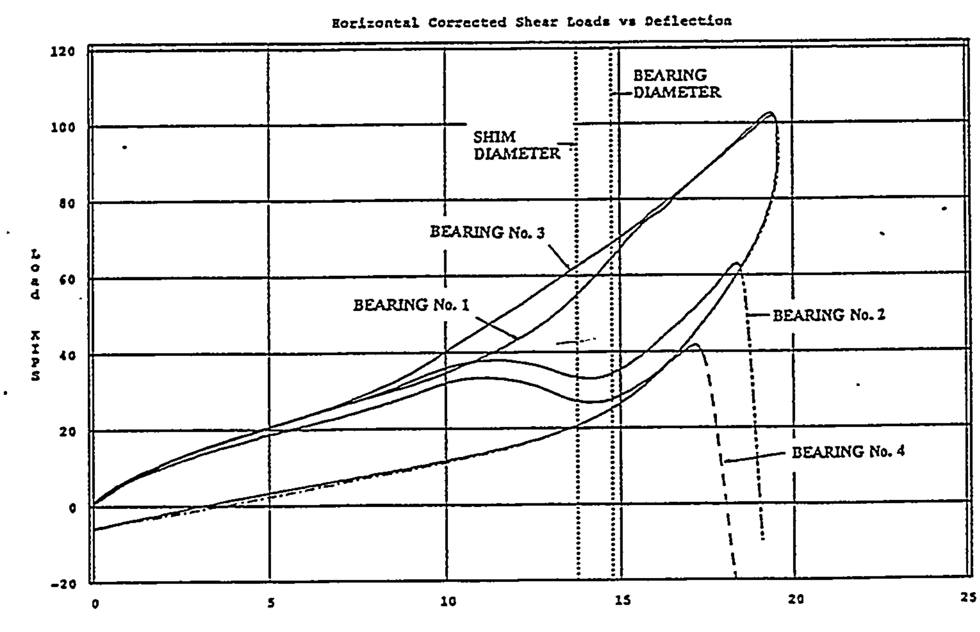


Fig. 20 Force-Displacement Plots for Dynamic Rubber Products Bearings in Shear Failure Tests

Appendix A

Estimation of Equivalent Viscous Damping

When the force displacement characteristic of an element or component is modeled by the standard structural model, the force is related to the displacement through

$$F = K\Delta + C\dot{\Delta}$$

If such an element is subjected to a sinusoidal displacement

$$\Delta = \Delta_0 \sin\omega t$$

the resulting force is

$$F = K\Delta_0 \sin\omega t + C\omega\Delta_0 \cos\omega t$$

The maximum force is given by the value of ωt for which $\dot{F} = 0$, namely,

$$\tan\omega t = \frac{K}{C\omega}$$

and the force

$$F_{\max} = K\Delta_0 \frac{K/C\omega + C\omega/K}{(1 + (K/C\omega)^2)^{1/2}}$$

or

$$F_{\max} = K\Delta_0 \left(1 + \left(\frac{C\omega}{K}\right)^2\right)^{1/2}$$

The effective stiffness, K_{eff}, is defined by

$$K_{eff} = \frac{F_{\max}}{\Delta_{\max}} = K\left(1 + \left(\frac{C\omega}{K}\right)^2\right)^{1/2}$$

The total energy dissipated by the element in a complete cycle is given by

$$W_D = \int_0^{2\pi/\omega} F(t)\,\dot{\Delta}(t)\,dt = \pi C\omega\Delta_0^2$$

Thus, in a sinusoidal test we measure the area of the hysteresis loop and K_{eff}, and from these we can determine K and C.

If the K and C were associated with a vibrating mass, m, in a single degree-of-freedom system

$$\frac{K}{m} = \omega_0^2 \quad \text{and} \quad \frac{C}{m} = 2\omega_0\beta$$

where ω_0 is the fundamental frequency and β is the fraction of critical damping. It follows that

$$C = \frac{2\beta}{\omega_0} K$$

and thus

$$W_D = 2\pi K \frac{\omega}{\omega_0} \Delta_o^2 \beta$$

leading to an estimate of β as

$$\beta = \frac{\omega_0}{\omega} \frac{W_D}{2\pi K \Delta_o^2}$$

This formula shows that the viscous model predicts that the energy dissipation should be linear in the frequency and quadratic in the displacement. The equivalent viscous damping was calculated by using the following equation:

$$\beta = \frac{W_D}{2\pi K \Delta_0^2}$$

Lecture Two

List of Slides

LECTURE 7

Failure Mechanisms in Isolators

by

James M. Kelly
Professor of Civil Engineering
Earthquake Engineering Research Center
University of California
Berkeley, California 94720

Short Course on Seismic Base Isolation
Department of Energy

August 10 - August 14, 1992

Department of Energy
Short Course on Seismic Base Isolation

J.M. Kelly
August 10-14, 1992

LECTURE 7

Failure Mechanisms in Isolators

Introduction

The design guidelines for isolated structures as typified by the current codes (SEAONC and UBC), are quite conservative. This degree of conservatism is motivated by the feeling that in the contrast to conventional structural design where there is believed to be a built-in redundancy that will insure the survival of the building in an earthquake much larger than that designed for, in an isolated building, the isolators are the sole line of defense and if these fail, collapse is inevitable. In addition, the framers of these codes have little or no experience in rubber technology and little or no confidence in the performance of elastomeric isolators. However, due to an interest in using isolation for nuclear facilities it has been possible to carry out an extensive series of tests on the failure mechanisms of elastomeric isolators. In the nuclear industry it is essential to determine the capacity of all structural components and to determine the safety margins and structural response to beyond design-basis events.

To address this issue, a test program was developed to test the dynamic characteristics and failure mechanisms of Bridgestone high-damping isolation bearings. The program under which these tests were carried out is the result of a joint effort between the Earthquake Engineering Research Center at the University of California, Berkeley, and the Reactor Engineering Division of Argonne National Laboratory. Four test bearings were obtained from Bridgestone Corporation and tested at the Structural Research Laboratory at EERC. The tests have confirmed that the bearings have excellent damping characteristics and that they are able to sustain very large shear deformations before failure occurs. The failure mechanism in all four bearings was tearing of the rubber itself with no failure of the rubber-steel bond. The bearings were tested at four different levels of vertical pressure 0, 3.45, 6.90, 10.34 MPa (0, 500, 1000, 1500 psi). The failure strain was only slightly affected by the pressure but a significant increase in damping resulted from the increase in pressure. The damping factor was fairly constant over the range of strain up to 200% and at the highest pressure of 10.34 MPa (1500 psi) averaged 15% as opposed to around 11% at zero pressure.

The results of the test series provide further evidence of the value of base isolation as a seismic resistant design strategy and should allow the structural engineering profession to use the technique with confidence. The results indicate that a building using base isolation will not only be protected against the most extreme earthquakes but will perform better than conventional structures under moderate or strong earthquakes.

Bridgestone Test Bearings

The dimensions of the test bearings provided by the Bridgestone Corporation are shown in Fig. 7-1. The bearings have 22 thin layers of .079 in. thick rubber and 21 steel shims which are 0.031 in. thick. The shim diameter is 9.45 in. and there is 0.098 in. of cover for a total diameter of 9.65 in. The bearings have oversize end plates that permit them to be bolted to the test machine.

The compound used in the bearings is designated KL401 and this identifies it as the high-stiffness, high-damping rubber which is a blend of natural and synthetic rubber with around 31% carbon filler. With a nominal vertical load of 30 tonnes (66.2 kips) corresponding to 6.52 MPa (945 psi) vertical pressure, these bearings are represented by the manufacturer as providing a horizontal frequency of 0.85 Hz at 100% and a vertical frequency of 29.5 Hz. This very high vertical frequency is a consequence of the extremely thin rubber layers. The vertical frequency of a rubber isolator is controlled by the ratio of the diameter to the thickness of the individual rubber layers. This is expressed by the shape factor S, defined for a circular bearing by

$$S = \frac{\Phi}{4t}$$

where Φ is the diameter and t is the thickness of an individual layer. Here, $S = 30$, which is very high and it has been shown [Chalhoub & Kelly 1990]] that for this range of shape factors the elementary formulas for vertical stiffness which assume that the rubber is incompressible do not apply. Compressibility of the material must be taken into account. This adds some problems in the prediction of the vertical stiffness since the compressibility of rubber, and particularly filled rubber, is not yet well understood.

Test Facilities

The testing program was carried out on a base isolation test machine at the Earthquake Simulator Laboratory and is described below.

Single Bearing Test Machine - Test Setup

The mechanical characteristic tests for the elastomeric bearings were performed in a test machine capable of subjecting single bearings to simultaneous generalized horizontal and vertical dynamic loadings. Fig. 7.2 shows the details of the test machine.

The test machine consists of two rigid-reaction frames supporting one horizontal actuator and two vertical actuators. The test bearing is mounted on a force transducer which measures shear force, axial force, and bending moment. The force transducer is located on a braced pedestal which is attached to a base block. The base block consists of a concrete block and a wide flange steel beam to provide anchorage to the test floor. Loads are applied to the test bearing by a beam; vertical loads by two vertical actuators to simulate gravity loads on the bearings; and the horizontal loads by a horizontal actuator which acts along the longitudinal axis of the load beam.

The rigid pedestal is placed between the transducer and the test floor to maximize the length of the vertical actuators so that the change in the vertical load component due to horizontal displacements during testing is not significant. Two lateral struts are connected to opposite ends of the load beam to stabilize the setup in the transverse direction. To apply constant axial loads to the test bearing, the vertical actuators are under force control, which means that the vertical load is maintained at a constant and is independent of the horizontal displacements of the load beam. In addition, the differential displacement between the two vertical actuators is maintained at zero, ensuring that the load beam is kept horizontal.

Control of the hydraulic system is performed by an MTS 443 Controller. The hydraulic actuator can develop a maximum dynamic load of 338.94 kN (76.2 kips) at a hydraulic pressure of 20.67 MN/m^2 (3000 psi). The maximum travel of the horizontal actuator is 152.4 mm (± 6 in.) (i.e., 304.8 mm or 12 in. stroke). Maximum piston velocity is 769.62 mm/sec. (30.3 in/sec.) and the servo-valve on the actuator has a flow capacity of 200 gpm. If displacements in excess of the 152.4 mm (± 6 in.) limit are required, the setup can be modified to obtain a maximum displacement of 254 mm (10 in.) for loading in one horizontal direction only. A maximum load of 1334.4 kN (300 kips) can be applied by the two vertical actuators, each with a servo-valve capacity of 25 gpm.

Signal control is performed by an IBM-AT 286 personal computer. Control signals for both components of loading can be completely general in nature.

Instrumentation

A total of 14 channels of data were recorded for the tests of the elastomeric bearings in the single bearing test machine. The following is a brief description of the components that were measured.

The loads applied by the hydraulic actuators were measured by precalibrated load cells. The compression load on the bearing is calculated by summation of the measured forces in the two vertical actuators. Shear force, axial force, and bending moment were measured by a force transducer located underneath the bearing. A linear variable differential transformer (LVDT) built into the horizontal actuator measured the horizontal displacement of the load beam (this being the lateral displacement of the bearing). Linear potentiometers attached to both of the vertical actuators provided feedback signals for the control of the vertical load. Four direct current differential transformers (DCDTs) measured the vertical displacements of the load beam near the corners of the top of the bearing. To observe any shortening of the pedestal assemblage and other components below the test bearing, two DCDTs were connected between the base block and the bottom bearing plate (situated between the bearing and the force transducer).

TEST PROGRAM

(i) Horizontal Tests

Each bearing was subjected to an identical test program with one variation; the vertical pressure was different for each bearing. Three sequences of horizontal displacement cycles were imposed on each bearing. The sequences included five cycles of displacement at each level of strain as follows

Sequence 2 .sp .5.ta 0180u 0181u.nr 3101.f .n

Each bearing was subjected to these loading histories with only a few minutes between sequences to verify that the data collected was secure.

(ii) Vertical Tests

Each bearing was subjected to five cycles of vertical loading centered around 3.45, 6.90 and 10.34 MPa (500, 1000 and 1500 psi) vertical pressure. The variation in the vertical load in each of these tests corresponded to ±0.69 MPa (±100 psi). This particular test series is close to the limits of the test system capability. Since the vertical displacements generated by the varying vertical load are of the order of several thousandths of an inch and the minimum resolution of the LVDTs used on the test rig is one thousandth of an inch, the results of this test series are not very accurate. The measurement of the extremely small vertical deformations of the bearing requires very sensitive equipment not available at EERC.

The horizontal and vertical tests were carried out at a frequency of 0.5 Hz.

(iii) Failure Tests

After the dynamic test sequence was completed, each bearing was loaded monotonically at a rate of 2.5 in/min. to failure. Failure of a bearing was assumed when the bearing could carry no further load. The vertical load on the bearings during the failure tests was the only variable. The complete test program is given in Table I.

Test Results

Dynamic Properties in Horizontal Shear

The effective stiffness and the equivalent viscous damping are the characteristics of most interest to be determined from the dynamic tests. The effective stiffness was computed from the secant measured from peak to peak in each hysteresis loop. The equivalent viscous damping was computed from the area of the hysteresis loop. At each level of maximum strain, there is a certain amount of softening between the first cycle and subsequent cycles. The effective stiffness and damping for the first and fifth cycles are given in Table II.

The loads and displacements were very small in the tests at ±5%, both close to the resolution limit of the transducers and the accuracy of the results is less than that at the higher strains. Accordingly, the results at 5% strain have not been included in the table although the hysteresis loops are given in the figures. The force-displacement plots for Sequences I - III with zero vertical pressure are shown in Figs. 7.3 - 7.5, with 500 psi (3.45 Mpa) vertical pressure in Figs. 7.6 - 7.8, with 1000 psi (6.90 Mpa) vertical pressure in Figs. 7.9 - 7.11, and with 1500 psi (10.34 Mpa) vertical pressure in Figs. 7.12 - 7.14. The change in the appearance of the hysteresis loops as the strains increase is clear from these plots. The loops change from being quite elliptical to being elongated with parallel sides and strong hardening.

The reduction in the damping ratios in the higher strain cycles is a consequence of the strong hardening of the material when the strain exceeded 200%. The damping factor quoted is based on modeling the bearing as an elastic and linear viscous element. This model predicts that the energy dissipation is quadratic in the displacement and the effective stiffness is independent of the displacements. In the actual component, the energy dissipation is not quadratic but varies roughly as displacement to the power 1.5, and the effective stiffness increases at the higher strains. Both factors act to reduce the damping factor. This reduction however, is of very little significance to the response of a system using the isolators since at these large strains it is futile to predict the dynamic response of the isolated structure by linear modeling. The important aspect of the bearing behavior is the energy dissipated and this continues to increase while the strong hardening of the elastomer eliminates the possibility of resonant response.

The relationship between the hysteresis loops generated at the various strain levels is shown in Figs. 7.15 - 7.18 where only the first cycle has been plotted for all four levels of pressure. These diagrams indicate it might be useful to use one model to analyze the response at the design level, for example, for strains that do not exceed 200%, and a second model for extreme events at strains above that. It should be emphasized, however, that the dynamic tests were carried out within a time frame of a few hours and that the large strain cycles followed many cycles at lower levels. Thus, the results at the larger strains are obtained on bearings having undergone many tests in rapid succession. The response of a bearing might be somewhat different if it were to be tested to ± 350% strain initially. Unfortunately, neither time nor number of bearings allowed such tests to be made.

7.3 Influence of Pressure on the Dynamic Properties

Isolation bearings are generally used at pressure levels ranging from 5 to 7 MPa (700 to 1000 psi), although for some very conservative designs, pressures as low as 1.72 MPa (250 psi) have been proposed. Bearings with low shape factors and the range of height to width ratios that result from the selection of a low pressure tend to be somewhat sensitive to vertical load due to the fact that the vertical load is a significant fraction of the bearing buckling load. For bearings of the Bridgestone type, which have very large shape factors and a squat aspect ratio, the buckling load is so much larger than the design vertical load that the influence of vertical load on the horizontal characteristics through the stability of the bearing is negligible. However, the vertical pressure does play a role in the horizontal response, and in these bearings the effect must be through an interaction between pressure and shear in the elastomer. This is a reasonable conjecture since for such large shape factors it has been shown [Chalhoub & Kelly 1990] that the normal assumption of incompressible material behavior does not hold. Thus, the vertical load on the bearing produces a volume change in the material and this could interact with the shear behavior to modify the horizontal response.

The dynamic tests were carried out at four levels of vertical pressure, 0, 3.45, 6.90, 10.34 MPa (0, 500, 1000 and 1500 psi). The bearing stiffness at the various peak strains were computed using peak to peak measurements in the resulting hysteresis loops. The stiffnesses are given in Table II and shown in Fig. 7.19. At smaller strain levels of less than 200%, there is no identifiable effect of pressure on stiffness; above 200% the stiffness is smaller at the higher pressure but the effect is small and can be ignored. The pressure, however, has a very definite effect on the damping. It is obvious from the hysteresis loops that their area for fixed strain increases with increasing pressure, leading to much larger damping factors.

The damping factors for each pressure level and each peak strain level are given in Table II and shown in Fig. 7.20. The damping values over the range of strains up to 200% varies very little with strain and averages 12% for 0 MPa (0 psi), 12% for 3.45 MPa (500 psi), 13.6% for 6.90 MPa (1000 psi) and 15% for 10.34 MPa (1500 psi). These are very high values for damping especially when it is recalled that for the high damping bearings with the natural rubber compound used in the U.S. [Tarics, Way & Kelly 1984], the damping drops steadily with strain. The fact that the Bridgestone compound retains its high values of damping over such a wide range of strain is a significant advance in rubber technology. Furthermore, the fact that the higher pressure can generate higher damping factors with no detrimental effects should encourage the use of higher bearing pressures.

Since the horizontal natural frequency of an elastomeric isolation system is governed by the ratio of pressure to shear modulus, using a higher pressure could allow the use of a stiffer elastomer. Since it is generally easier to increase the damping in the rubber by increasing the stiffness, the result is an increase in damping both from the increased pressure and from the increased stiffness. The alternative to using a stiffer rubber is of course to use a smaller bearing. This may be advantageous since the level of damping is already so high that very little further benefit may be possible by further increase.

Failure Test Results

Each bearing was loaded monotonically to failure at a rate of 6.35 cm/min. (2.5 in/min.). The appearance of one of the bearings at moderate and at large deformations are shown in Figs. 7.21 - 7.24. The force-displacement curves for the four levels of pressure are superimposed in Fig. 7.25. The force-displacement curve for the bearing with zero vertical load lies below the others but continues to a displacement which is larger than the others. The failure of this bearing was also different from the others in that the force rises continuously to the point of failure whereas for the loaded bearings the curve near failure is more rounded, increasingly so with increasing pressure. In the bearing carrying the heaviest load, the force-displacement curve reaches a maximum and begins dropping before the rubber fails. The peak load in the bearing was developed at 507% strain in the case of the bearing with 6.90 MPa (1000 psi) pressure and the rubber failed at 516% strain. For the bearing with 10.34 MPa (1500 psi), the peak load was developed at 473% strain and the rubber failed at 513% strain. In the bearing with zero pressure, the failure was instantaneous, in the others it was more gradual. It is interesting to note that the displacement at failure exceeds the diameter of the bearing in the case of the bearings with 0 and 3.45 MPa (0 and 500 psi) vertical pressure.

The deformation of the bearings in terms of stress versus strain is shown in Fig. 7.26. This shows that the stress-strain relation of the elastomer is trilinear. The bearing has an initial stiffness of around 2.276 Mn/m (13 kip/in.) for a strain applied up to around 15%, a stiffness of 350 kN/m (2 kips/in.) up to a strain of around 250%, and the third segment of the force-displacement curve has a stiffness of 2.276 Mn/m (13 kips/in.) from 250% to failure.

These results have implications for the analysis of buildings using this type of isolation system. The initial stiffness is significant for estimating the response of the isolated building to wind load and ground borne vibrations from traffic and other sources of low level vibration. For seismic loading at design levels, the initial stiffness can be ignored and the system analyzed as a linear system with viscous damping. For an estimate of the response under earthquake loading beyond the design level, the system can be analyzed using a bilinear model with the second stiffness much larger than the first. The analysis will be nonlinear and a realistic modeling of the energy dissipation can be made by combining viscous damping and hysteretic damping.

The strongly stiffening character of the force-displacement curves also has implications for design. For example, for the bearing with 6.90 MPa (1000 psi) pressure, the force level at 200% strain which could safely be considered as the design level, is around 44.5 kN (10 kips) and at the maximum of the force displacement curve it is 275.7 kN (62 kips). Thus, if a superstructure is designed just to be at yield level at the design strain of the isolators, the superstructure would have to be able to sustain a base shear of six times the yield level in order that the isolation system would fail before the superstructure would collapse. Although this is possible it is highly unlikely. With this approach to isolation design it can be anticipated that the isolation system is not the weakest link. Current codes that provide design requirements for base isolated buildings namely [OSHPD], [SEAOC], and [UBC] are very conservative in comparison to corresponding codes for conventionally based structures. Some of this conservatism could be relaxed on the basis of these test results.

The failure mechanism of the bearings at all levels of pressure was tearing of the rubber. No evidence of bond failure was observed. Photographs of the failed surfaces in all four bearings illustrating the rippled failure surface characteristic of tearing of the rubber are shown in Figs. 7.27 - 7.30.

The test program has confirmed the extremely high quality of the Bridgestone isolators. They have been shown to be capable of extremely large shear strains before failure even under high levels of vertical pressure. The fact that the failure mechanism is a) only slightly affected by pressure and that the stiffness is unaffected by pressure and b) while the damping is increased by pressure can be used in the design process to lead to smaller isolators than those currently used.

The damping in the bearings was found to be somewhat less than that claimed by Bridgestone. However, the difference in damping is not an important issue. It is not generally realized that the effect of an isolation system of the elastomeric type is the result of the difference between the fixed-base frequency of the superstructure and the overall frequency of the isolated building. When the ratio between these is large, the participation factor of the higher modes that involve structural deformation is low. This means that the displacement that results from the earthquake input is almost entirely in the isolation system. Also, if there is large energy in the earthquake input at these higher frequencies, this energy is not transmitted into the structure as it is in conventionally based structures. This effect is achieved even in the absence of damping. Damping is only needed to counteract the possibility of resonance at the isolation frequency. In fact, damping can be viewed as a contaminant of the isolation process. The more damping there is in the isolation system, especially if this damping is produced by nonlinear mechanisms such as mechanical dampers, the more energy leaks into the higher modes thus counteracting the beneficial effects of the isolation system. Furthermore, the difference in the dynamic response of a structure on a system with the 17% equivalent viscous damping and that obtained in the test program, 15%, is completely negligible. A two percent difference is only significant for systems with almost no damping.

The entire search to find damping mechanisms to accompany isolation systems has been a misplaced effort. Much more important for the design of practical isolation systems is the fact that the elastomer hardens very strongly after a level of shear strain has been exceeded. In these tests, the stiffness increases by a factor of six beyond 250% shear strain. Thus, if 200% shear strain is taken as the nominal design level, at which level of base shear the superstructure is just at the yield point, then the base shear must increase by at least a factor of six before the isolation system fails. This means that the failure mechanism for the entire structure will be in the superstructure and not in the isolators. Conceptually, the collapse of an isolated structure is no different from that of a conventional structure with, however, the proviso that the level of earthquake impact that produces the failure must be much greater for the isolated structure due to the large displacement capacity of the isolation system. The strong hardening response of the system beyond the design level means that if the system exceeds this level, the period shortens significantly and the system becomes non-resonant.

The results of this program should give the designer confidence that base-isolated buildings can be designed and built and that their performance in moderate and strong earthquakes will be superior to conventional buildings and that the isolation system will provide damage control to the structure and to internal equipment and contents. The performance in very strong earthquakes much beyond those assumed for design will also be superior to conventional structures but will have the same type of collapse mechanisms. These results should lead to increased confidence in the use of this promising new technology for buildings of all types and particularly for buildings housing sensitive internal equipment such as data centers, computer manufacturing facilities, telephone exchanges and buildings that must be able to continue operation after strong earthquakes such as emergency control centers and hospitals. The possibility of using isolation for nuclear power facilities is also enhanced by the results of this test program.

References

Chalhoub, M.S. & J.M. Kelly 1990. "Effect of Bulk Compressibility on the Stiffness of Cylindrical Base Isolation Bearings," *International J. Solids and Structures*, 26(7):743-760.

Tarics, A.G., D. Way, & J.M. Kelly 1984. "The Implementation of Base Isolation for the Foothills Communities Law and Justice Center," *Technical Report, RTA*, San Francisco.

Building Safety Board 1987. "Requirements for the Design and Construction of Base Isolated Hospitals," Office of Statewide Health Planning and Development (OSHPD), Hospital Building Safety Board, State of California, Sacramento.

Structural Engineering Association of California (SEAOC) 1989. "General Requirements for the Design and Construction of Seismically Isolated Structures," Ad-Hoc Base Isolation Subcommittee of the Seismology Committee, Appendix to Chapter 1 of the SEAOC Blue Book.

International Conference of Building Officials 1990. "Earthquake Regulations for Seismic-Isolated Structures," *Uniform Building Code*, Chapter 23.

Table I

Test Matrix for Bridgestone Bearings

Filename Test No.	EERC ID No.	Bearing ID No.	Vertical Pressure (psi)	Time (sec)	Rate	Remarks
901023.01	1	HR030-2	500	13	120	Vertical Test (±100psi) SPV=20
901023.02	1	HR030-2	1000	13	120	Vertical Test (±100psi) SPV=20
901023.03	1	HR030-2	1500	13	120	Vertical Test (±100psi) SPV=20
901023.04	1	HR030-2	1000	70	60	Horizontal Test (strain:5-100%) SPH=154.6
901023.05	1	HR030-2	1000	55	60	Horizontal Test (strain:100-250%) SPH=395
901023.06	1	HR030-2	1000	55	60	Horizontal Test (strain:200-350%) SPH=536
901023.07	4	HR030-5	500	13	120	Vertical Test (±100psi) SPV=20
901023.08	4	HR030-5	1000	13	120	Vertical Test (±100psi) SPV=20
901023.09	4	HR030-5	1500	13	120	Vertical Test (±100psi) SPV=20
901023.10	4	HR030-5	1500	70	60	Horizontal Test (strain:5-100%) SPH=158
901023.11	4	HR030-5	1500	70	60	Horizontal Test (strain:100-200%) SPH=395
901023.12	4	HR030-5	1500	70	60	Horizontal Test (strain:200-350%) SPH=546
901023.13	2	HR030-3	500	13	120	Vertical Test (±100psi) SPV=20
901023.14	2	HR030-3	1000	13	120	Vertical Test (±100psi) SPV=20
901023.15	2	HR030-3	1500	13	120	Vertical Test (±100psi) SPV=20
901023.16	2	HR030-3	1500	70	60	Horizontal Test (strain:5-100%) SPH=158
901023.17	2	HR030-3	1500	70	60	Horizontal Test (strain:100-200%) SPH=395
901023.18	2	HR030-3	1500	70	60	Horizontal Test (strain:200-350%) SPH=546
901023.19	3	HR030-4	500	13	120	Vertical Test (±100psi) SPV=20
901023.20	3	HR030-4	1000	13	120	Vertical Test (±100psi) SPV=20
901023.21	3	HR030-4	1500	13	120	Vertical Test (±100psi) SPV=20
901023.22	3	HR030-4	0	70	60	Horizontal Test (strain:5-100%) SPH=158
901023.23	3	HR030-4	0	70	60	Horizontal Test (strain:100-200%) SPH=395
901023.24	3	HR030-4	0	70	60	Horizontal Test (strain:100-200%) SPH=546
901023.25	3	HR030-4	0	290	25	Shear Failure, SPH=1000
901024.01	2	HR030-3	500	5	120	Zero
901024.02	2	HR030-3	500	290	25	Shear Failure, SPH=1000
901024.03	1	HR030-2	1000	5	120	Zero
901024.04	1	HR030-2	1000	290	25	Shear Failure, SPH=1000
901024.05	4	HR030-5	1500	5	120	Zero
901024.06	4	HR030-5	1500	290	25	Shear Failure, SPH=1000

Table II

Horizontal Shear Test

BRIDGESTONE BEARING ID: HR030-4
EERC ID: No. 3
VERTICAL PRESSURE: 0 PSI

File No.	901023.22					901023.23				901023.24			
γ (±)	10%	25%	50%	75%	100%	100%	150%	200%	250%	200%	250%	300%	350%
$K_{(eff)}$ (1st cy)	12.58	9.22	7.28	5.89	5.14	4.86	4.53	4.41	4.62	3.28	3.84	4.5	4.49
Kips/in. (5th cy)	11.21	8.75	6.54	5.45	4.67	4.61	3.94	3.80	3.74	3.13	3.48	3.52	3.62
β (1st cy)	12.25	12.24	12.01	11.95	12.09	13.18	12.17	11.39	9.97	13.64	10.02	8.74	7.89
(%) (5th cy)	13.39	13.14	12.06	12.09	13.86	12.01	12.70	11.61	10.61	13.22	10.47	9.4	8.4
W_D (1st cy)	0	1	4	6	10	11	21	35	50	31	43	62	70
Kips/in (5th cy)	0	1	3	6	11	10	19	30	43	29	39	52	60
G (1st cy)	311	228	180	145	127	120	112	109	114	81	95	111	111
Ksi (5th cy)	277	216	162	135	115	114	97	94	93	77	86	87	90

BRIDGESTONE BEARING ID: HR030-3
EERC ID: No. 2
VERTICAL PRESSURE: 500 PSI

File No.	901023.16					901023.17				901023.18			
γ (±)	10%	25%	50%	75%	100%	100%	150%	200%	250%	200%	250%	300%	350%
$K_{(eff)}$(1st cy)	13.11	10.22	7.77	6.43	5.56	5.27	4.89	4.6	4.61	3.4	3.79	4.46	4.5
Kips/in (5th cy)	12.7	9.41	7.08	5.79	4.98	4.94	4.15	3.78	3.71	3.18	3.49	3.6	3.62
β (1st cy)	10.22	12.91	12.6	12.15	12.8	13.39	12.85	12.11	10.59	14.2	10.68	9.06	8.42
(%) (5th cy)	11.21	11.65	11.87	12.77	14.03	13.56	13.55	12.89	11.88	14.49	11.3	9.98	9.06
W_D (1st cy)	0	1	4	7	11	12	24	38	53	33	44	63	77
Kips/in (5th cy)	0	1	3	7	11	11	22	34	48	31	42	56	67
G (1st cy)	324	253	192	159	137	130	121	114	114	84	94	110	111
Ksi (5th cy)	314	232	175	143	123	122	103	93	92	79	85	89	89

γ (±): Shear Strain (%)
$K_{(eff)}$: Effective Stiffness (Kips/in)
β: Equivalent Viscous Damping (%)
W_D: Energy Dissipation (Kips/in)
G: Shear Modulus (psi)

Table II - Continued

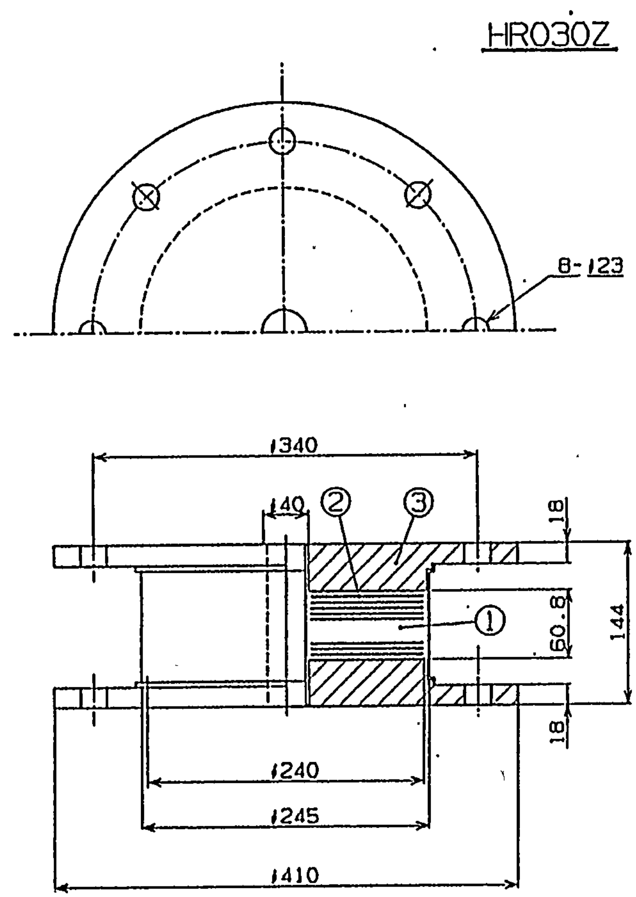
BRIDGESTONE BEARING ID: HR030-2
EERC ID: No. 1
VERTICAL PRESSURE: 1000 PSI

File No.	901023.04					901023.05				901023.06			
γ (±)	10%	25%	50%	75%	100%	100%	150%	200%	250%	200%	250%	300%	350%
$K_{(eff)}$(1st cy)	12.66	9.61	7.36	5.89	5.08	4.87	4.58	4.53	4.83	3.31	3.91	4.65	4.75
Kips/in (5th cy)	12.71	8.6	6.32	5.41	4.72	4.55	3.9	3.77	3.84	3.13	3.54	3.7	3.71
β (1st cy)	13.71	12.99	12.48	13.24	14.01	15.45	13.86	12.24	10.57	15.68	10.99	9.11	8.29
(%) (5th cy)	13.73	11.08	14.07	15.1	14.91	15.03	14.66	13.02	11.31	15.25	11.74	10.18	9.46
W_D (1st cy)	0	1	4	7	11	13	24	38	55	50	45	64	78
Kips/in (5th cy)	0	1	3	7	11	12	22	34	48	31	43	57	70
G (1st cy)	313	237	182	145	125	120	113	112	119	82	97	115	117
Ksi (5th cy)	314	213	156	134	117/113	96	93	96	77	87	92	92	

BRIDGESTONE BEARING ID: HR030-5
EERC ID: No. 4
VERTICAL PRESSURE: 1500 PSI

File No.	901023.10					901023.11				901023.12			
γ (±)	10%	25%	50%	75%	100%	100%	150%	200%	250%	200%	250%	300%	350%
$K_{(eff)}$(1st cy)	15.76/	1.04	8.56	6.13	5.09	4.66	4.38	4.28	4.33	2.97	3.52	4.15	4.2
Kips/in (5th cy)	11.31	8.45	6.32	5.08	4.48	4.36	3.74	3.46	3.44	2.77	3.14	3.23	3.19
β (1st cy)	12.71	14.52	14.34	14.84	16.39	17.13	15.77	14.06	12.81	18.58	13.24	10.85	9.92
(%) (5th cy)	14.64	14.75	14.52	15.97	16.52	16.16	16.96	15.46	13.67	17.61	14.11	12.4	11.59
W_D (1st cy)	0	1	4	8	13	13	26	41	60	37	50	70	85
Kips/in (5th cy)	0	1	4	7	12	12	24	36	50	33	48	62	75
G (1st cy)	389	273	211	151	126	115	108	106	107	73	87	103	104
Ksi (5th cy)	279	209	156	125	111	108	92	86	85	68	78	80	79

γ (±): Shear Strain (%)
$K_{(eff)}$: Effective Stiffness (Kips/in)
β: Equivalent Viscous Damping (%)
W_D: Energy Dissipation (Kips/in)
G: Shear Modulus (psi)



Horizontal Characteristics				
X(cm)	γ(%)	Kh(Kgf/cm)	Fh(Hz)	heq
2.2	50	1230	1.01	0.17
4.4	100	868	0.85	0.16
6.6	150	820	0.82	0.14
8.8	200	958	0.89	0.11
Vertical Characteristics				
Kv=1050(tf/cm), Fv=29.5(Hz)				

RUBBER 2x22=44

STEEL .8x21=16.8

NO.	NAME	MATERIAL	QUANTITY	REMARKS
3	FLANGE	SS41(JIS G 3101)	2	
2	STEEL PLATE	SPCC(JIS G 3141)	21	0.8t
1	RUBBER	KL401	22	2t

REVISION NO.	DATE	REMARKS
3		
2		
1		

APPROVED CHECKED REVISED

FOR Earthquake Research Engineering Center

DWG NO. HR030Z-1

SCALE	DATE
~	1990. 8.10

APPROVED	CHECKED	DRAWN
N. Kageyama	N. Murota	U. Noriuchi

BRIDGESTONE
TOKYO JAPAN

Fig. 7.1 Design Details for Test Bearings

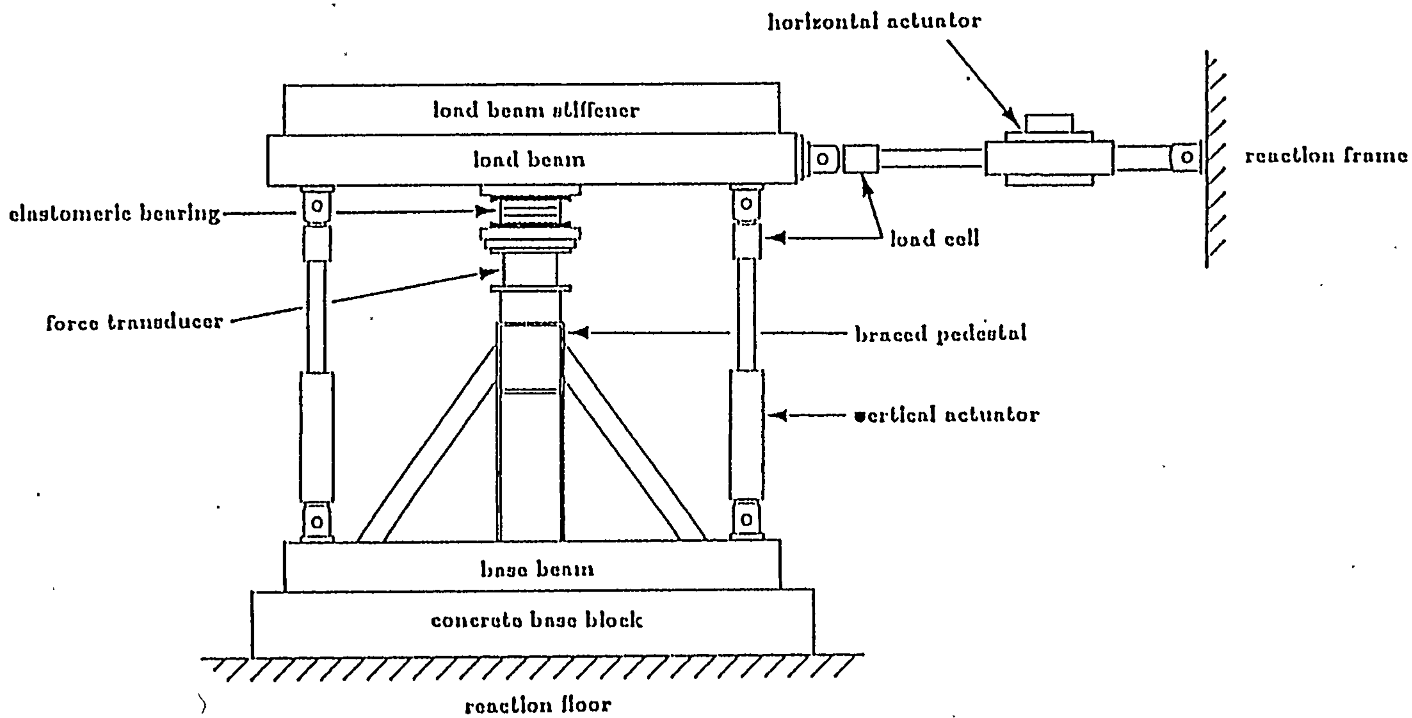


Fig. 7.2 Single Bearing Test Machine

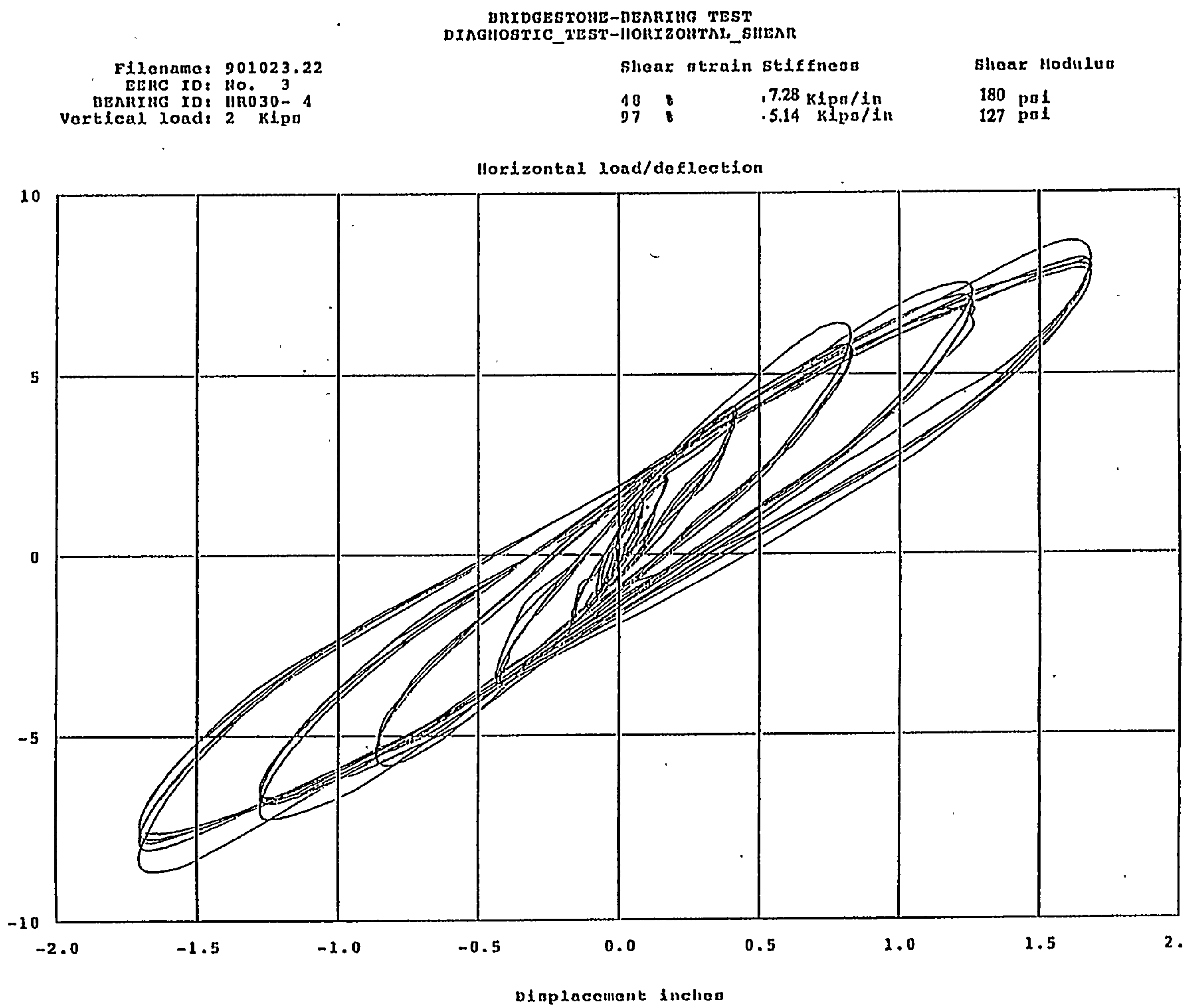


Fig. 7.3 Force Displacement Plots for Sequence I - Vertical Zero Pressure

BRIDGESTONE-BEARING TEST
DIAGNOSTIC_TEST-HORIZONTAL_SHEAR

Filename: 901023.23	Shear strain	Stiffness	Shear Modulus
EERC ID: No. 3			
BEARING ID: HR030- 4	146 %	4.53 Kips/in	112 psi
Vertical load: 1 Kips	245 %	4.62 Kips/in	114 psi

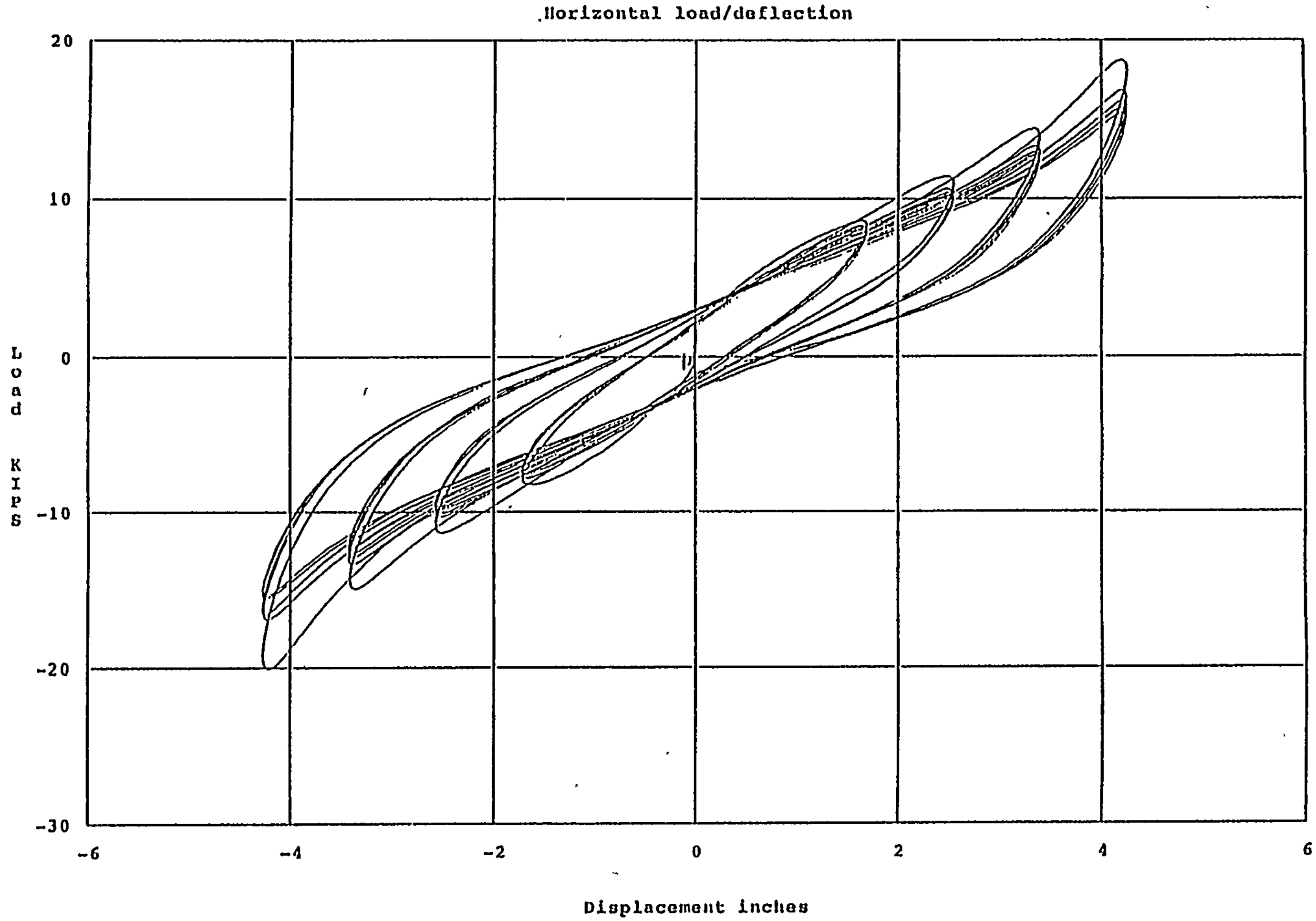


Fig. 7.4 Force Displacement Plots for Sequence II - Vertical Zero Pressure

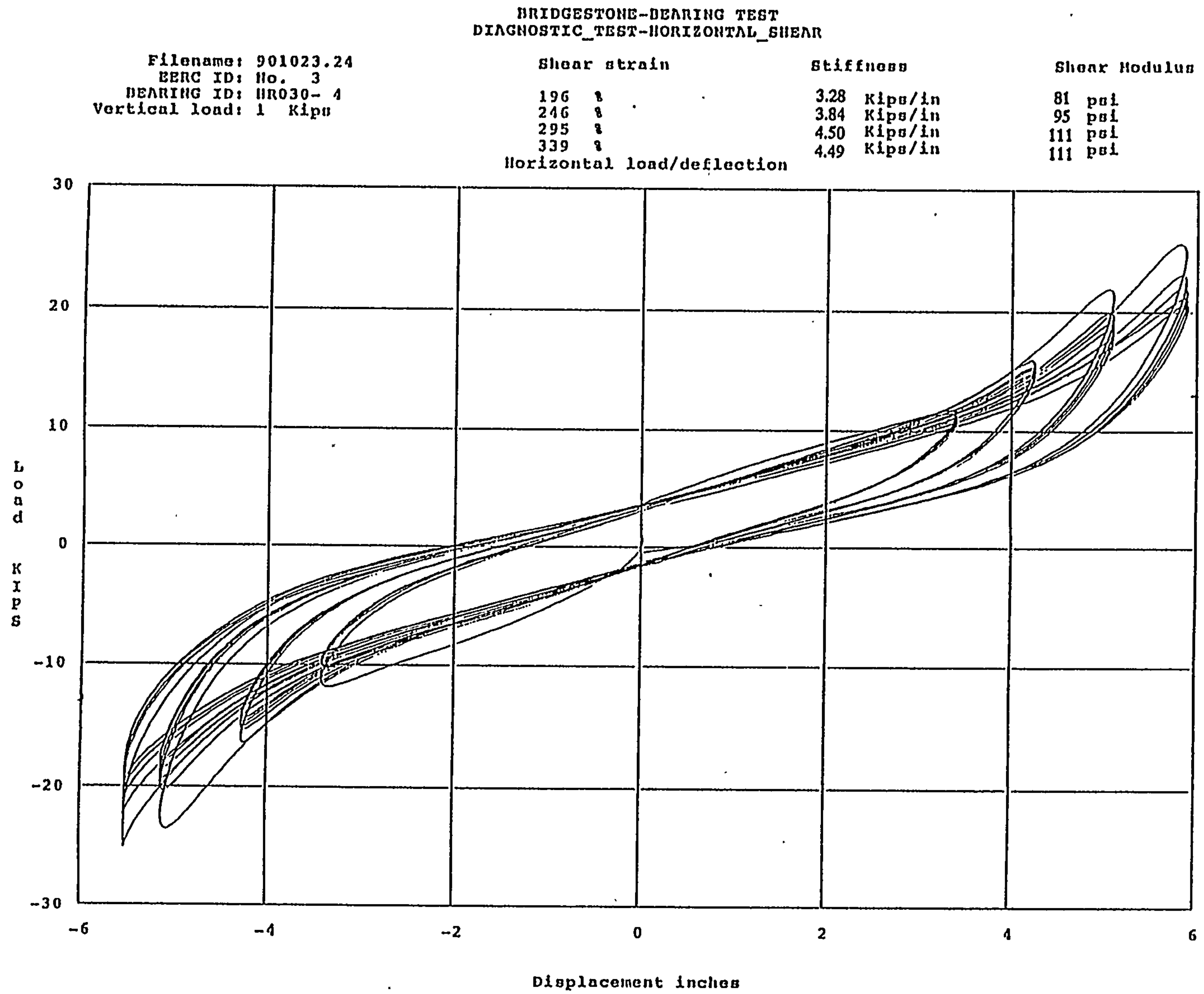


Fig. 7.5 Force Displacement Plots for Sequence III - Vertical Zero Pressure

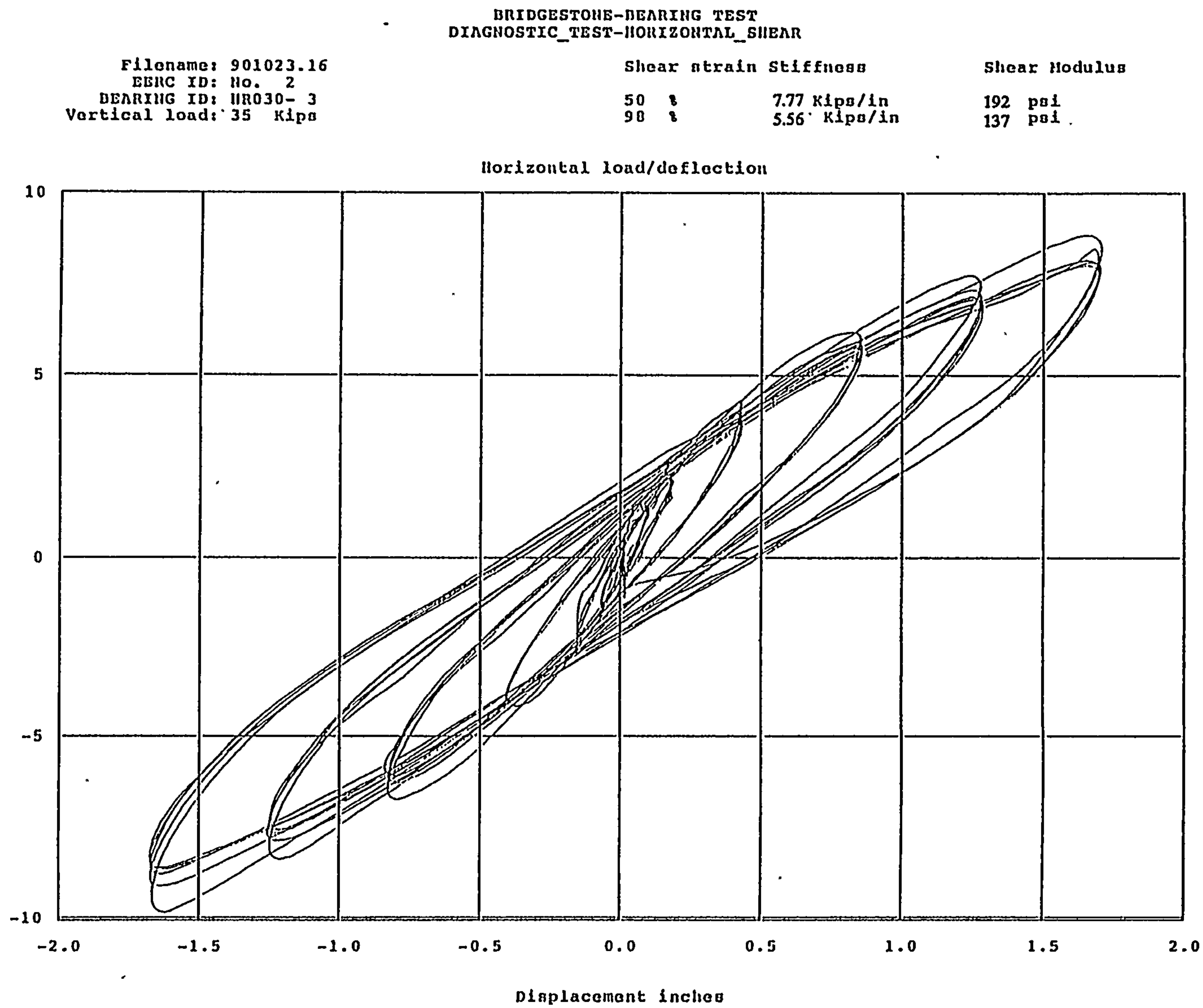


Fig. 7.6 Force Displacement Plots for Sequence I - 500 psi (3.45 MPa) Vertical Pressure

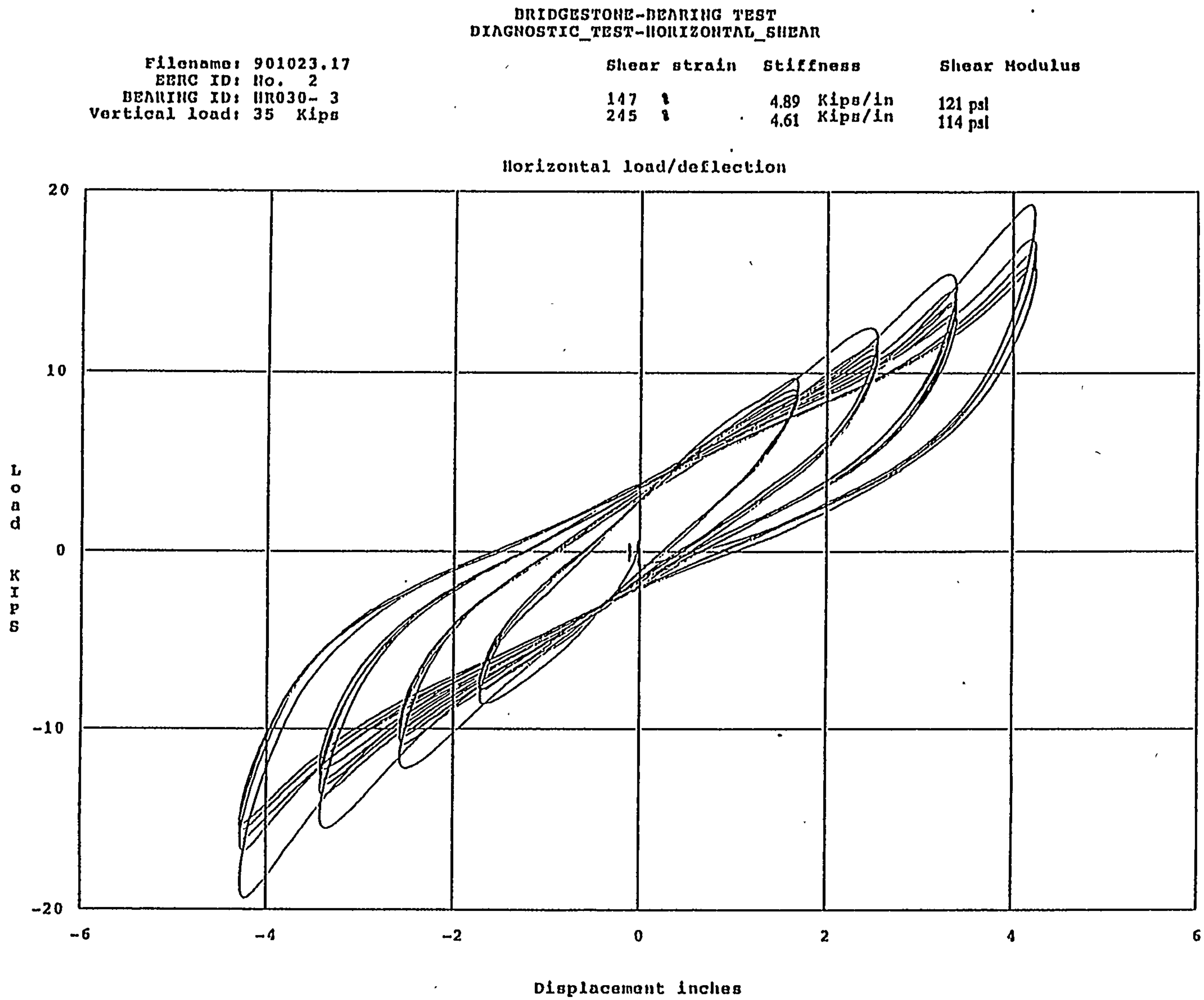


Fig. 7.7 Force Displacement Plots for Sequence II - 500 psi (3.45 MPa) Vertical Pressure

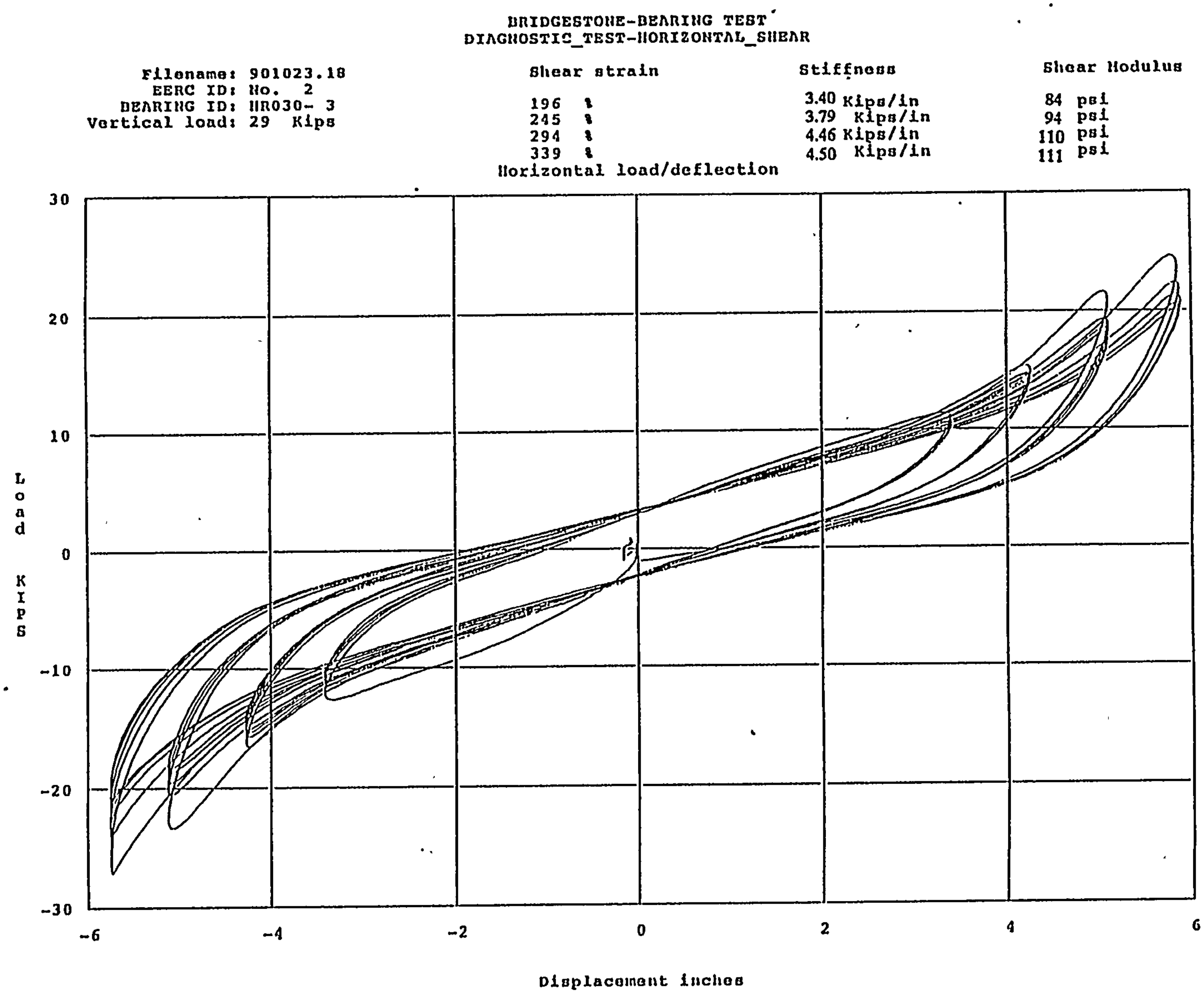


Fig. 7.8 Force Displacement Plots for Sequence III - 500 psi (3.45 MPa) Vertical Pressure

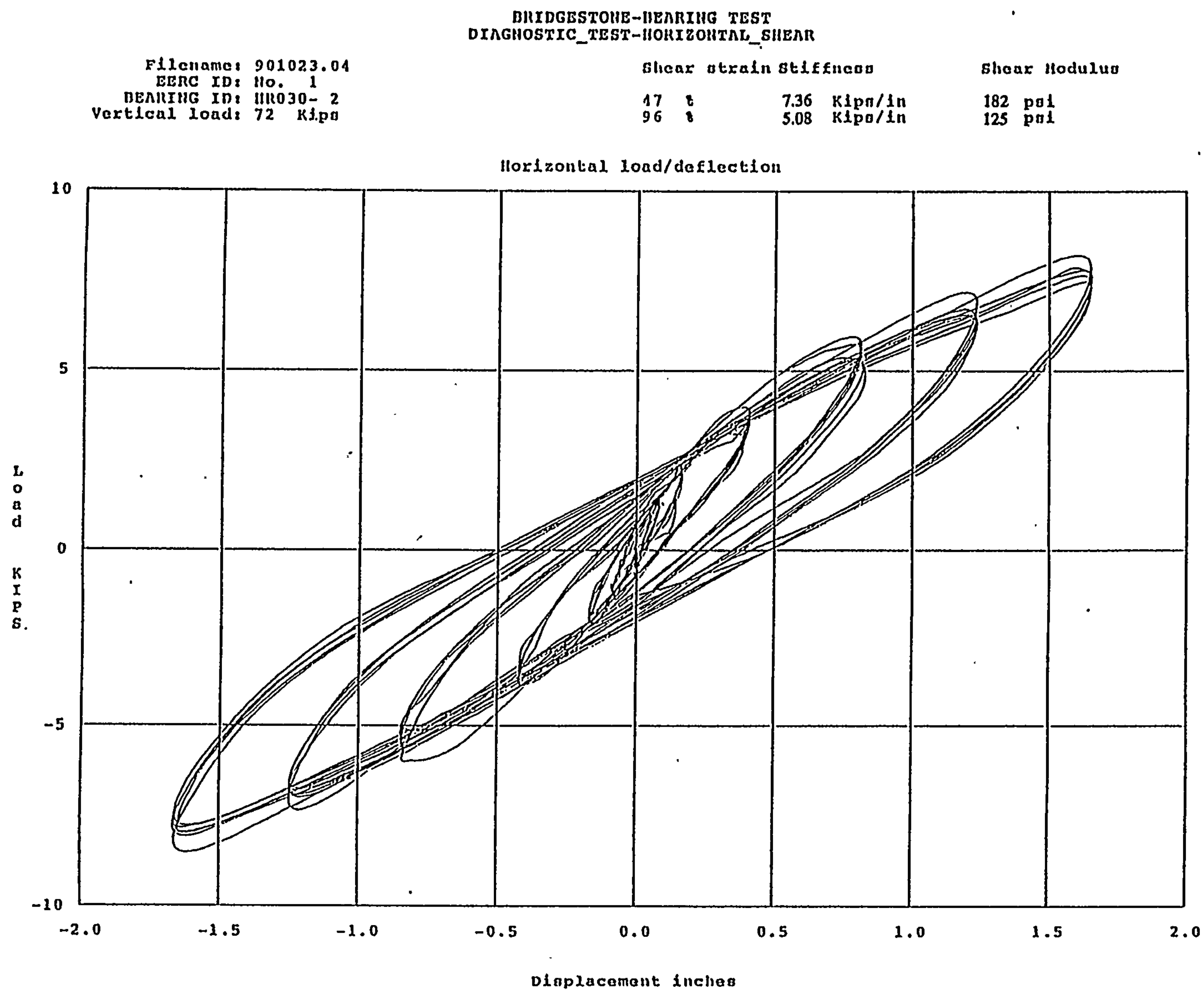


Fig. 7.9 Force Displacement Plots for Sequence I -1200 psi (6.90 MPa) Vertical Pressure

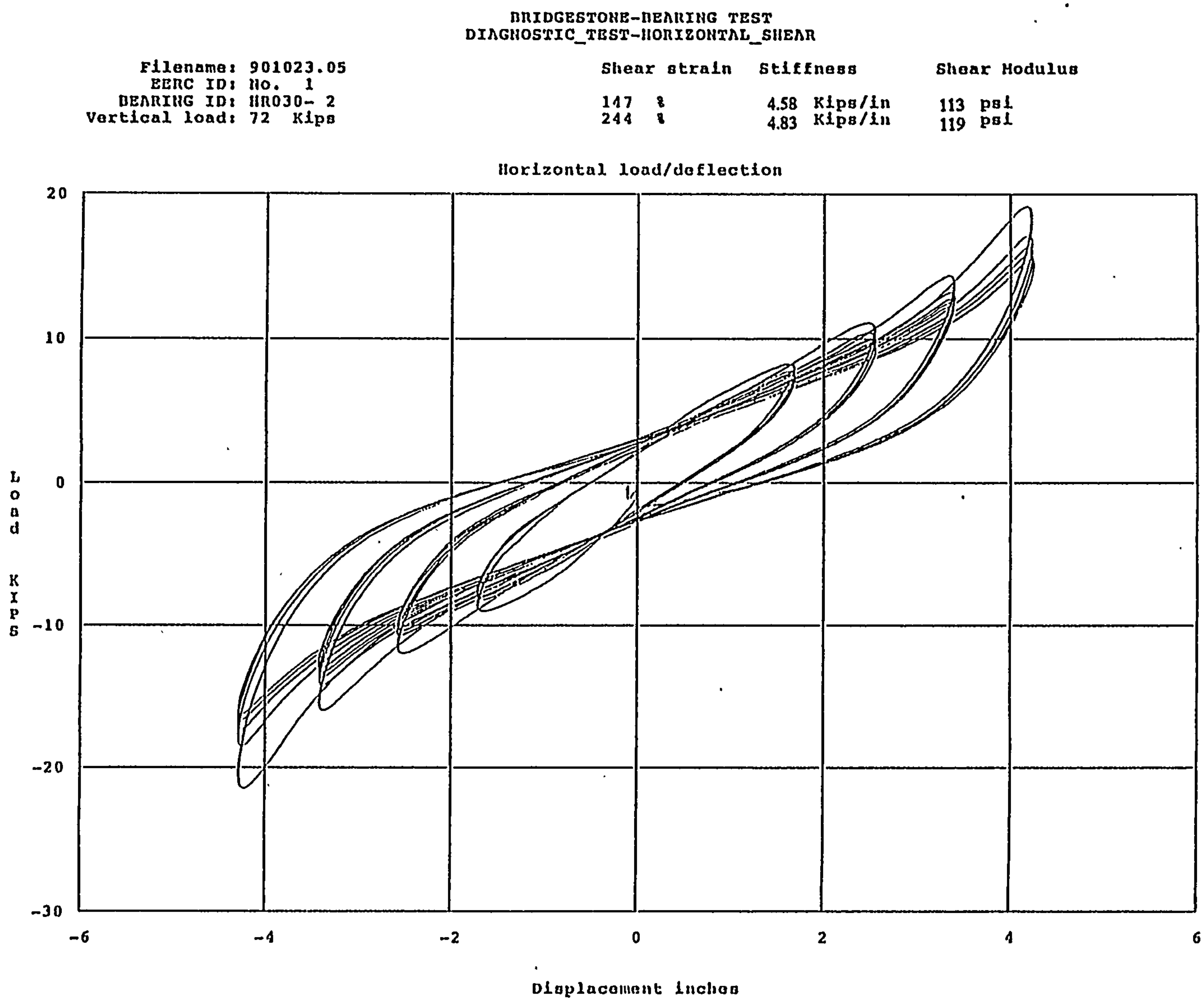


Fig. 7.10 Force Displacement Plots for Sequence II -1200 psi (6.90 MPa) Vertical Pressure

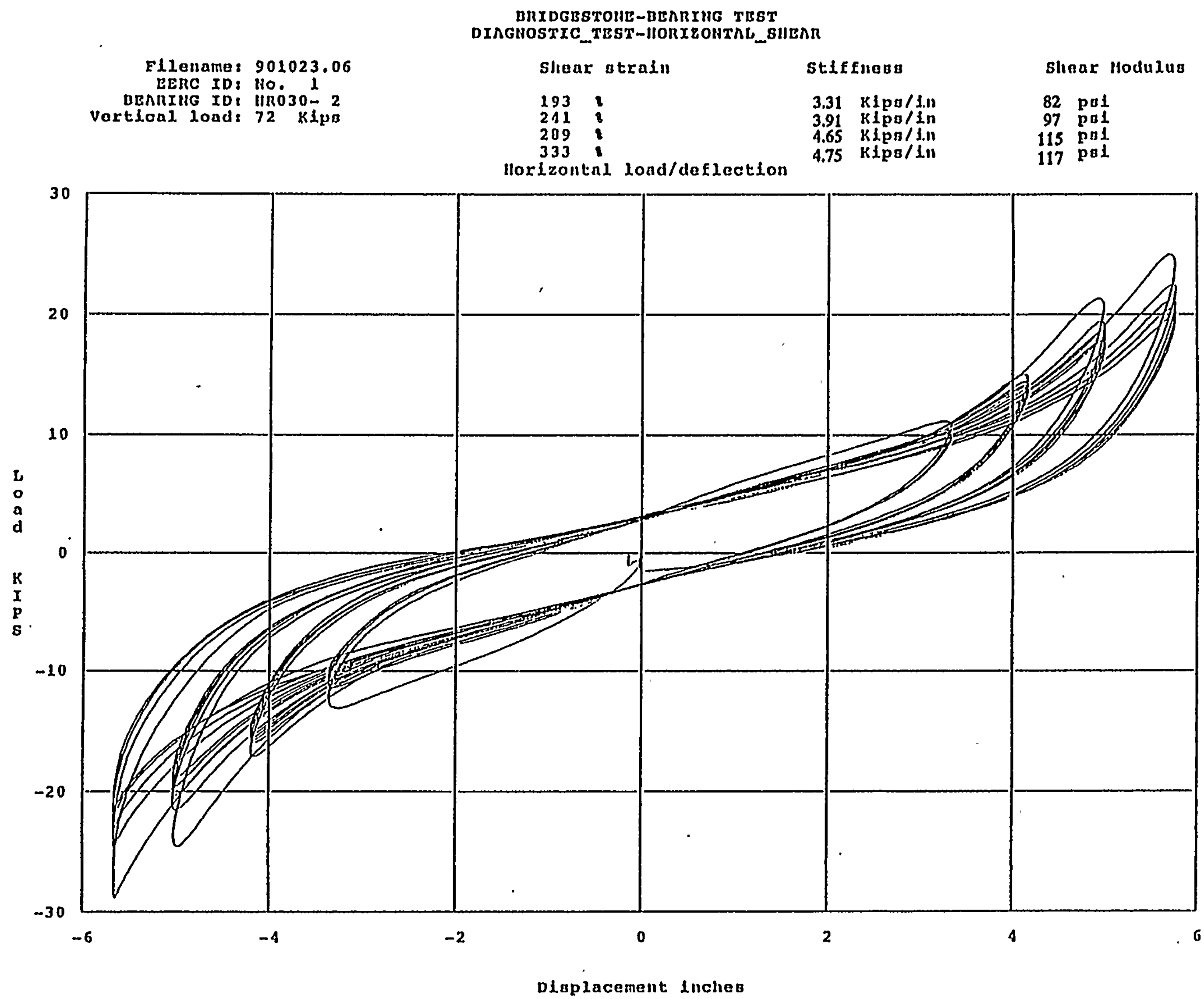


Fig. 7.11 Force Displacement Plots for Sequence III -1200 psi (6.90 MPa) Vertical Pressure

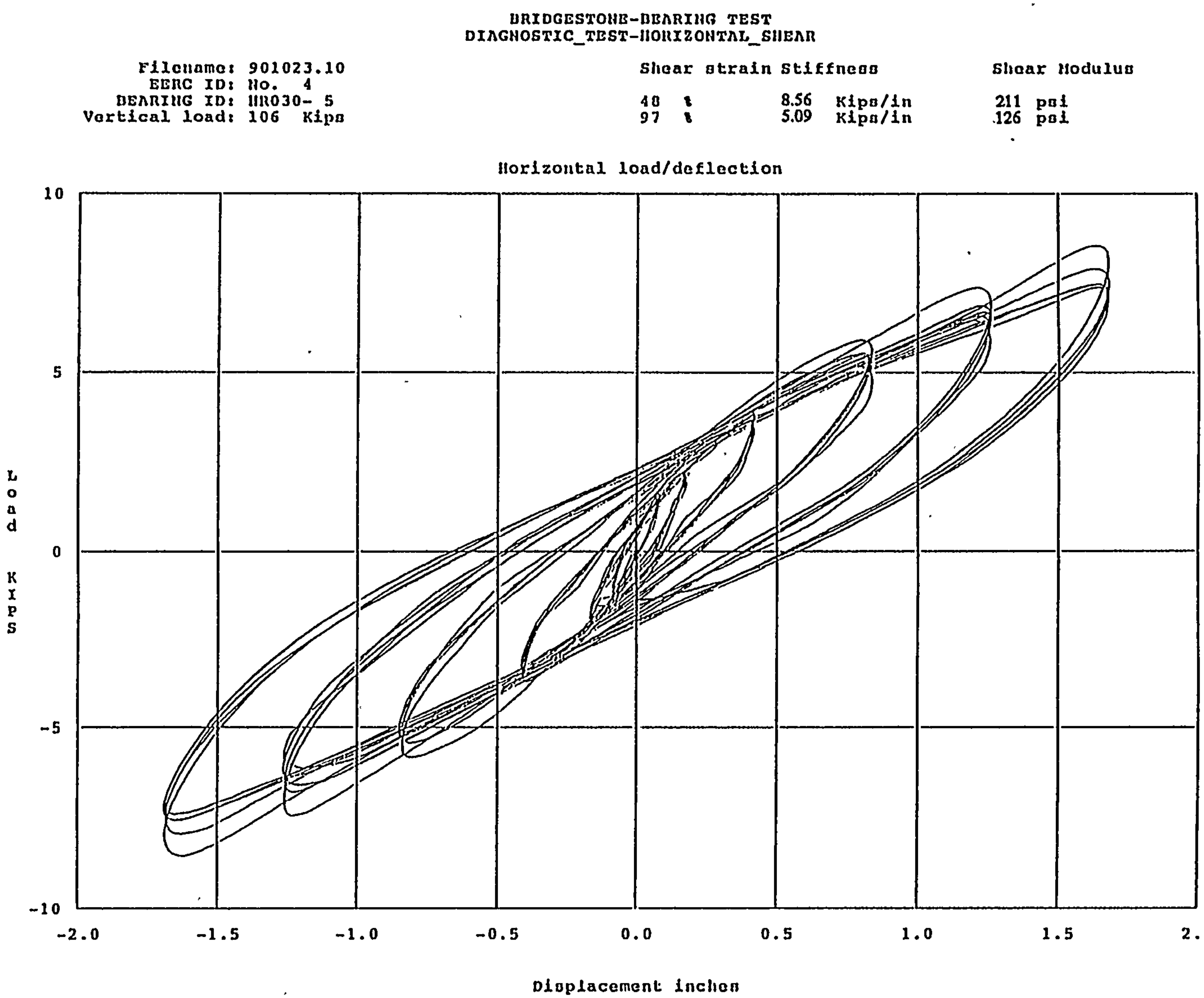


Fig. 7.12 Force Displacement Plots for Sequence I - 1500 psi (10.34 MPa) Vertical Pressure

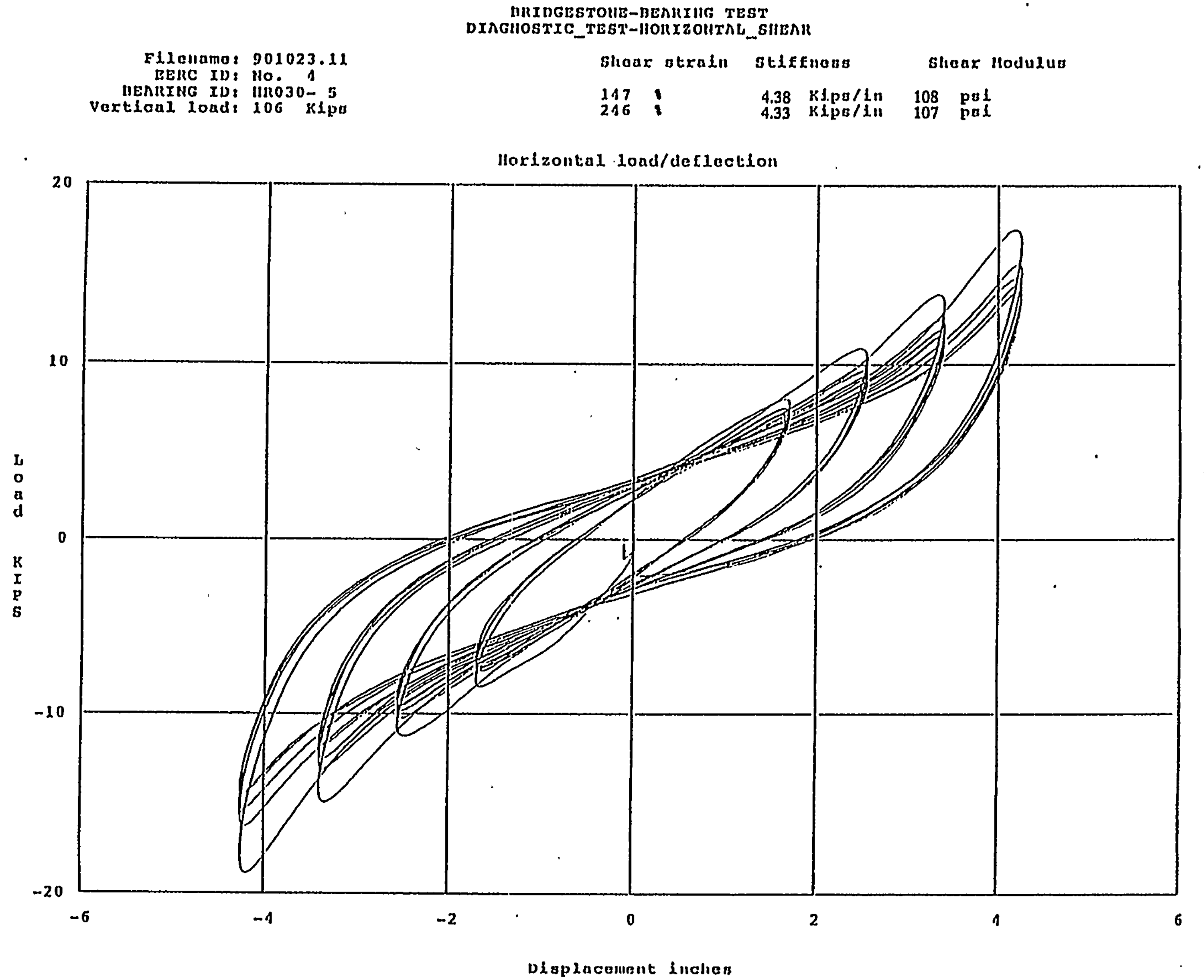


Fig. 7.13 Force Displacement Plots for Sequence II - 1500 psi (10.34 MPa) Vertical Pressure

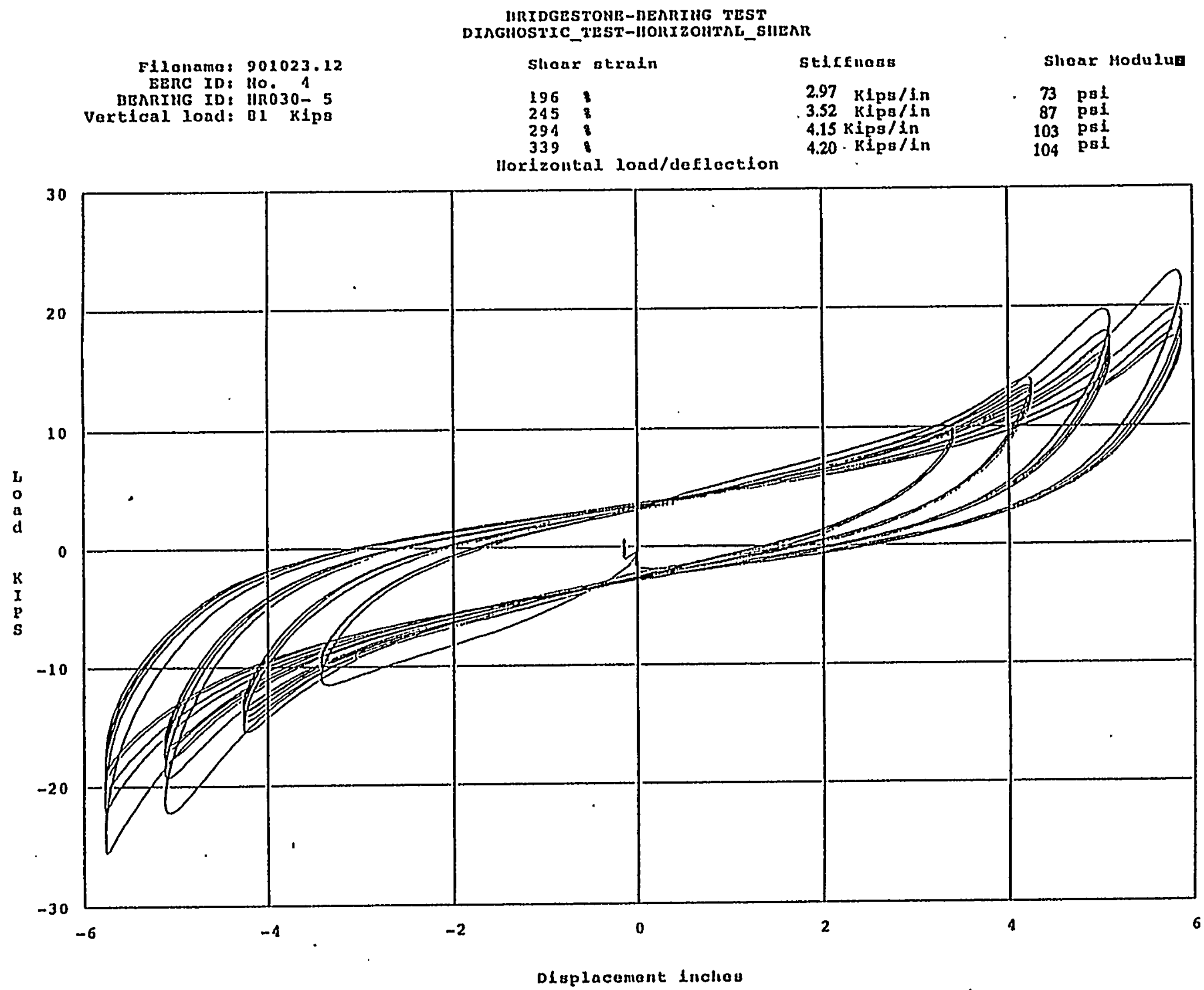


Fig. 7.14 Force Displacement Plots for Sequence III - 1500 psi (10.34 MPa) Vertical Pressure

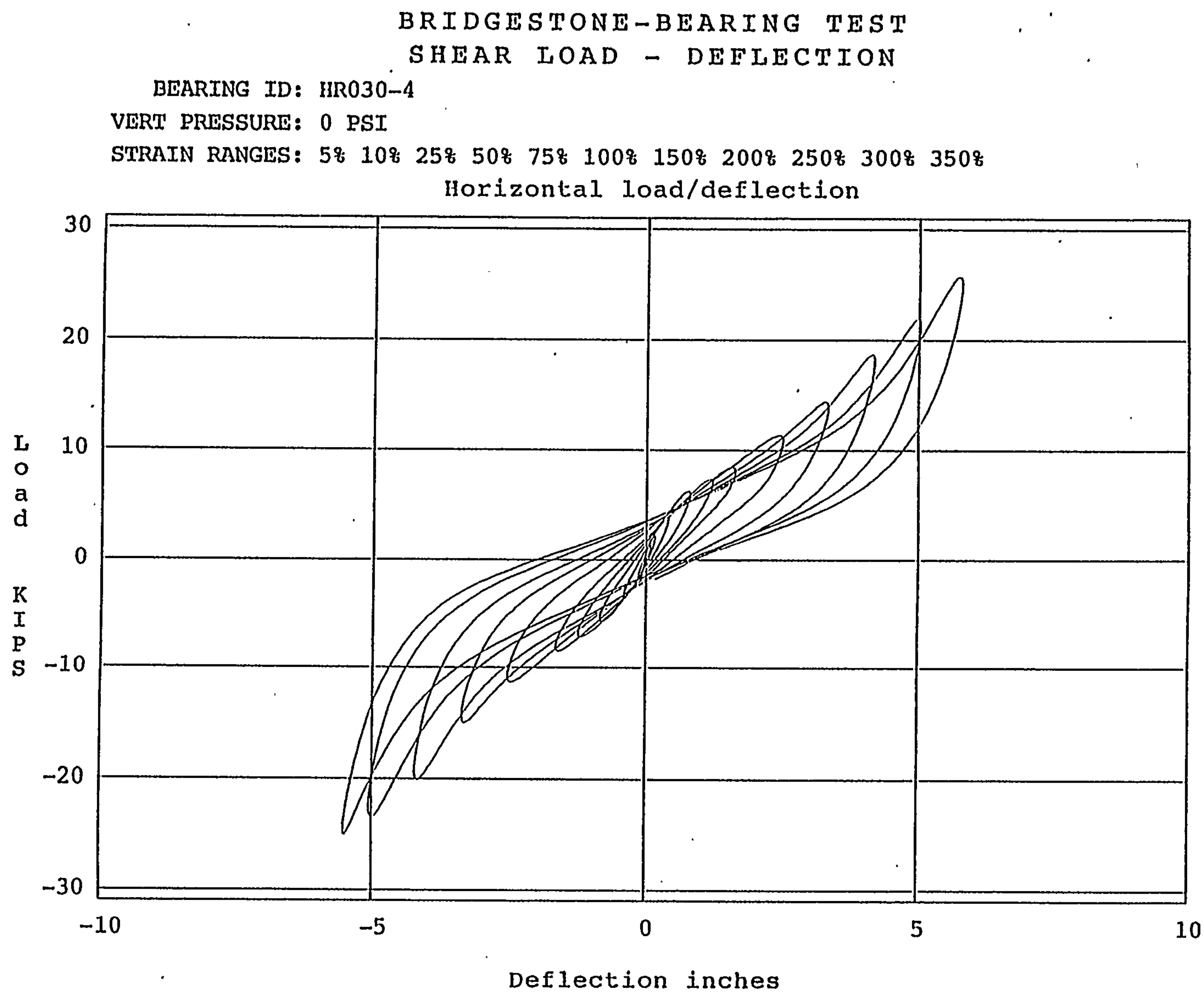


Fig. 7.15 First Cycle Force Displacement Plots at Zero Vertical Pressure

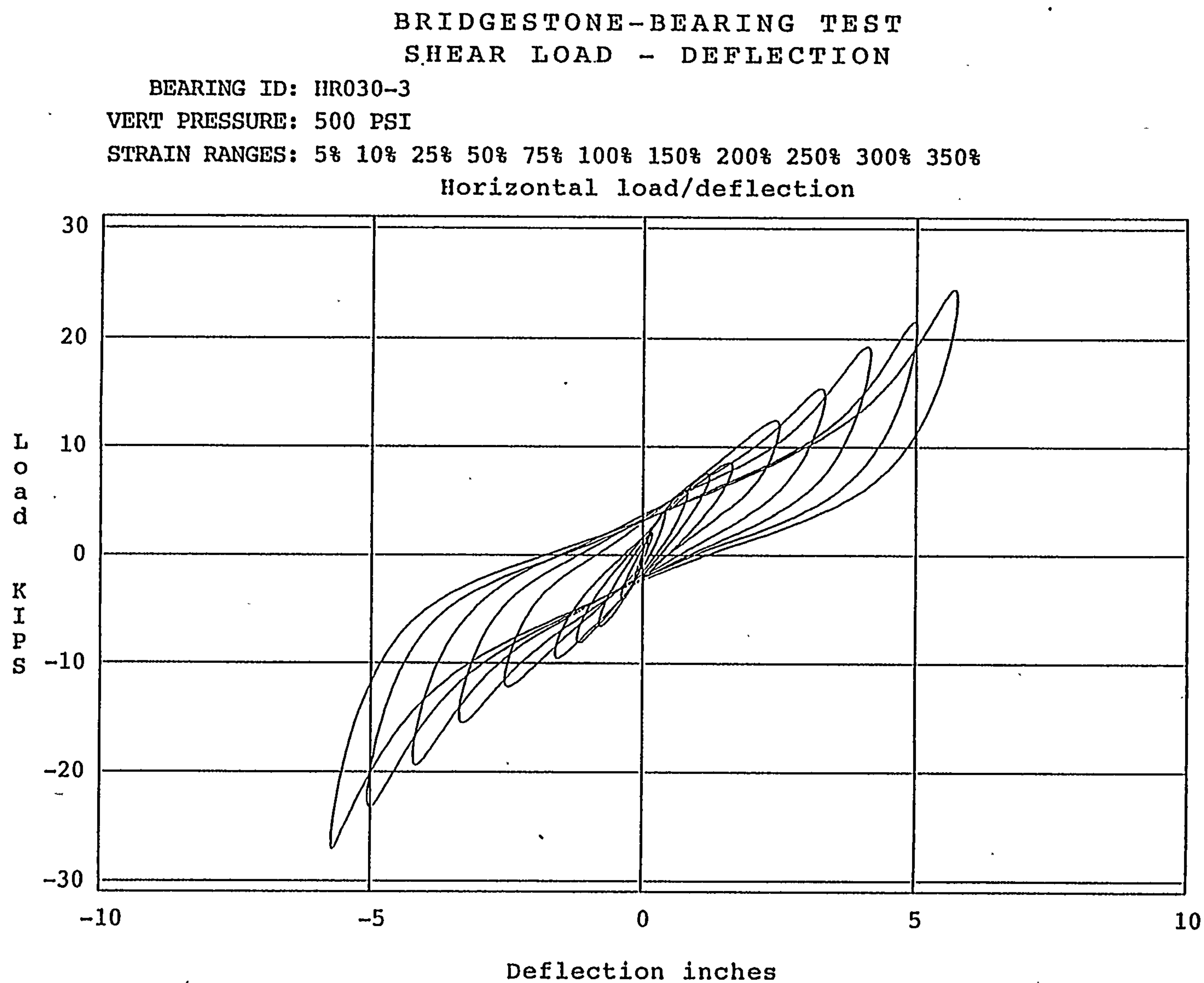


Fig. 7.18 First Cycle Force Displacement Plots at 500 psi (3.45 MPa) Vertical Pressure

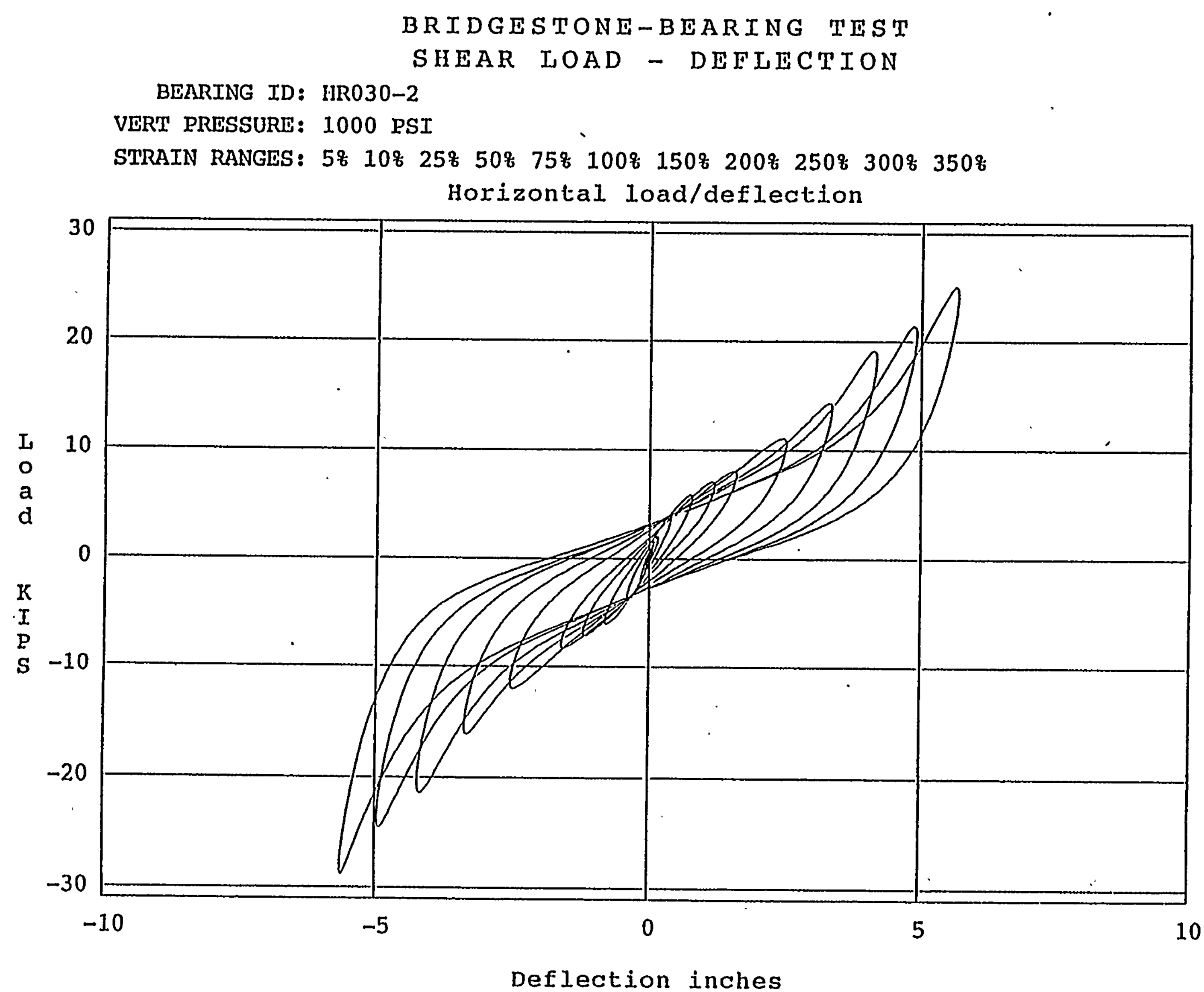


Fig. 7.17 First Cycle Force Displacement Plots at 1000 psi (6.90 MPa) Vertical Pressure

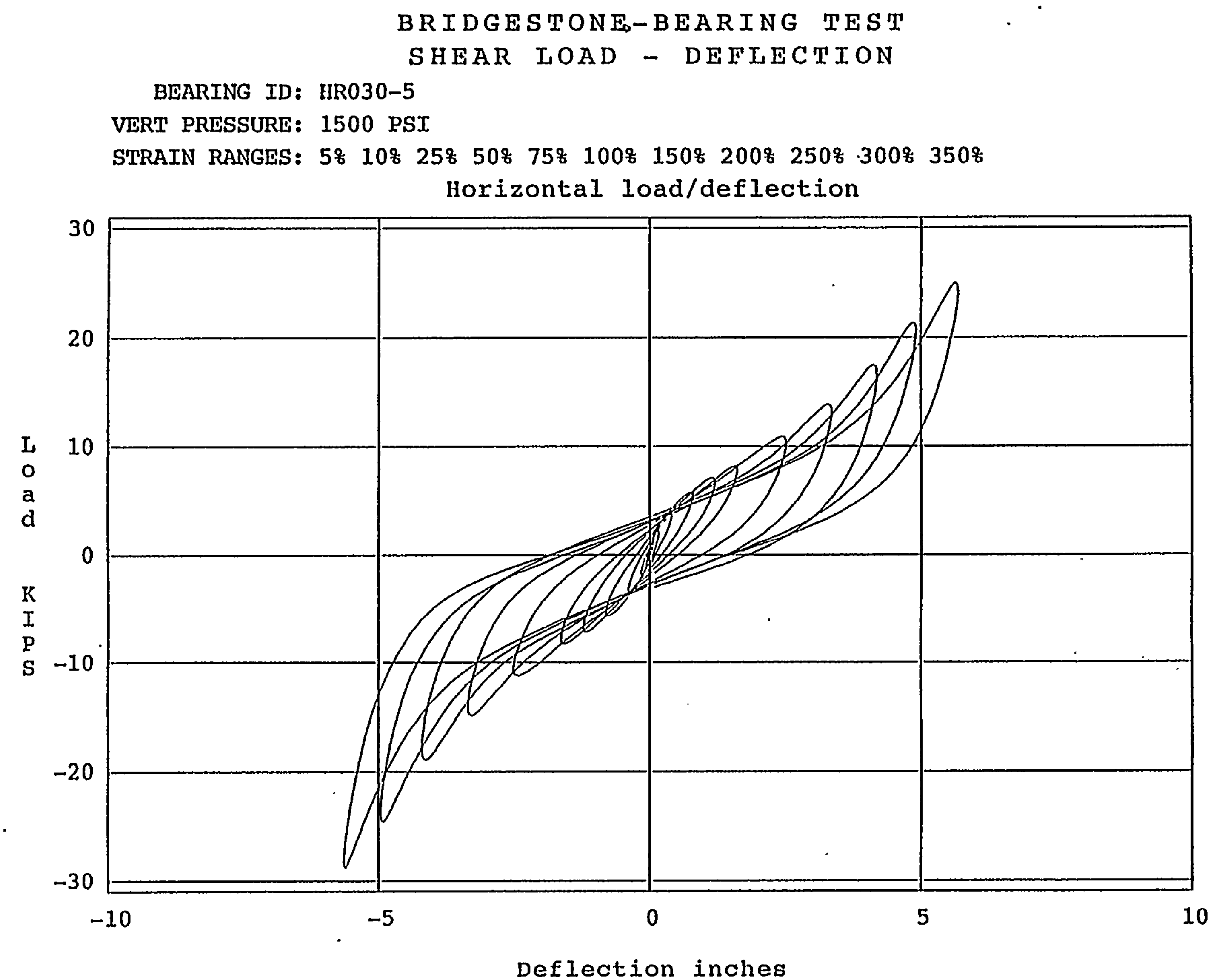


Fig. 7.16 First Cycle Force Displacement Plots at 1500 psi (10.34 MPa) Vertical Pressure

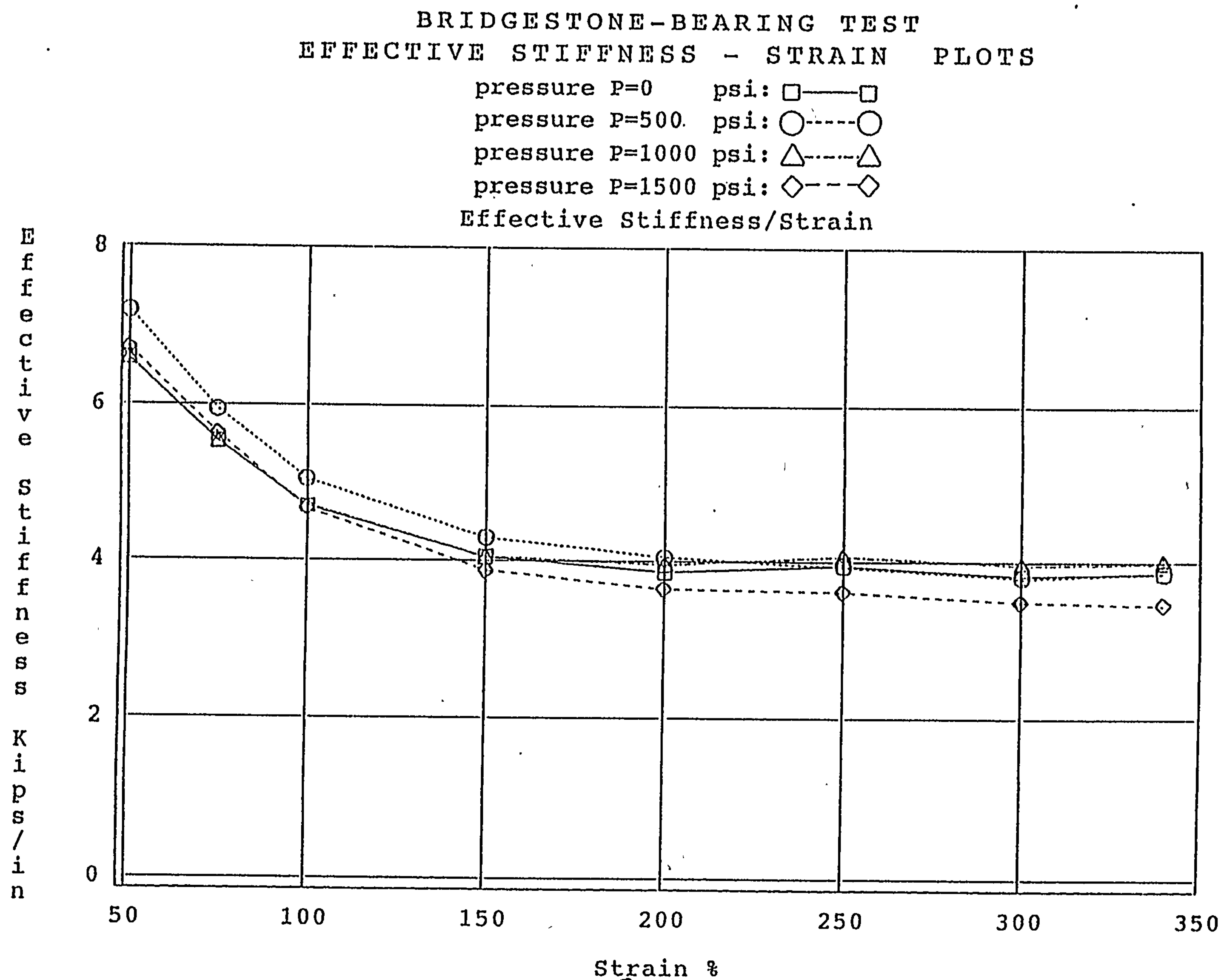


Fig. 7.19 Influence of Vertical Load and Strain on Bearing Stiffness

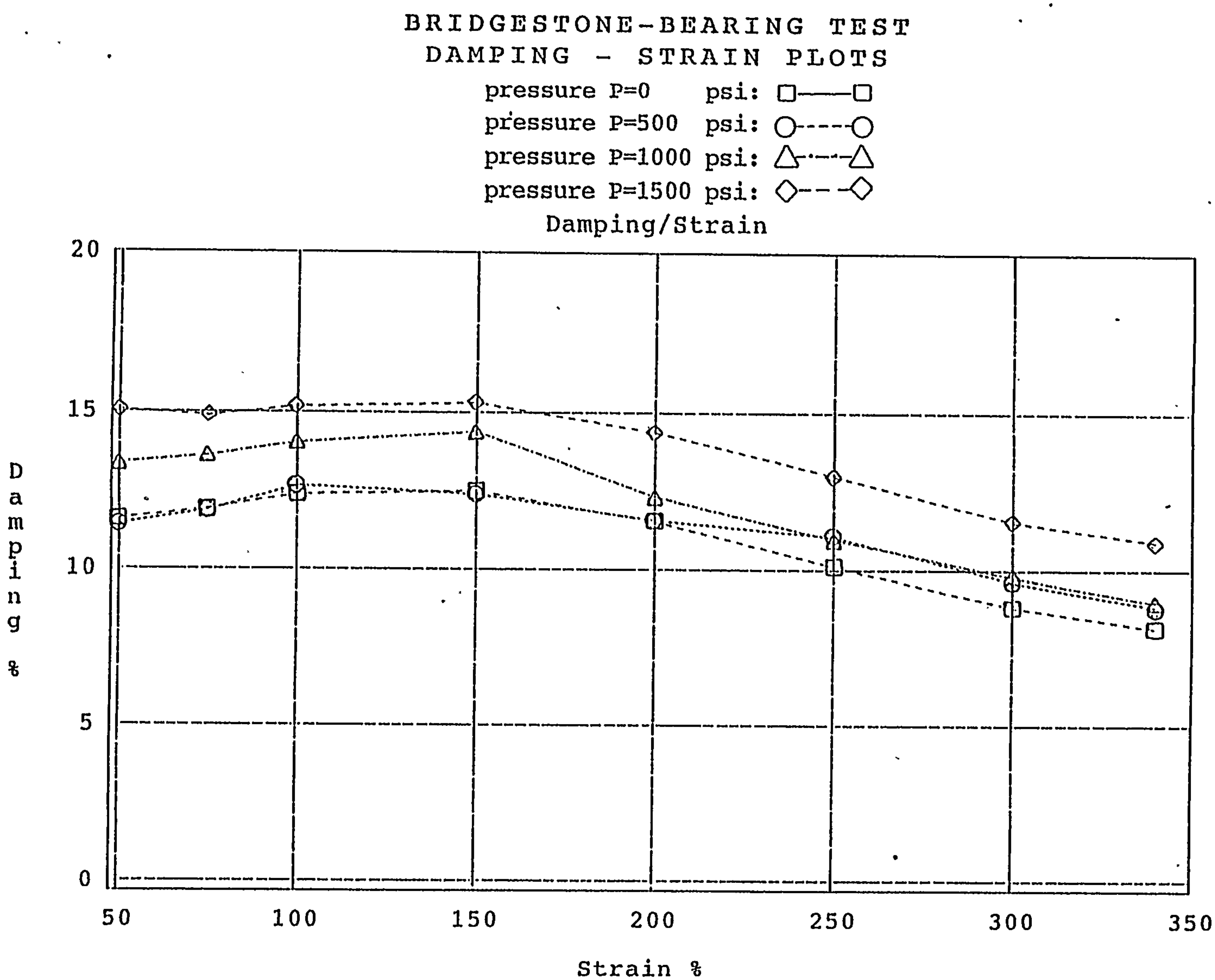


Fig. 7.20 Influence of Vertical Load and Strain on Bearing Damping

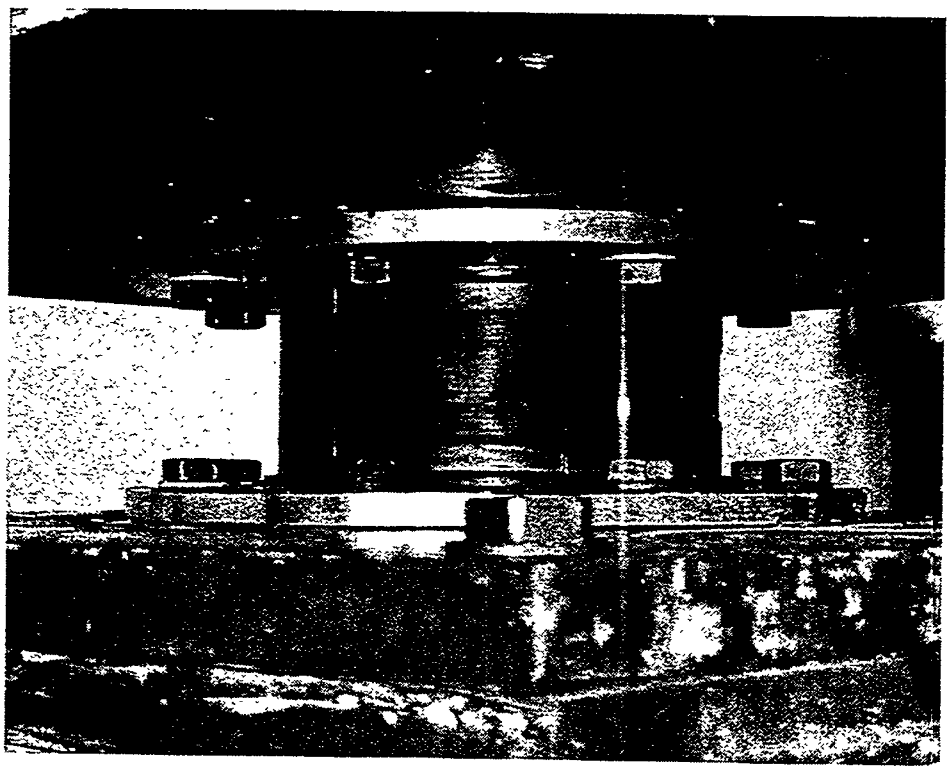


Fig. 7.21 Configuration of Bearing before Failure Tests

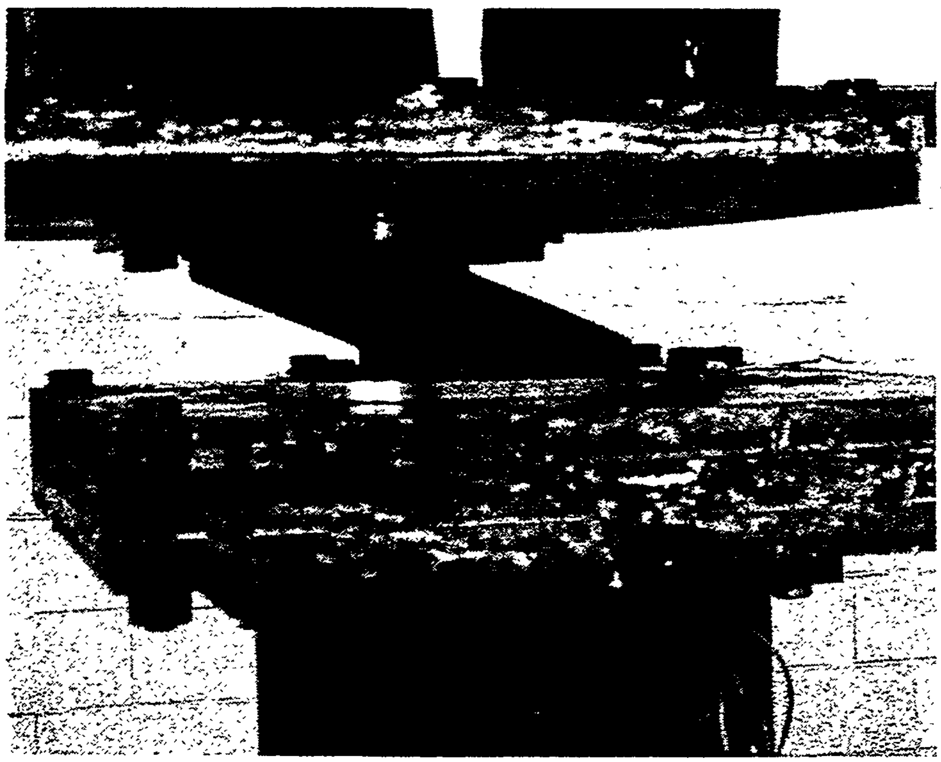


Fig. 7.22 Bearing Appearance at 200% Strain

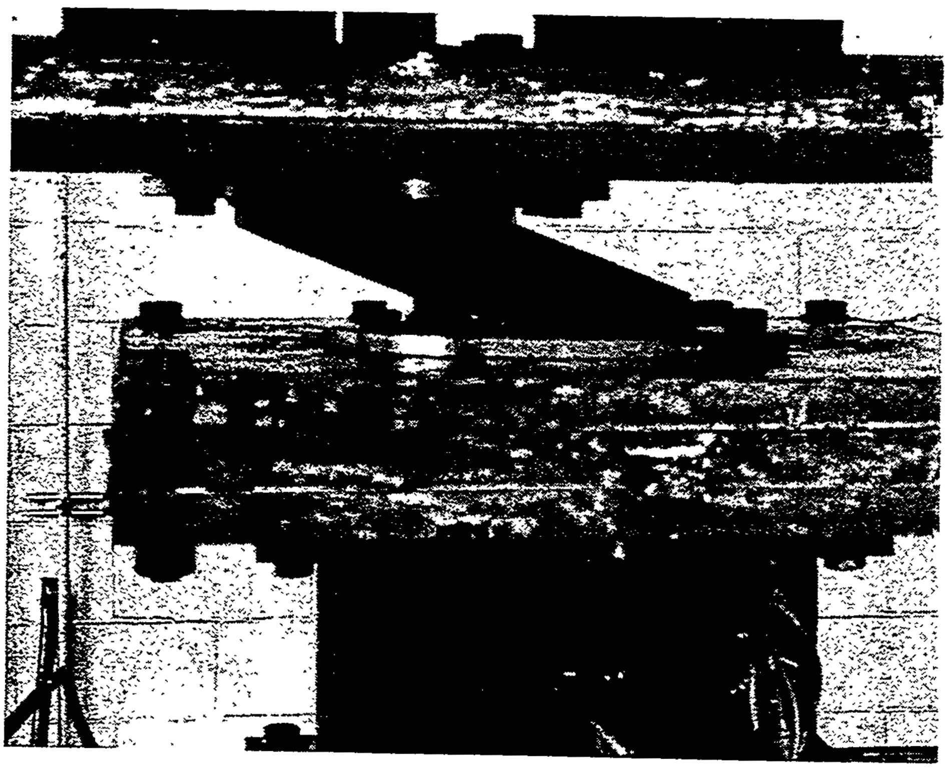


Fig. 7.23 Bearing Appearance at 300% Strain

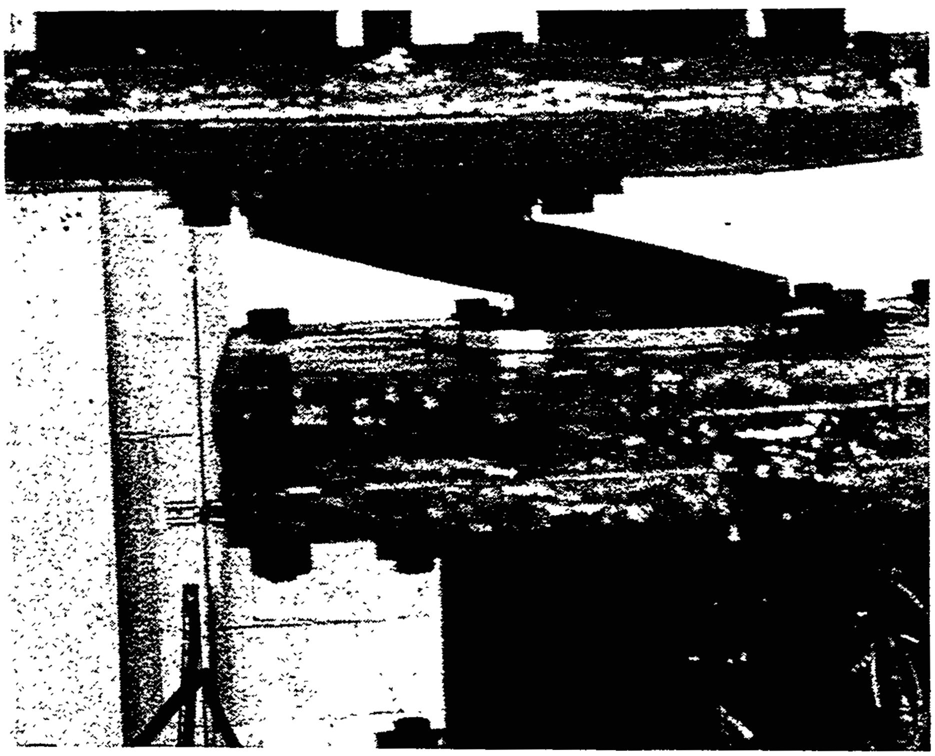


Fig. 7.24 Bearing Appearance at 500% Strain

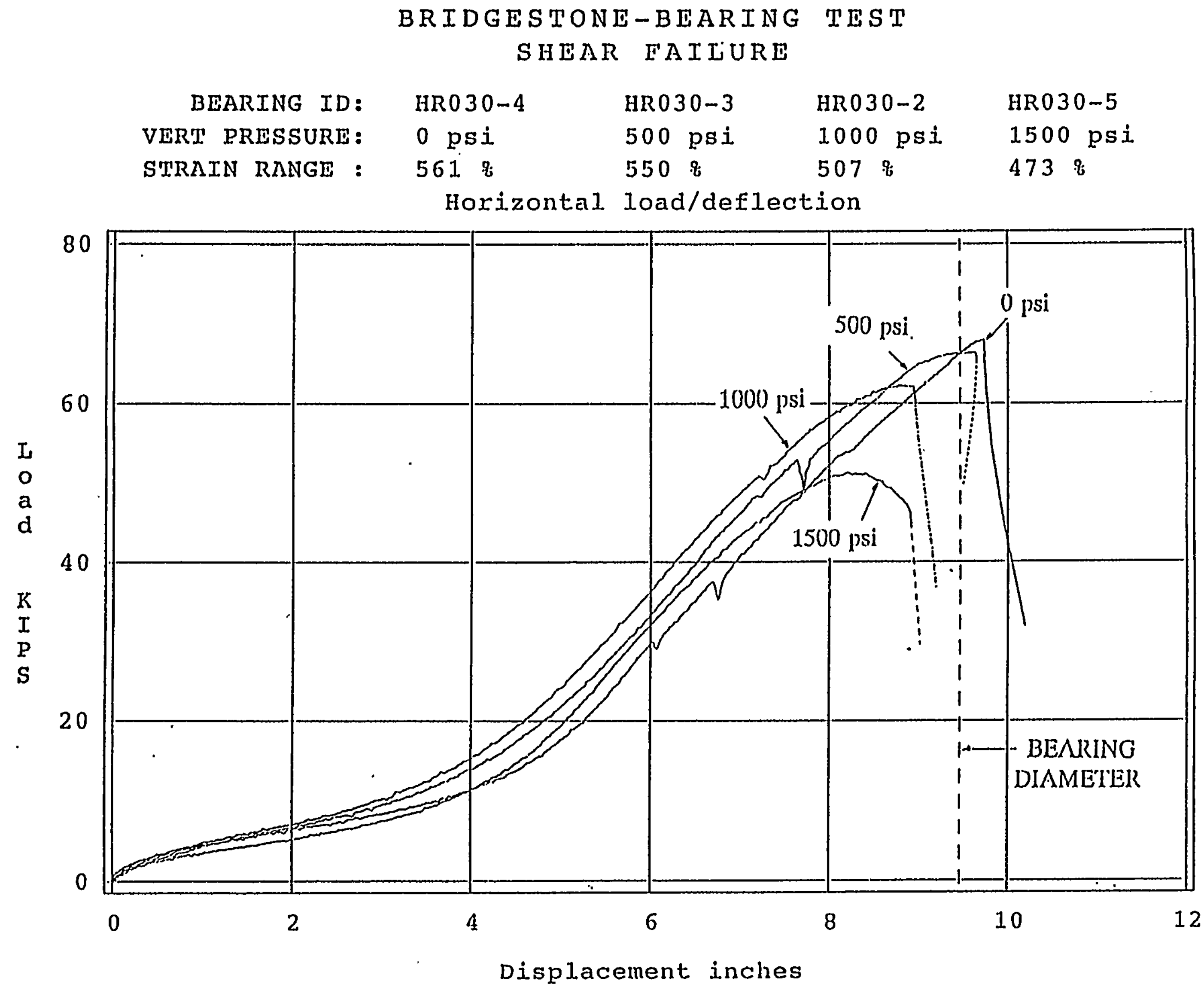


Fig. 7.25 Force Displacement Curves for Bearings in Failure Tests

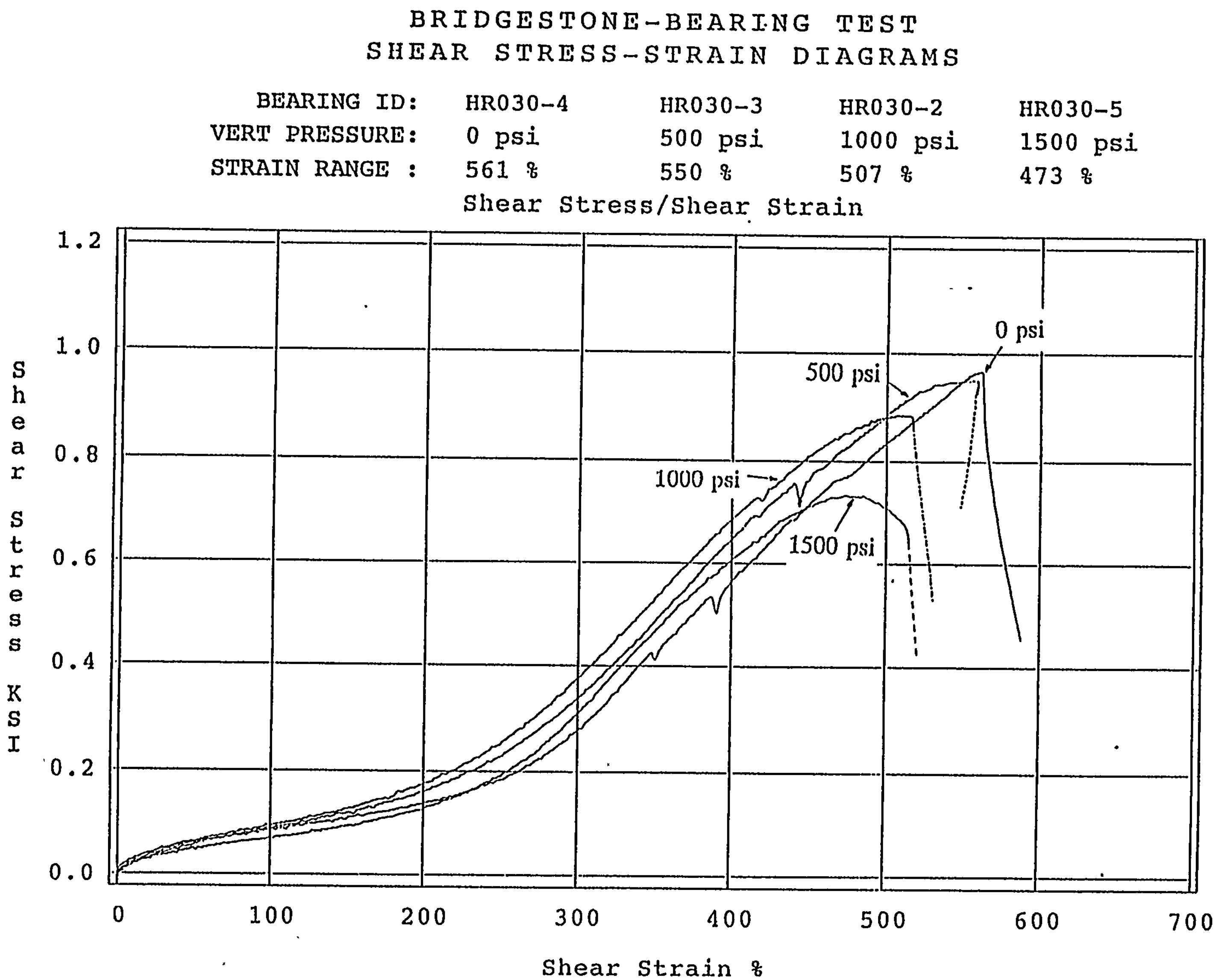
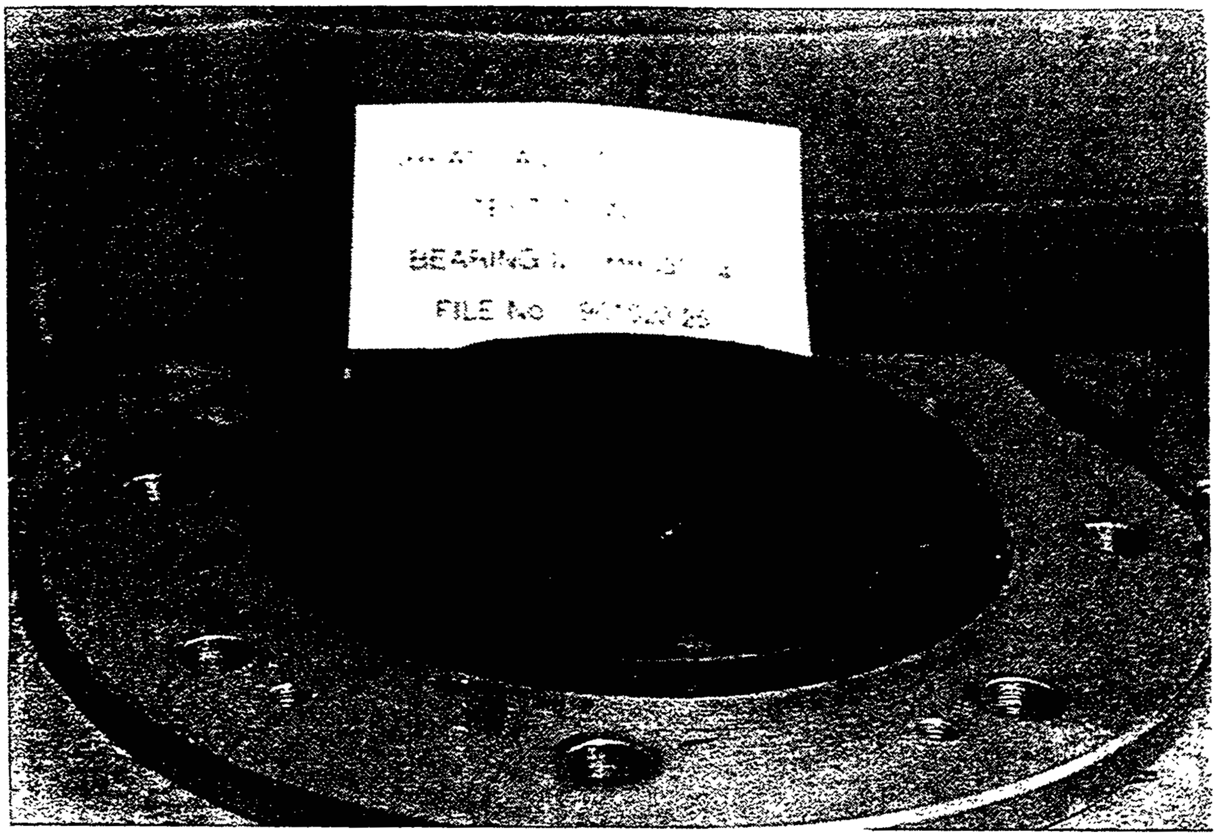


Fig. 7.26 Stress-Strain Curves for Bearings in Failure Tests



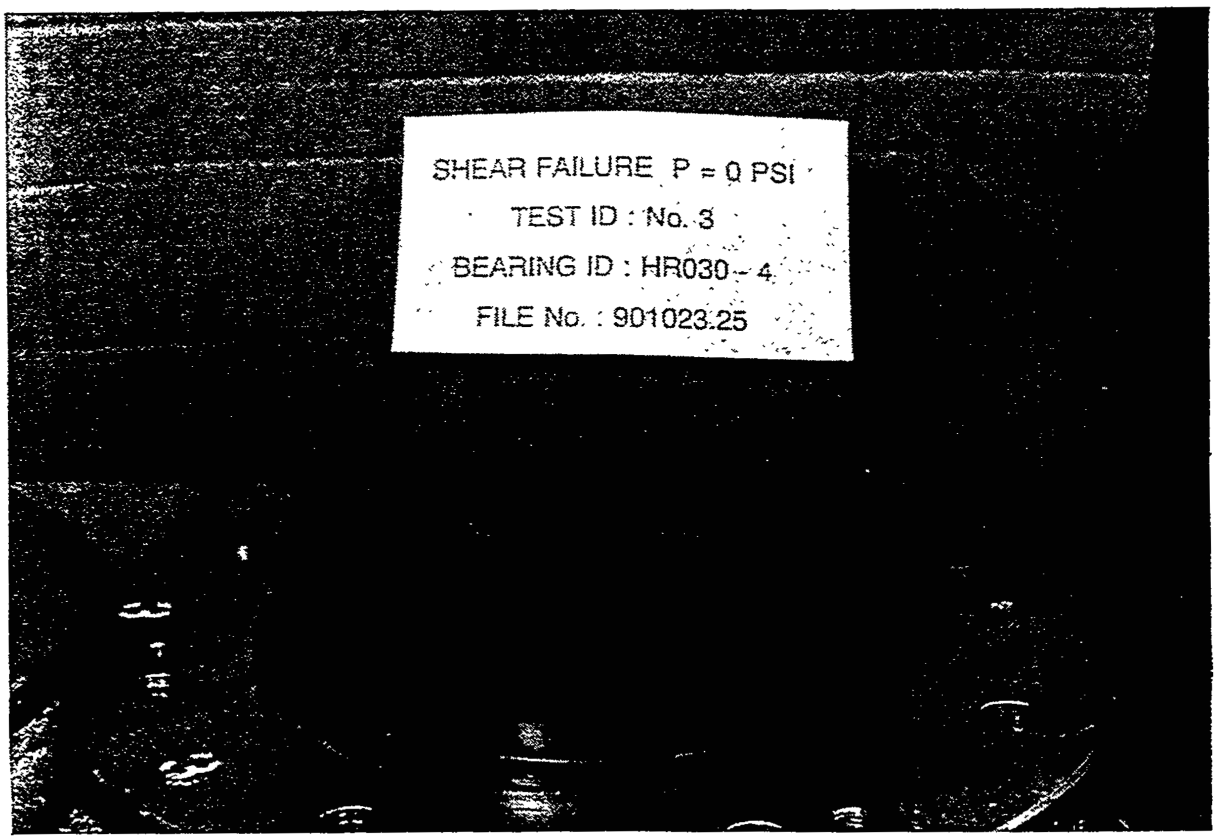


Fig. 7.27 Appearance of Failure Surface at Zero Vertical Pressure

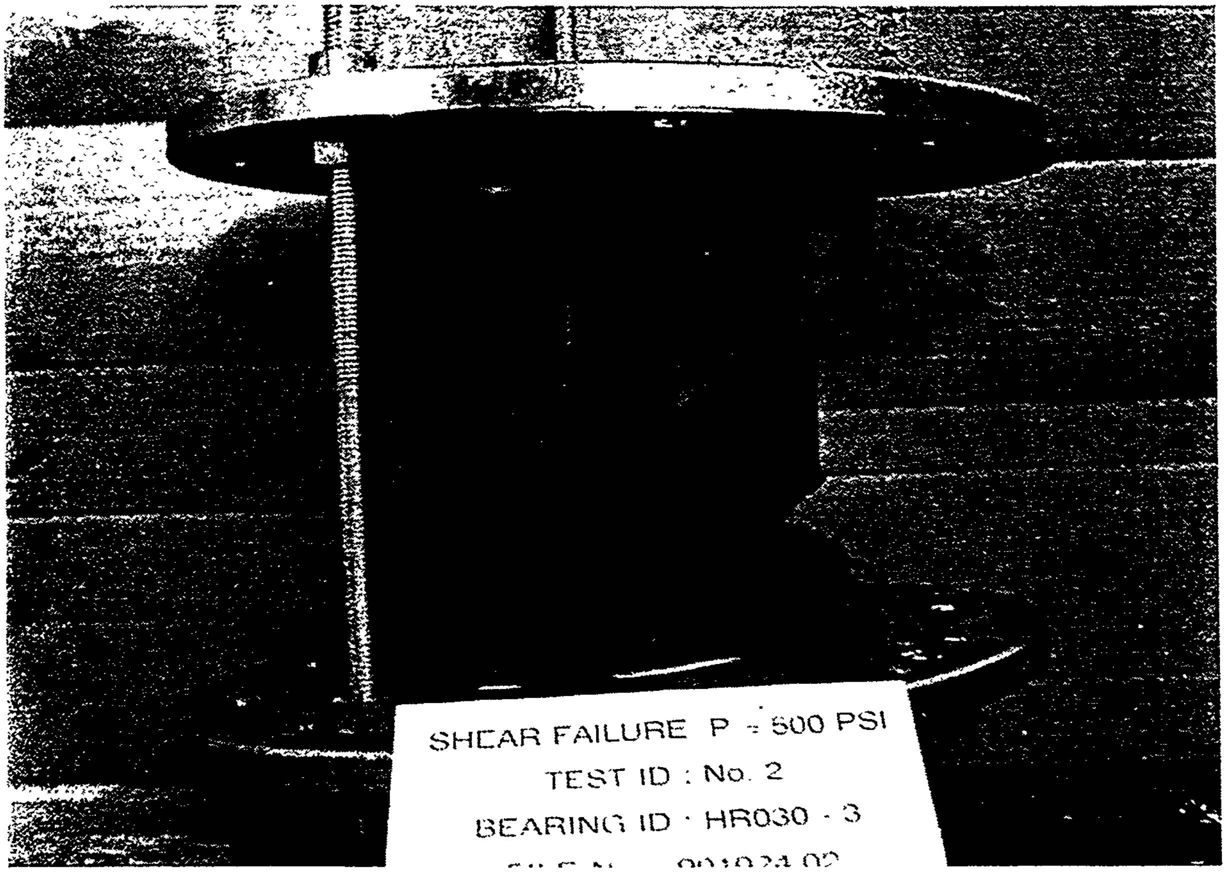
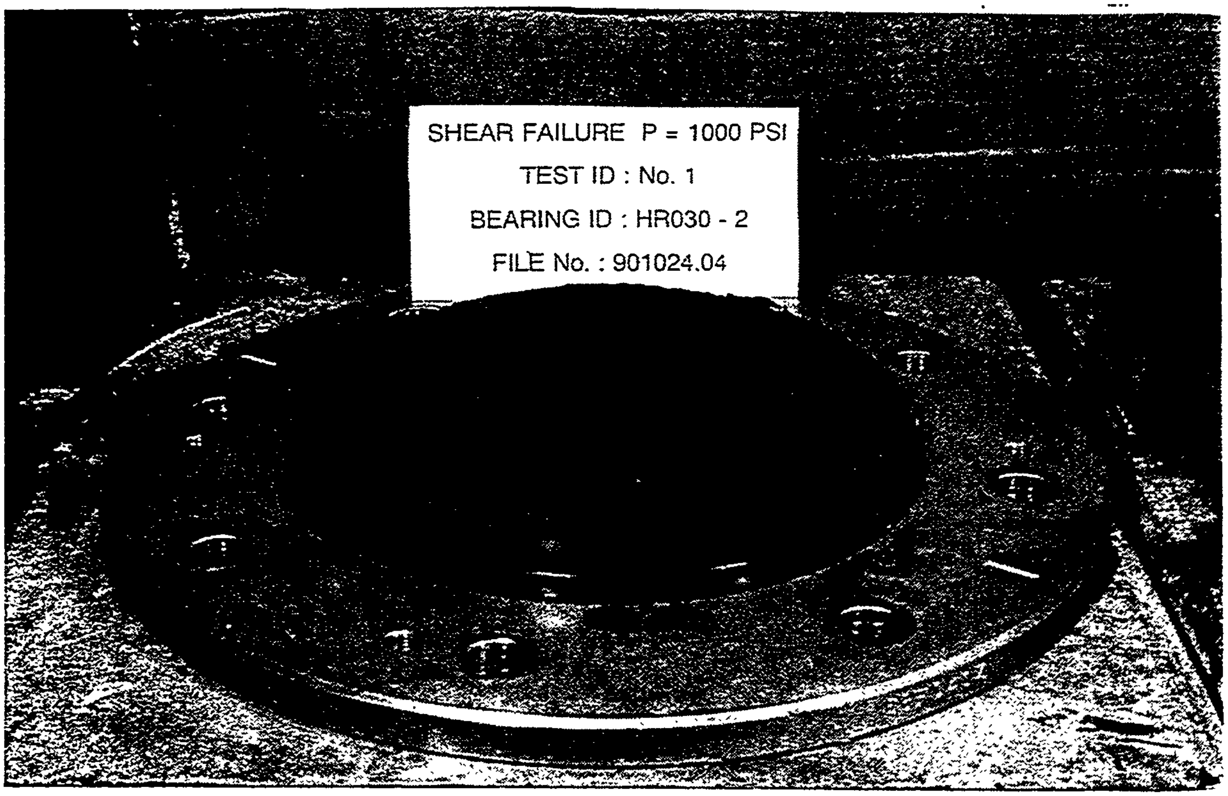


Fig. 7.28 Failed Bearing (held apart for clarity) at 1000 psi (6.90 MPa) Vertical Pressure



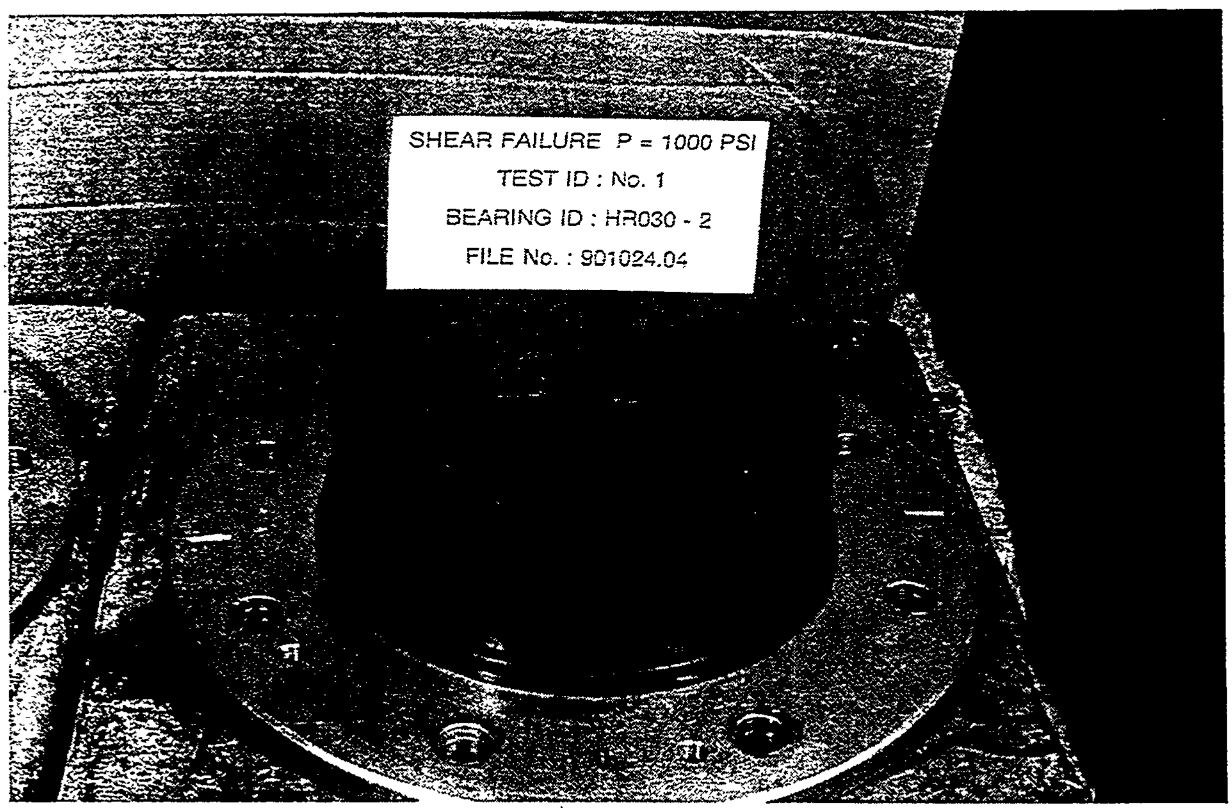


Fig. 7.29 Appearance of Failure Surface at 1000 psi (6.90 MPa) Vertical Pressure

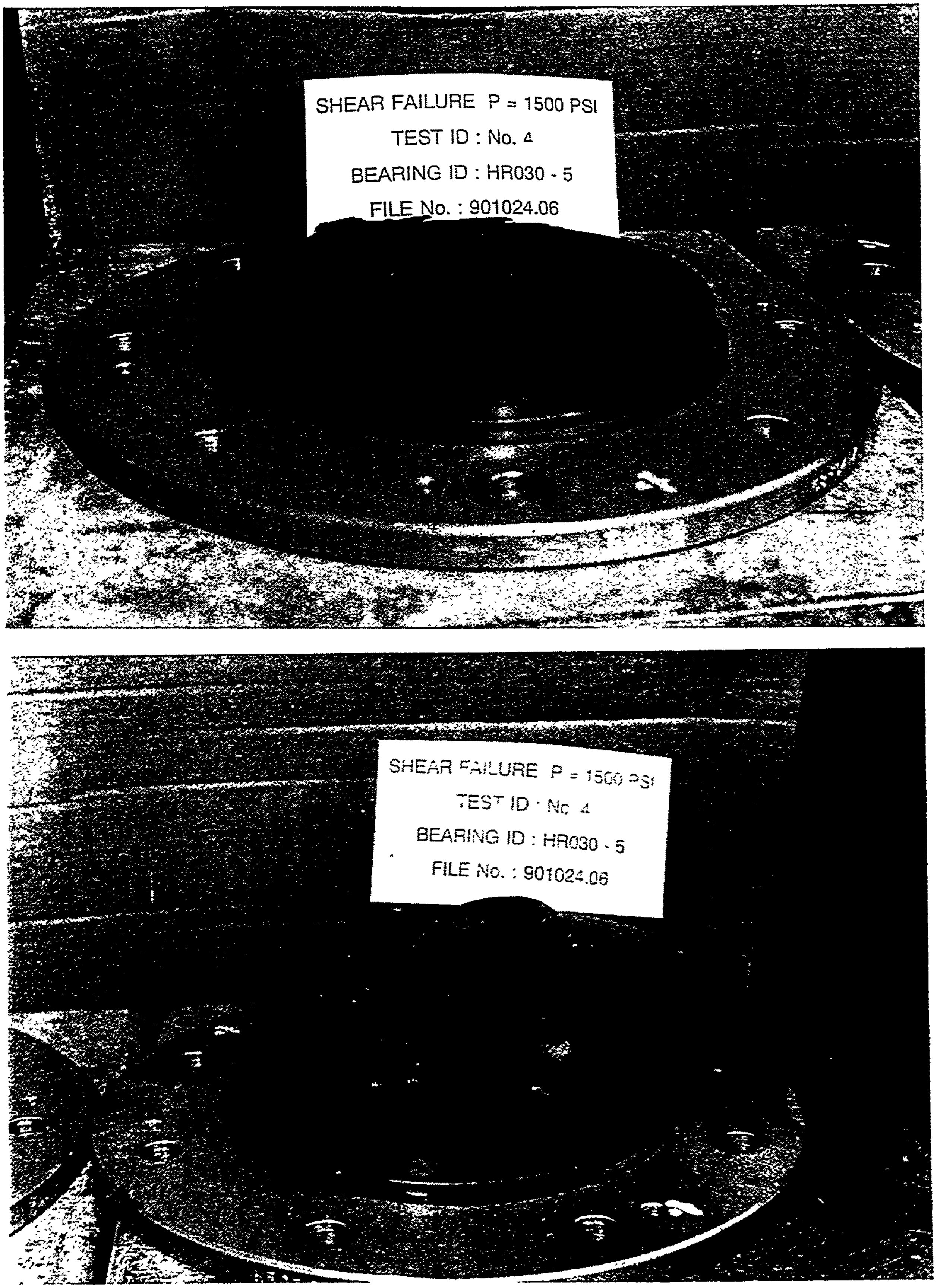


Fig. 7.30 Failed Bearing at 1500 psi (10.34 MPa) Vertical Pressure

LECTURE 5

Design Process for Multilayer Elastomeric Bearings

by

James M. Kelly
Professor of Civil Engineering
Earthquake Engineering Research Center
University of California
Berkeley, California 94720

Short Course on Seismic Base Isolation
Department of Energy

August 10 - August 14, 1992

Department of Energy
Short Course on Seismic Base Isolation

J.M. Kelly

August 10-14, 1992

LECTURE 5

Design Process for Multilayer Elastomeric Bearings

Introduction

The design process for elastomeric bridge bearings is governed by a number of code specifications that reflect the fact that the loads to which the bearings are subjected are well-defined and happen on a regular basis. If these provisions were to be applied to elastomeric bearings for isolation, they would result in unnecessarily conservative designs. In the design of seismic isolation bearings for buildings, it has to be recognized that codes such as the UBC 1991 require the isolation system to be designed for *very severe seismic loading* and that this loading may be interpreted as ultimate state loading and that the isolators should be designed reflecting this; conservatism is already incorporated in the specified site seismicity and need not be further increased by over-conservative design of the isolators, and further, the extreme loads to which the isolator may be subjected will occur, if at all, no more than once or twice over the lifetime of the structure.

Preliminary Bearing Design Process

The preliminary design of a bearing in an isolation system begins with the determination of the load to be carried by the bearing. In most buildings, the design load at each column (based on dead load plus fixed partitions, equipment, furniture, etc.) can vary quite widely. It will generally be necessary to minimize the number of different types of isolators, and thus, the first decision to be made will be how many different bearing types to design. Once this decision is made, the design load for each bearing type can be selected to minimize the variation of load on that type.

After the design load, W, is selected, the design specifications will fix the following quantities:

f_H = horizontal frequency or

T = horizontal period

f_V = vertical frequency

γ_{max} = maximum permissible shear strain

D = design displacement (from response spectrum or SEAOC formula)

Two safety factors will also be needed. The first is the safety factor against buckling: this should be based on dead load plus live load on the bearing. The second is the safety factor against roll-out which should be based on the minimum load on the bearing.

The design quantities to be selected are a or ϕ, t_r, t, n, G, and h, where

ϕ = diameter of a round bearing

t_r = total rubber thickness in the bearing

t = thickness of individual layer

n = number of layers

G = shear modulus of elastomer

h = total height of the bearing

If the SEAOC formula is used for initial design we have

$$D = \frac{10NZST}{B} \tag{5.1}$$

and the total thickness of rubber, t_r, should not be less than

$$t_r = \frac{D}{\gamma_{max}} \tag{5.2}$$

The quantities N, Z, and S will be specified and depend on the seismicity and soil conditions at the site. The damping factor, B, will depend on the choice of elastomer and the level of strain. A reasonable approach is to assume a damping factor based on experience with previously used rubber compounds and, if necessary, to recompute D if the damping assumed is found to be too high or too low for the rubber compound selected.

The horizontal stiffness of the bearing is given by

$$K_H = \frac{GA}{t_r} \tag{5.3}$$

and is related to the design load, W, through

$$K_H = \frac{W}{g}(2\pi f_H)^2 = \frac{GA}{t_r} \tag{5.4}$$

Dividing both sides by A gives

$$\frac{G}{t_r} = \frac{p}{g}(2\pi f_H)^2 \tag{5.5}$$

A guide for initial design is that p should normally be about 1000 psi, and with this selected, the choice of G and t_r can be made. It is then necessary to verify that a compound with the required G is available and that the damping in the compound corresponds to that assumed when D was calculated by the code formula. If the value required for G cannot be matched by any available compound, p can be adjusted up or down until an available rubber modulus is found.

With G, t_r, and p selected, the overall size of the bearing (a *or* ϕ) can then be selected. The next step is to determine the number of layers of elastomer. The thickness of an individual layer controls the vertical stiffness through the shape factor, S, which is a dimensionless measure of the thinness of a layer.

For a square pad, S is given by

$$S = \frac{a}{4t}$$

and for a circular pad

$$S = \frac{\phi}{4t}$$

The estimation of the compression stiffness of a bearing is dependent on knowing the effective compression modulus, E_c, of a single layer. This has been determined using elastic analysis for a number of different pad configurations. The effective compression modulus for a circular pad is

$$E_c = 6GS^2 \tag{5.6}$$

and for a square pad is

$$E_c = 6.75GS^2 \tag{5.7}$$

For a circular pad with a large hole, e.g., with an internal diameter greater than one tenth of the external diameter, the result is

$$E_c = 4GS^2 \tag{5.8}$$

and here the shape factor S is given by

$$S = \frac{\phi_0 - \phi_i}{4t} \tag{5.9}$$

The shear modulus to be used in these formulae is somewhat uncertain since in highly filled rubbers this varies with shear strain level and in compression, the shear strain varies through each layer.

In computing the compression stiffness, the bending stiffness, and the buckling load of a bearing it is necessary to assume a value for the shear modulus G. In highly filled compounds this varies with shear strain (although not severely) over the range 30—100 %. The same analysis that provides the compression modulus, E_c, gives a value for the maximum shear strain due to compression, γ_c, as a function of the compression strain ε_c. This is

$$\gamma_{c_{max}} = 6S\varepsilon_c \tag{5.10}$$

The shear strain, γ_c in the isolator varies with position in the pad, being highest at the edges of the bonded surfaces. The above value, being the maximum, is not appropriate for use in determining the required value of the shear modulus. A more appropriate value is the average, in the sense of root-mean-square average, over the whole volume of the pad.

This is

$$\gamma_{c_{ave}} = \sqrt{6}S\varepsilon_c \tag{5.11}$$

This provides a method for estimating the shear modulus for inclusion in Eqs. 5.6, 5.7, or 5.8. An extensive series of tests on the compression of pads was carried out by C.J. Derham [1982] and he found that a good approximation for both square and circular pads was to take

$$E_c = 5.6GS^2 \tag{5.12}$$

where the shear modulus used was that at 50 % shear strain.

The vertical stiffness of the entire bearing is

$$K_V = \frac{E_cA}{t_r} \tag{5.13}$$

When the vertical frequency, f_V, is specified, then

$$\frac{K_V}{K_H} = \left[\frac{f_V}{f_H}\right]^2$$

and in turn

$$\frac{K_V}{K_H} = \frac{E_c}{G}$$

Thus, if the shape factor is moderate, the pad is circular, and Eq. (7.10) is used, the required shape factor is given by

$$6S^2 = \left[\frac{f_V}{f_H}\right]^2$$

or

$$S = \frac{1}{2.4}\frac{f_V}{f_H} \tag{5.14}$$

The approximate formula for E_c is limited to shape factors of less than 10 so that this result is not good if $f_V > 24f_H$. If we select the horizontal period to be 2 seconds, then the formula holds for $f_V < 12$ Hz. This is quite high, as for most buildings the vertical frequency will be in the range 8—12 Hz, but in certain applications, for example in nuclear structures, the vertical frequency may be specified to be much higher and high-shape factors will result. In these cases, the high-shape factor formulae that include the effects of compressibility can be used.

The simple formula for E_c is based on the assumption of incompressibility in the elastomer and when E_c becomes comparable to the bulk modulus of the rubber this no longer holds. A more refined analysis which includes bulk compressibility is needed for these examples. It can be shown that E_c can never exceed the bulk modulus, K, and for filled natural rubber $K \approx$ 300,000 psi. Since G is approximately 150 psi, this means that the maximum value of f_V/f_H is about 44, and thus for a 2 second horizontal period, f_V cannot be greater than 22 Hz.

Once S is determined, the layer thickness, t, is obtained and the number of layers chosen. Since the number of layers must be an integer it may be necessary to adjust t to ensure $nt = t_r$.

It remains to select the thickness of the steel shims between each layer of rubber, and the end plates at the top and bottom of the bearing. There are no design formulae for these quantities. The shim thickness, t_s, is generally taken as not less than 1/10-inch and not greater than 1/8-inch and the end plates are usually between 3/4-inch to 1 1/2-inches thick depending on the overall size of the bearing. The total height, h, used in the stability estimates can now be determined.

To verify the stability of the bearing against vertical load we must calculate the buckling load of the bearing. This is given by

$$P_{cr} = \sqrt{P_S \, P_E} \tag{5.15}$$

where

$$P_S = GA_S \tag{5.16}$$

$$P_E = \frac{\pi^2 EI_{eff}}{h^2} \tag{5.17}$$

The appropriate choice of G for this computation is the value at 20% shear strain. The quantity A_S is the actual area A modified by h/t_r, since it can be assumed that the end plates and the shim do not deform in shear.

The value of EI_{eff} is very close to

$$\frac{1}{3} E_C I \cdot \frac{h}{t_r}$$

Using these values and the design load, W, we have

$$S.F. = \frac{P_{cr}}{W} = \frac{\left\{ GA \frac{h}{t_r} \pi^2 \frac{1}{3} \frac{E_c I}{h^2} \frac{h}{t_r} \right\}^{1/2}}{W}$$

$$= \frac{\pi}{\sqrt{3}} \left\{ \frac{GAg}{t_r W} \right\}^{1/2} \left\{ \frac{E_c Ag}{t_r W} \right\}^{1/2} \left\{ \frac{I}{A} \right\}^{1/2} \tag{5.18}$$

Now

$$\frac{GA}{t_r W} = (2\pi f_H)^2$$

and

$$\frac{E_c A g}{t_r W} = (2\pi f_V)^2$$

So that the safety factor is given approximately by

$$S.F. = \frac{\pi}{\sqrt{3}\, g}\,(2\pi f_H)\,(2\pi f_V)\left[\frac{I}{A}\right]^{1/2} \tag{5.19}$$

For a square bearing

$$\left[\frac{I}{A}\right]^{1/2} = \left[\frac{\frac{1}{12}a^4}{a^2}\right]^{1/2} = \frac{a}{2\sqrt{3}}$$

and for a circular bearing

$$\left[\frac{I}{A}\right]^{1/2} = \left[\frac{\pi\phi^4}{64}\Big/\frac{\pi\phi^2}{4}\right]^{1/2} = \frac{\phi}{4}$$

It follows that for fixed f_H and f_V, the safety factor simply increases in terms of the plan dimension of the bearing, i.e., making a bearing wider makes it more stable. The final check of a bearing design is its roll-out stability. The maximum horizontal displacement, δ_{max}, it can sustain under the lightest load, W, is estimated from

$$\delta_{max} = \frac{Wa}{W + K_H h} \tag{5.20}$$

or

$$\delta_{max} = \frac{W\phi}{W + K_H h} \tag{5.21}$$

This can be evaluated assuming K_H is based on G at 50% shear strain, and if we take $p = W/A$, can be written as

$$\frac{\delta_{max}}{a} = \frac{1}{1 + \frac{G}{p}\,\frac{h}{t_r}} \tag{5.22}$$

Since G is usually about 150 psi, p about 1,000 psi, and $\frac{h}{t_r}$ about 1.2, we have

$$\delta_{max} \approx 0.89a \qquad \text{(circular bearing)} \tag{5.23}$$

or

$$\delta_{max} \approx 0.89\phi \qquad \text{(square bearing)} \tag{5.24}$$

The design displacement, D, should be less than δ_{max}.

The calculation for roll-out (Eq. 5.22) is for a single bearing. The parameter of interest is of course the roll-out displacement for the entire system. If the bearings are all identical, then the roll-out displacement can be obtained by using the average dead load in the above equation. If the bearings are of several different kinds, then the roll-out displacement can be estimated by calculating the value for each type of bearing for the average dead load on each type and determining the value for the whole system by using a weighted average of the values for each type.

The computation of the roll-out limit is applicable directly to isolators that are dowelled in place. In all isolation designs so far used in the U.S. and the one system in Italy, this has been the method of attachment. This has been done to insure that the elastomer is not loaded in tension. However, recent research has shown that the elastomer can sustain a substantial level of tension and it may be more common in the future to use a bolted connection. If the bearings are bolted into place, the roll-out formula is still useful since it will provide an approximate estimate of the displacement limit which will not produce large tension stresses in the elastomer.

The critical factor in the design of a bearing is the bond between the rubber and the steel shims. In the ideal case, the bonding compound should be sufficiently strong and the workmanship of the molding process reliable enough to insure that failure is always by tearing of the rubber and not delamination by bond failure. In the U.S., concern about the quality of the bonding has meant that the maximum design shear strain has been 100% and this has led to bearing designs that are rather tall, and consequently somewhat unstable and prone to roll-out. Experience with Japanese designs of isolators such as those manufactured by Bridgestone Corporation which tend to use design shear strains of 200% and even in some cases 300%, suggests that a better proportioned isolator will result. It will be more squat and less subject to instability and roll-out and in a sense, makes a more efficient use of the high intrinsic strength of the elastomer by making material failure the mechanism of failure rather than buckling or roll-out which are overall rather than local modes of failure. If the bond strength can be guaranteed, then the elastomer can be relied on to sustain shear strains of 450%-500%. The computation for roll-out suggests that roll-out of dowelled bearings or severe tension in bolted bearings will not develop if the displacement is less than 90% of the plan dimension. Accordingly, the diameter of the circular bearing should be about five times the rubber thickness. If the maximum shear strain is 200%, then the rubber thickness should be half the design displacement. Thus, we have two requirements.

$$\phi = 5\, t_r$$

$$t_r = \frac{D}{2}$$

Such a bearing would be very stable. Also, since

$$\frac{K_H}{A} = \frac{G}{t_r} = \frac{M}{A} w^2 = \frac{p\,(2\pi f_H)^2}{g}$$

and $D = CT$, where $C = \dfrac{10NZS}{B}$

$$T = \frac{1}{f_H}$$

we have

$$\frac{2G}{D} = \frac{p}{g} = \cdot \frac{(2\pi)^2}{T^2}$$

Rearranging this and using $D = cT$ gives

$$\frac{p}{G} = \frac{2gT^2}{(2\pi)^2 \cdot cT} = \frac{2T}{\frac{NZS}{B}}$$

Now p/G will be in the range 5 to 10 and NSZ/B around 0.4 to 0.6 and this equality can be satisfied for a range of practical design possibilities. As an example, consider a case where $N = 1.0$, $Z = 0.4$, $S = 1.2$ and B corresponding to 15% damping is 1.2. If for this case we take $T = 2.0$ sec., then we have $p/g = 10$.

This would imply that for a compound with $G = 125$ psi as a reasonable value for available high damping rubbers, then $p = 1250$ psi which is again, reasonable. The final design for the bearing would then be:

$$t_r = 4 \; in.$$

$$\phi = 20 \; in.$$

and the carried load 392 kips. With a moderate shape factor, say 10, the thickness t, of each layer would be given by:

$$t = \frac{\phi}{4S} = 1/2 \; in.$$

The bearing would have 8 layers and 7 shims, at say 1/8 in. each, with two 1 in. end plates, giving a total height of 6-7/8 in. The buckling load, P_c for this bearing given by

$$P_c = \sqrt{GAs \; P_E}$$

is 1607 kips for a safety factor of 4 and the safety factor against roll-out under the design vertical load is 2.125.

On the other hand, it may not be possible for a specified vertical load to exactly achieve this ideal bearing configuration, but a wide range of isolation designs are possible by varying p and T.

References

Derham, C.J. 1982. "The Design of Laminated Bearings," *Proceedings*, International Conference Natural Rubber for Earthquake Protection of Buildings and Vibration Isolation, pp. 247-257, Kuala Lumpur, Malaysia.

f_H = horizontal frequency or
T = horizontal period

f_V = vertical frequency

γ_{max} = maximum permissible shear strain

D = design displacement (from response spectrum or SEAOC formula)

The design quantities to be selected are a or ϕ, t_r, t, n, G, and h, where

ϕ = diameter of a round bearing

t_r = total rubber thickness in the bearing

t = thickness of individual layer

n = number of layers

G = shear modulus of elastomer

h = total height of the bearing

$$D = \frac{10NZST}{B} \tag{5.1}$$

$$t_r = \frac{D}{\gamma_{\max}} \tag{5.2}$$

$$K_H = \frac{GA}{t_r} \tag{5.3}$$

$$K_H = \frac{W}{g} \left(2\pi f_H \right)^2 = \frac{GA}{t_r} \tag{5.4}$$

$$\frac{G}{t_r} = \frac{p}{g} \left(2\pi f_H \right)^2 \tag{5.5}$$

Square Pad $S = \dfrac{a}{4t}$

Circular Pad $S = \dfrac{\phi}{4t}$

Annular Pad $S = \dfrac{\phi_0 - \phi_i}{4t}$

Square Pad $E_c = 6.75\, GS^2$

Circular Pad $E_c = 6\, GS^2$

Annular Pad $E_c = 4\, GS^2$

$$\gamma_{c_{\max}} = 6 S \varepsilon_c \tag{5.10}$$

$$\gamma_{c_{ave}} = \sqrt{6} S \varepsilon_c \tag{5.11}$$

$$E_c = 5.6 G S^2 \tag{5.12}$$

$$K_V = \frac{E_c A}{t_r} \tag{5.13}$$

$$\frac{K_V}{K_H} = \left[\frac{f_V}{f_H} \right]^2$$

$$\frac{K_V}{K_H} = \frac{E_c}{G}$$

$$6S^2 = \left[\frac{f_V}{f_H}\right]^2$$

$$S = \frac{1}{2.4}\,\frac{f_V}{f_H} \tag{5.14}$$

$$P_{cr} = \sqrt{P_S\;P_E} \tag{5.15}$$

$$P_S = GA_S \tag{5.16}$$

$$P_E = \frac{\pi^2 EI_{eff}}{h^2} \tag{5.17}$$

$$\frac{1}{3} E_C I \cdot \frac{h}{t_r}$$

$$S.F. = \frac{P_{cr}}{W} = \frac{\left[GA \frac{h}{t_r} \pi^2 \frac{1}{3} \frac{E_c I}{h^2} \frac{h}{t_r} \right]^{1/2}}{W}$$

$$= \frac{\pi}{\sqrt{3}} \left[\frac{GAg}{t_r W} \right]^{1/2} \left[\frac{E_c Ag}{t_r W} \right]^{1/2} \left[\frac{I}{A} \right]^{1/2} \tag{5.18}$$

$$\frac{GA}{t_r W} = (2\pi f_H)^2$$

$$\frac{E_c A g}{t_r W} = (2\pi f_V)^2$$

$$S.F. = \frac{\pi}{\sqrt{3}\, g} (2\pi f_H)(2\pi f_V) \left[\frac{I}{A}\right]^{1/2} \tag{5.19}$$

For a square bearing

$$\left[\frac{I}{A}\right]^{1/2} = \left[\frac{\frac{1}{12} a^4}{a^2}\right]^{1/2} = \frac{a}{2\sqrt{3}}$$

and for a circular bearing

$$\left[\frac{I}{A}\right]^{1/2} = \left[\frac{\pi\phi^4}{64} \Big/ \frac{\pi\phi^2}{4}\right]^{1/2} = \frac{\phi}{4}$$

$$\delta_{max} = \frac{Wa}{W + K_H h} \tag{5.20}$$

$$\delta_{max} = \frac{W\phi}{W + K_H h} \tag{5.21}$$

$$\frac{\delta_{max}}{a} = \frac{1}{1 + \frac{G}{p}\,\frac{h}{t_r}} \tag{5.22}$$

Circular bearing $\quad \delta_{max} \approx 0.89a \quad$ (5.23)

Square bearing $\quad \delta_{max} \approx 0.89\phi \quad$ (5.24)

$$\phi = 5\ t_r$$

$$t_r = \frac{D}{2}$$

$$\frac{K_H}{A} = \frac{G}{t_r} = \frac{M}{A} w^2 = \frac{p\ (2\pi f_H)^2}{g}$$

and $D = CT$, where $C = \dfrac{10NZS}{B}$

$$T = \frac{1}{f_H}$$

$$\frac{2G}{D} = \frac{p}{g} = \cdot\ \frac{(2\pi)^2}{T^2}$$

$$\frac{p}{G} = \frac{2gT^2}{(2\pi)^2 \cdot cT} = \frac{2T}{\frac{NZS}{B}}$$

$$t_r = 4 \ in.$$

$$\phi = 20 \ in.$$

$$t = \frac{\phi}{4S} = 1/2 \ in.$$

$$P_c = \sqrt{GAs \ P_E}$$

SEISMIC ISOLATION DECISION METHODOLOGY AND APPLICATION TO THE L.A. COUNTY FCCF

R.E. Bachman
Fluor Daniel, Inc.
Irvine, Calif.

13 Aug. 1992
Department of Energy Short Course on Seismic Isolation
Berkeley, Calif.

FLUOR DANIEL

O:\PS\111\216\00139.PM4-30

Design Decisions about Isolation Systems

- Trade Studies
- Cost/Benefit Analysis
- Procurement

Reference: T. L. Anderson, "Seismic Isolation for the Los Angeles County Fire Command and Control Facility," Workshop Reference Binder

FLUOR DANIEL

DESIGN DECISION

- How do you make it?
- Retrofit or new construction?
- Equal performance or code minimums?
- DOE or commercial facility?
- What's involved in an honest selection?

FLUOR DANIEL

O:\PS\111\216\00140.PM4-3

DESIGN DECISION

- Building height, configuration
- Performance requirements
  - Function
  - Containment
  - Investment
  - Code limitations
- Options
  - Framing
  - Hardening
  - Isolation
  - Acceptable Damage

FLUOR DANIEL

O:\PS\111\216\00140.PM4-4

DESIGN DECISION

- Design Conditions
  - Seismic exposure
  - Site geology
  - Contents fragility
  - Functional requirements
  - Risk avoidance
- Performance measurement
  - Building
  - Contents
  - Personnel
  - Cost
    * Construction
    * Life cycle
- Suppliers
- Decision making

FLUOR DANIEL

O:\PS\111\216\00140.PM4-5

DESIGN DECISIONS - TRADE STUDIES

1. Establish performance requirements
2. Set seismic design criteria
   - Applicable code
   - Hazard
     * Zone/fault proximity
     * Spectra
     * Time histories
     * Allowable stresses/deflections
     * Inelasticity
     * 1 or 2 level event

FLUOR DANIEL

TRADE STUDIES

3. Select isolation system types

   - Technical merit
   - Cost influence (secondary)
   - Musts
   - Wants
   - Decision Analysis methodology*

---

* Kepner Tregoe, Reference Binder

FLUOR DANIEL

O:\PS\111\216\00140.PM4-7

EXAMPLE

a. MUSTS:
1. Reduce base shear
2. Attenuate accelerations
3. Shake table tested

b. WANTS:
1. Simplicity
2. Displacement control
3. Wind drift
4. Temporal stability
5. Previous building use
6. High frequency attenuation
7. Stick slip
8. Dwell
9. Vendor support
10. Cost

FLUOR DANIEL

O:\PS\111\216\00140.PM4-8

<u>Example</u> (continued)

KT Scoring Matrix for
Seismic Isolation Solution

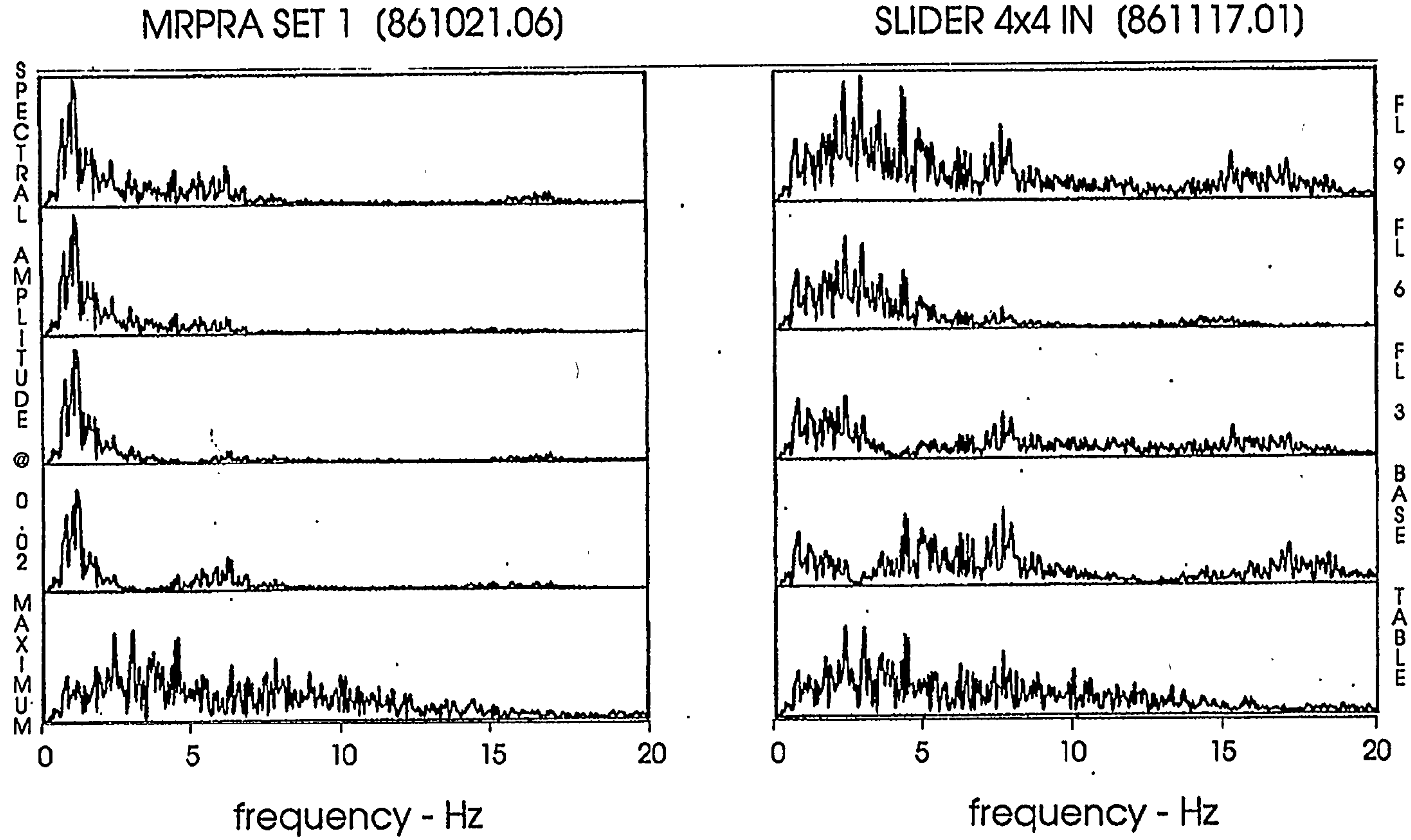
CANDIDATE SYSTEMS

WANTS	Wt	EHLR	DC	EWLC	FP	SLIDER
Simplicity	7					
Ultimate Restraint	5					
Wind Drift	8	7	7	9	10	10
Temporal Stability	9					
Previous Bldg. Use	6					
High Frequency Attenuation	10	10	10	9	3	3
Stick-Slip	6					
Dwell	7	10	10	10	8	2
Vendor Support	4					
Cost	0					
WEIGHTED TOTALS		XXX	XXX	XXX	XXX	XXX

EHLR = Elastomeric high loss rubber
DC = EHLR w/Displacement Control
EWLC = Elastomeric with lead core
FP = Friction pendulum
SLIDER = Low friction slide plates

FLUOR DANIEL

ACCELERATION FOURIER TRANSFORMS



EL CENTRO HORIZONTAL SPAN 150

FLUOR DANIEL

O:\PS\111\216\00140.PM4-10

Example (continued)

c. Use weighted table to short list (select) systems for adverse consequence analysis

Adverse consequences (for each alternative)

		*	**
1. 2. 3.	AAA BBB CCC		

* Probability of occurrence
** Seriousness if it should happen

- H,M,L Scale
- H/H is unsatisfactory
- Seek to minimize risk

For example, if wind deflection is higher than calculated, human discomfort under the design wind may be possible

P = L; S = M

FLUOR DANIEL

O:\PS\111\216\00140.PM4-11

TRADE STUDIES

4. Preliminary Analyses and Design (*)
   - Linear analysis
   - SDOF models
   - Solution bounds
   - Base shear/accelerations
   - Isolator sizes and capacity
   - Displacement and rattle space
   - Superstructure type/sizing
   - Foundation requirements
   - Structural deformations
   - Instructure accelerations

---

* T. L. Anderson, "Seismic Isolation Design and Construction Practice," 4NCEE, May 1990, Palm Springs, CA, Reference Binder

FLUOR DANIEL

O:\PS\111\216\00140.PM4-12

DESIGN DECISIONS - COST/BENEFIT ANALYSIS

Construction Cost for Alternatives

- Fixed-base design(s)
- Seismic hardening multipliers

Ex:

Seismic Hardening Multipliers*

- Computer Room HVAC unit = 1.2
- Chillers = 3.0
- Air Handling Units = 1.5
- Cooling Tower = 1.5
- Boiler = 3.0
- Electrical Equipment = 1.15

* Experience and manufacturer's recommendations

Normal equipment cost (used in isolated case) x hardening factor = equipment cost in fixed-base case

FLUOR DANIEL

COST/BENEFIT ANALYSIS

Construction cost for alternatives

- Isolated design(s)

- Isolators
  - Prototype
  - Testing
  - Installation
  - Typical costs

- Foundation modifications
  - Additional floor(?)
  - Rattle space
  - Retaining walls
  - Utilities

FLUOR DANIEL

O:\PS\111\216\00140.PM4-14

COST/BENEFIT ANALYSIS

- Feedback construction costs into K-T analysis (?)
- Comparative construction cost evaluation
  - Experience to date
  - Comparative performance (?)

FLUOR DANIEL

O:\PS\111\216\00140.PM4-15

COST/BENEFIT ANALYSIS

Life Cycle Costs & Benefits

1. Building Performance
   - Drift
2. Equipment
   - Acceleration
3. Contents
   - Acceleration
   - Administrative measures
4. Intangibles
   - Subjective
   - Hardening concerns
   - Human response/skill level
   - Visibility

FLUOR DANIEL

O:\PS\111\216\00140.PM4-16

COST/BENEFIT ANALYSIS

Life Cycle Costs (LCC)
Seismic Loss Estimate

LCC = Construction cost + present value of future seismic losses (PVL)*

$$PVL = AEL\ (P) \sum_{i=1}^{t} (1+RR)^{-i} \qquad (1)$$

where: AEL = Annual expected loss = loss/design life in years

P = Probability of damage in t years

t = Design life in years

RR = Expected rate of return

---

* T. L. Anderson, "Pipeline Fault-Crossing Design Strategy," 8WCEE, San Francisco, CA, July 1984, Reference Binder

FLUOR DANIEL

O:\PS\111\216\00140.PM4-17

COST/BENEFIT ANALYSIS

and $P = P_E \cdot P_D/E$ (2)

where P_E = probability of a specified level of shaking occurring during design life = probability of exceedance

P_D/E = conditional damage probability = probability of damage given a specified level of shaking

$P/E \approx$ damage ratio*

Element Damage Cost = Original cost x repair multiplier x damage ratio

---

* Reference J. M. Ferritto, "Economics of Seismic Design for New Buildings, " ASCE J. Str. Div., V.110, No. 12, Dec. 1984, pp 2925 - 2938, Reference Binder

FLUOR DANIEL

O:\PS\111\216\00139.PM4-18

Damage Ratios-Drift, Steel Structure

Element (1)	Cost, in dollars (2)	Repair multi-plier (3)	INTERSTORY DRIFT								
			0.001 (4)	0.005 (5)	0.010 (6)	0.020 (7)	0.030 (8)	0.040 (9)	0.070 (10)	0.100 (11)	0.140 (12)
1a. Rigid frames		2.0	0	0.01	0.02	0.05	0.10	0.25	0.35	0.50	1.00
b. Braced frames		2.0	0	0.03	0.14	0.22	0.40	0.85	1.00	1.0	1.0
c. Shear walls		2.0	0	0.05	0.30	0.30	0.60	0.85	1.00	1.0	1.0
2. Non-seimic structural frame		1.5	0	0.005	0.01	0.02	0.10	0.30	1.00	1.0	1.0
3. Masonry		2.0	0	0.10	0.20	0.50	1.00	1.0			
4. Windows and frames		1.5	0	0.30	0.80	1.00					
5. Partitions, architectural elements		1.25	0	0.10	0.30	1.00					
6. Floor		1.5	0	0.01	0.04	0.12	0.20	0.35	0.80	1.00	1.0
7. Foundation		1.5	0	0.01	0.04	0.10	0.25	0.30	0.50	1.0	1.0
8. Building equipment and plumbing		1.25	0	0.02	0.07	0.15	0.35	0.45	0.80	1.0	1.0
9. Contents		1.00	0	0.02	0.07	0.15	0.35	0.45	0.80	1.0	1.0

(After Ferritto, 1984)

FLUOR DANIEL

O:\PS\111\216\00139.PM4-19

Damage Ratios (P_D/E) as a Function of Acceleration for Steel Structures

Element	Dollars	Repair multiplier	Floor Acceleration in g's				
			0.08	0.18	0.50	1.2	1.4
1. Floor and roof system		1.5	0.01	0.02	0.10	0.50	1.0
2. Ceilings and lights		1.25	0.01	0.10	0.60	0.95	1.0
3. Building equipment and plumbing		1.25	0.01	0.10	0.45	0.60	1.0
4. Elevators		1.5	0.01	0.10	0.50	0.70	1.0
5. Foundations (slab on grade, sitework)		1.5	0.01	0.02	0.10	0.50	1.0
6. Contents		1.05	0.05	0.20	0.60	0.90	1.0

(After Ferritto, 1984)

FLUOR DANIEL

O:\PS\111\216\00140.PM4-20

COST/BENEFIT ANALYSIS

Example: LACFD FCCF Project
Seismic Damage Loss for Fixed-Base FCCF

Element	Level			$(K) Cost X	Repair Mult. X	Damage Ratio =	Damage Cost $(K)
	Gd	2nd	Rf				
Floor/Roof System	X			97	1.5	0.07	18.2
		X		97	1.5	0.3	43.7
			X	97	1.5	0.4	58.2
Ceilings and Lights		X		100	1.25	0.7	87.5
			X	100	1.25	0.9	112.5
Building Equipment and Plumbing	X			833	1.25	0.3	312.4
		X		311	1.25	0.5	194.4
			X	156	1.25	0.6	117.0
Foundations	X			90	1.5	0.07	9.5
Contents	X			125	1	0.4	50.0
		X		125	1	0.7	87.0
			X	—	1	0.8	

Total Loss = $1090K

FLUOR DANIEL

O:\PS\111\216\00140.PM4-21

COST/BENEFIT ANALYSIS

Example: (continued)

Seismic Damage Loss for Base-Isolated FCCF

Element	$(K) Cost X	Repair Mult. X	Damage Ratio =	Damage Cost $(K)
Floor/Roof System	290	1.5	0.01	4.4
Ceilings & Lights	200	1.25	0.01	2.5
Building Equipment & Plumbing	1,300	1.25	0.01	16.3
Foundations	250	1.5	0.01	3.8
Contents	250	1.0	0.05	12.5

Total Loss = $39.5K

FLUOR DANIEL

O:\PS\111\216\00140.PM4-22

COST/BENEFIT ANALYSIS

Example: (continued)

SEISMIC RISK - FCCF

- Fixed Base
  1. LAC Building Code - 75 Year Return Period
  2. Project Imposed - 1,400 Year (1.8%)
- Base-Isolated
  1. SEAOC - 1,000 Year (2.4%)

FLUOR DANIEL

O:\PS\111\216\00139.PM4-23

COST/BENEFIT ANALYSIS

Example: (continued)

PRESENT VALUE OF FUTURE SEISMIC LOSSES

Fixed Base:

$$PVL = \frac{\$1{,}090{,}000}{25} (0.018) \sum_{i=1}^{25} (1+0.08)^{-i}$$

$$= \$8{,}720 \longleftarrow$$

Base Isolated:

$$PVL = \frac{\$39{,}500}{25} (0.012) \sum_{i=1}^{25} (1+0.08)^{-i}$$

$$= \$200 \longleftarrow$$

- Sensitivity
- 40 to 1

FLUOR DANIEL

O:\PS\111\216\00140.PM4-24

DESIGN DECISION - PROCUREMENT

- Sources
- Performance specification versus design specification
- Commodity approach
- "Or equal" clause (with suitable backup)
- State-of-the-practice
  - Off the shelf design
  - Custom design

FLUOR DANIEL

O:\PS\111\216\00140.PM4-25

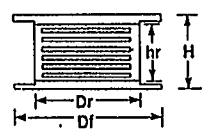
Performance Table (Standard Type)

HHD: Bearing with High Elasticity & High Damping

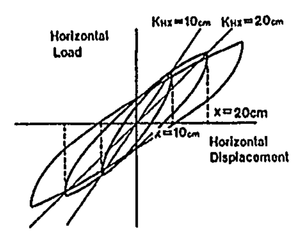
(Other types are available on request)

Product No.	Application Load Range Wo	Performance					Dimensions				Weight	
		Horizontal Direction				Vertical Direction						
		K_{HX}=10cm	K_{HX}=20cm	Heq.	Δx	Kv	Dr	hr	Df	H	W	W
	(M. ton)	(ton/cm)	(ton/cm)		(cm)	(ton/cm)	(cm)	(cm)	(cm)	(cm)	(kg)	(lb)
HHD-060	150 ~ 200	1.70	1.20	0.15	35	1,400	60	29	88	34	500	1,100
HHD-065	180 ~ 240	2.00	1.40	0.15	35	1,700	65	28	93	33	500	1,100
HHD-070	210 ~ 290	2.30	1.70	0.15	35	2,000	70	27	98	32	600	1,300
HHD-075	240 ~ 320	2.70	1.90	0.15	40	2,300	75	27	107	32	700	1,500
HHD-080	270 ~ 350	3.00	2.10	0.15	40	2,500	80	27	112	33	800	1,800
HHD-085	300 ~ 400	3.40	2.40	0.15	40	2,800	85	26	117	33	900	2,000
HHD-090	340 ~ 450	3.80	2.70	0.15	40	3,200	90	26	122	32	1,000	2,200
HHD-100	410 ~ 560	4.60	3.30	0.15	40	4,000	100	27	137	35	1,400	3,000
HHD-110	500 ~ 680	5.70	4.00	0.15	40	4,900	110	26	152	34	1,600	3,500
HHD-120	580 ~ 790	6.50	4.60	0.15	40	5,700	118	26	159	34	1,900	4,200

Dr : Rubber Diameter (Interior plate diameter)
Df : Flange Outer Diameter
hr : Total Thickness of Rubber
(1 layer of rubber x number of layers)
H : Total Height of Rubber Bearing



Kh(X=10) : Horizontal direction spring constant.
(Equivalent horizontal spring constant at horizontal displacement X=10cm.)
Kh(X=20) : Horizontal direction spring constant.
(Equivalent horizontal spring constant at horizontal displacement X=20cm.)
Heq Equivalent viscous damping ratio
Δx Designed Horizontal Displacement (max. load is to be 2 times as much as designed load)
Kv Vertical Direction Spring Constant
(min. assured value)



BRIDGESTONE

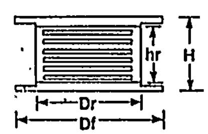
Performance Table (Standard Type)

LHD: Bearing with Low Elasticity & High Damping

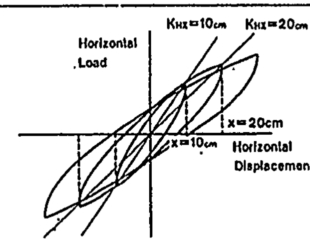
(Other types are available on request)

Product No.	Application Load Range Wo	Performance					Dimensions				Weight	
		Horizontal Direction				Vertical Direction						
		K_{HX}=10cm	K_{HX}=20cm	Heq	Δx	Kv	Dr	hr	Df	H	W	W
	(M. ton)	(ton/cm)	(ton/cm)		(cm)	(ton/cm)	(cm)	(cm)	(cm)	(cm)	(kg)	(lb)
LHD-050	70 ~ 90	0.83	0.55	0.15	35	1,200	52	19	71	23	400	900
LHD-060	100 ~ 120	1.10	0.72	0.15	35	1,400	60	17	85	22	500	1,100
LHD-070	130 ~ 170	1.50	1.00	0.15	35	2,200	70	17	99	22	600	1,300
LHD-080	170 ~ 210	1.90	1.30	0.15	40	2,800	79	16	116	22	800	1,800
LHD-090	220 ~ 280	2.50	1.70	0.15	40	3,600	92	19	126	24	900	2,000
LHD-100	270 ~ 330	3.00	2.00	0.15	40	4,400	101	19	141	24	1,200	2,600

Dr : Rubber Diameter (Interior plate diameter)
Df : Flange Outer Diameter
hr : Total Thickness of Rubber
(1 layer of rubber x number of layers)
H : Total Height of Rubber Bearing



Kh(X=10) : Horizontal direction spring constant.
(Equivalent horizontal spring constant at horizontal displacement X=10cm.)
Kh(X=20) : Horizontal direction spring constant.
(Equivalent horizontal spring constant at horizontal displacement X=20cm.)
Heq Equivalent viscous damping ratio
Δx Designed Horizontal Displacement (max. load is to be 2 times as much as designed load)
Kv Vertical Direction Spring Constant
(min. assured value)



BRIDGESTONE

Seismic Base Isolation System Suppliers

1. Andre
   Division of BTR Silvertown Ltd.
   PO Box 4
   Horninglow Road
   Burton-on-Trent
   Staffs DE130QN
   England
   Attn: Mr. W. Brake, Sales Manager
   (283) 510 510
   Telex 34419

2. Base Isolation Consultants, Inc.
   246 First Street, Ste. 304
   San Francisco, CA 94102
   Attn: Mr. D. Way, Vice President
   (415) 495-0515
   FAX (415) 777-2858

3. Bridgestone Engineered
   Products Company
   PO Box 6147
   Huntington Beach, CA 92615-6147
   Attn: Mr. R. Busch
   (714) 962-1666
   FAX (714) 968-3441

4. Dynamic Isolation Systems, Inc.
   2855 Telegraph Avenue, Suite 410
   Berkeley, CA 94705
   Attn: Mr. R. Mayes, President
   (415) 843-7233
   FAX (415) 843-0366

5. Fyfe Associates, Inc.
   1341 Ocean Avenue
   Del Mar, CA 92014
   Attn: Mr. E. Fyfe, President
   (619) 792-1501
   FAX (619) 259-3872

6. Kajima Corporation
   2-7, Motoakasaka 1-chome
   Minato-ku, Tokyo 107
   Japan
   (03) 404-3311
   FAX (03) 345-6049

FLUOR DANIEL

O:\PS\111\216\00140.PM4-27

Seismic Base Isolation System Suppliers (Cont.)

7. Shimizu Construction Company
   Ohsaki Research Institute
   Fukoku Seimei Bldg., 27th floor
   2-2-2, Uchisaiwai-cho
   Chiyoda-ku, Tokyo 100
   Japan
   Attn: Mr. Hiroshi Yamahara,
   General Manager
   (03) 508-8101

8. Skellerup Polymer Products Ltd.
   PO Box 19-555
   Woolston
   Christchurch, New Zealand
   89 91 89

9. Societe E. R. A.
   Residence "La Bruyere," Batiment B2
   2, Rue de 1' Horticulture
   13009 Marseille
   France
   Attn: Mr. P. Delfosse,
   Technical Director
   33 91 75 17 74
   FAX 33 91 26 55 56
   Telex 420322F

10. Telesis International
    4605 Lankershim, Suite 710
    No. Hollywood, CA 91602
    Attn: Mr. J. Brennen, President
    (818) 769-8894
    FAX (818) 769-4822

FLUOR DANIEL

O:\PS\111\216\00140.PM4-28

PROCUREMENT

- Performance specification
  - Period Range
  - Damping Range
  - Displacement Specified
  - Base Shear Limit
  - Displacement Margin
  - Ultimate Restraint System (?)
  - Testing
  - Confirmation Analyses

- Example - FCCF*

---

* Seismic Base Isolation System Requirements Specification, Reference Binder

FLUOR DANIEL

O:\PS\111\216\00140.PM4-29

APPLYING UBC CODE DESIGN REQUIREMENTS TO THE L.A. COUNTY FCCF

R.E. Bachman
Fluor Daniel, Inc.
Irvine, Calif.

13 Aug. 1992
Department of Energy Short Course on Seismic Isolation
Berkeley, Calif.

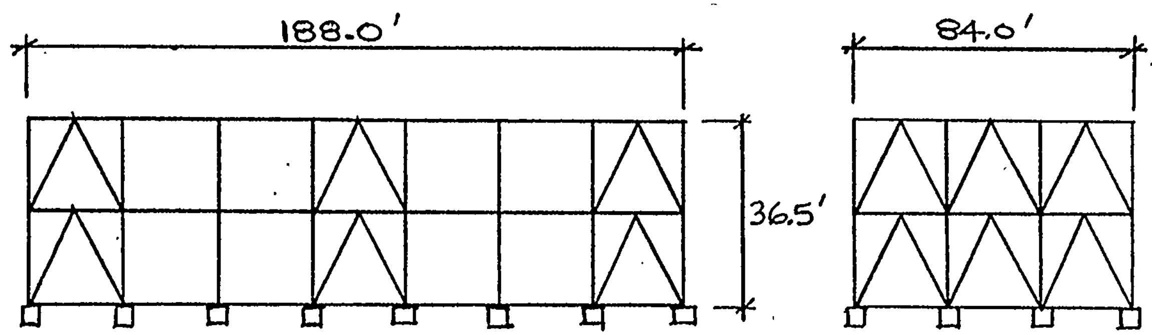
FLUOR DANIEL

O:\PS\111\216\00139.PM4-31

APPLYING UBC CODE REQUIREMENTS TO THE L.A. COUNTY FIRE COMMAND AND CONTROL FACILITY

PROBLEM: DEVELOP BASIC DESIGN PARAMETERS AND FORCES FOR ISOLATION SYSTEM AND BUILDING.

GIVEN: RECTANGULAR TWO STORY BUILDING.



PLAN DIMENSIONS, 188 FT BY 84 FT

HEIGHT = 36.5 FT

STRUCTURAL SYSTEM = STEEL BRACED FRAME WITH INVERTED CHEVRON BRACING

NUMBER OF COLUMNS = 32
ASSUME ISOLATORS UNDER EVERY COLUMN

BUILDING SITE AND LOCATION:

LOCATION: SOUTHERN CALIFORNIA

NEAREST ACTIVE FAULT SITE = 15.0 KM

SOIL CONDITION, SANDSTONE = SOIL TYPE S1

BASIC WIND SPEED = 70.0 MPH, EXPOSURE C
ANSI CLASSIFICATION III

BASIC STRUCTURAL PROPERTIES:

SEISMIC MASS		PSF
ROOF =	790. KIPS	50.
2ND FLOOR =	1775. KIPS	112.
1ST FLOOR =	1665. KIPS	105.
TOTAL =	4230. KIPS	

GROSS ECCENTRICITY = 3.4% BY CALCULATION

ESTABLISH BASIC DYNAMIC PARAMETERS AND FORCES:

STEP 1: DETERMINE LIMITING PARAMETERS OF THE ISOLATION SYSTEM

- BASE ISOLATION PERIOD, T_I
- DAMPING, β
- LIMITING DISPLACEMENT OF ISOLATOR, D

USUALLY PERIOD T_I RANGES BETWEEN 2 AND 2.5 SEC AND DAMPING β RANGES BETWEEN 10% AND 20%. DISPLACEMENT IS GENERALLY WHATEVER IS CALCULATED

ASSUME — $T_I = 2.2$ SEC. THIS WILL GENERALLY BE DEPENDENT ON TYPE OF ISOLATOR.

$\beta = 15\%$

STEP 2: DETERMINE THE DESIGN DISPLACEMENT, D

USING EQUATION E-1, PAGE 189-C OF APPENDIX IL

$$D = 10 Z N S_I T_I / B \quad \text{(INCHES)}$$

$Z = 0.4$ (FROM TABLE 1-A, PAGE 30)

$N = 1.0$ (FROM TABLE B, PAGE 202-C)

$S_I = 1.0$ (FROM TABLE A, PAGE 202-C)

$T_I = 2.2$ SEC. (ASSUMED)

$B = \left[(1.2 + 1.5)/2\right] = 1.35$ (FROM TABLE C, PAGE 202-C)

$$D = \left[\frac{10\,(0.4)(1.0)(1.0)(2.2)}{1.35}\right] = 6.5 \text{ INCHES}$$

STEP 3: DETERMINE ISOLATOR EFFECTIVE STIFFNESS K, AT DISPLACEMENT D

FOR EXAMPLE, ASSUME $K_{MIN} = K_{MAX} = K$
(FOR MANY ISOLATORS, THE RANGE OF EFFECTIVE STIFFNESS VALUES IS SMALL)

USING EQUATION E-2, PAGE 189-C OF APPENDIX IL

$$T_I = 2\pi \sqrt{\frac{W}{Kg}}$$

$$K = \left(\frac{2\pi}{T_I}\right)^2 \frac{W}{g}$$

$$K = \left(\frac{2\pi}{2.2}\right)^2 \times \frac{4230.}{386.4} = 89.3 \frac{\text{KIPS}}{\text{IN}} \text{ FOR 32 ISOLATORS}$$

$$K_{INDIVIDUAL} = \left(\frac{89.4}{32}\right) = 2.79 \frac{\text{KIPS}}{\text{IN}}$$

STEP 4: DETERMINE TOTAL DISPLACEMENT D_T OF ISOLATOR.

USING EQUATION E-3, PAGE 189-C OF APPENDIX IL

$$D_T = D\left[1 + \frac{12ye}{(b^2+d^2)}\right]$$

$y = 94.$ FT

$e = 3.4\% + 5.0\% = \frac{(8.4)(188)}{100} = 15.8^{FT}$

$b = 84.$ FT

$d = 188.$ FT

$$D_T = (6.5)\left[1 + 12(94.)(15.8) \Big/ \left[(84)^2 + (188)^2\right]\right] = 9.2 \text{ INCHES}$$

STEP 5: CALCULATE THE TOTAL MAXIMUM DISPLACEMENT D_{TM}

USING EQUATION E-4, PAGE 189-C OF APPENDIX IL

$$D_{TM} = 1.5\, D_T = (1.5)(9.2) = 13.8 \text{ INCHES}$$

STEP 6: CALCULATE THE MINIMUM LATERAL FORCE, V_b, FOR ALL STRUCTURAL ELEMENTS BELOW ISOLATOR INTERFACE.

USING EQUATION E-5, PAGE 189-C OF APPENDIX IL

$$V_b = \frac{K_{MAX}\, D}{1.5}$$

IN THIS CASE FOR ONE ISOLATOR USE $K_{MAX} = K_{INDIVIDUAL}$

$$V_b = \frac{(2.79)(6.5)}{1.5} = 12.1^{\text{KIPS}}$$

FOOTING AND ISOLATOR ANCHORAGE SHOULD BE DESIGNED FOR THESE FORCES.

STEP 7: COMPUTE THE MINIMUM LATERAL FORCE, V_S, FOR ALL STRUCTURAL ELEMENTS ABOVE ISOLATOR INTERFACE.

USING EQUATION E-6, PAGE 190-C OF APPENDIX IL

$$V_S = \frac{K_{MAX} D}{R_{WI}}$$

$R_{WI} = 1.8$ IF BRACE CARRIES GRAVITY LOADS.

$R_{WI} = 2.2$ IF BRACE DOES NOT CARRY GRAVITY LOADS

ALTHOUGH CODE REQUIRES BEAM TO BE DESIGNED AS IF BRACE DOES NOT EXIST, DESIGN AS IF BRACE CARRIES GRAVITY LOAD. (SINCE THE BRACING WILL CARRY SOME GRAVITY LOAD.)

K_{MAX} = K OF 32 ISOLATORS

$$V_S = \left[(89.3)(6.5)/1.8\right] = 322.5 \text{ KIPS}$$

COMPUTE FOR INTEREST, EQUIVALENT ACCELERATION COEFFICIENT, a:

$$a = (322.5/4230) = 0.076g$$

STEP 8: COMPUTE LIMITS OF V_S

V_S SHALL BE GREATER THAN OR EQUAL TO:

(1) THE LATERAL SEISMIC FORCE FOR A FIXED BASE STRUCTURE OF SAME WEIGHT AND PERIOD T_I

$$V = \frac{ZIC}{R_W} W$$

$$I = 1.0$$

$$Z = 0.4$$

$$S = 1.0$$

$$C = \frac{1.25\,S}{T^{2/3}} = \left[\frac{(1.25)(1.0)}{(2.2)^{2/3}}\right] = 0.74$$

$$R_W = 6.0$$

$$V = \left[\frac{(0.4)(1.0)(0.74)}{(6.0)}\right](4230) = 208.7^{\text{KIPS}}$$

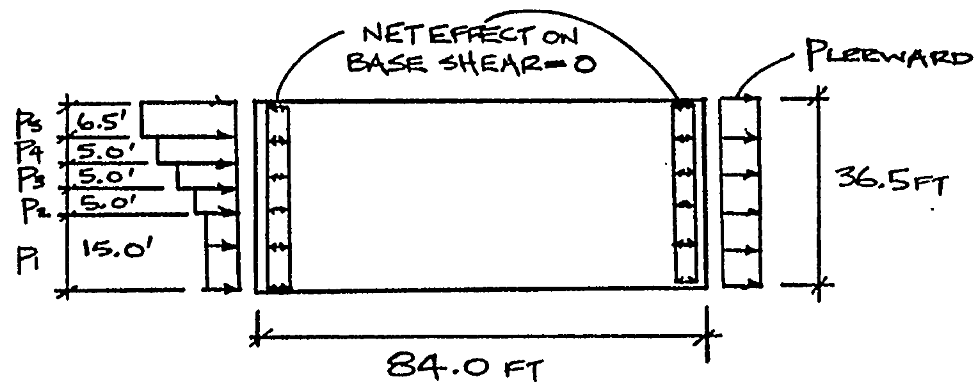
COMPUTE FOR INTEREST, EQUIVALENT ACCELERATION COEFFICIENT, a:

$$a = (208.7 / 4230.) = 0.049\,g$$

(2) THE BASE SHEAR CORRESPONDING TO THE DESIGN WIND LOAD

$$V_{WIND} = pA$$

$$p = \left[q G_h C_p - \cancel{q_h (G C_{pi})} \right]$$ DOESN'T CONTRIBUTE TO OVERALL BASE SHEAR.



$V = 70.0$ MPH

$I = 1.07$

$G_h = 1.23$

$C_p = 0.8$ WINDWARD WALL

$C_p = -0.5$ LEEWARD WALL

HEIGHT ZONE	K_z	q	p
0-15	0.80	11.49	11.3
15-20	0.87	12.49	12.3
20-25	0.93	13.36	13.1
25-30	0.98	14.07	13.8
30-36.5	1.06	15.22	15.0
LEEWARD WALL	1.06	15.22	-9.4

$$V_{WIND} = \left[(188)(15.)(P_1) + (188)(5.)(P_2 + P_3 + P_4) + (188)(6.5)(P_5) + (188)(36.5)\,P_{LEEWARD}\right]$$

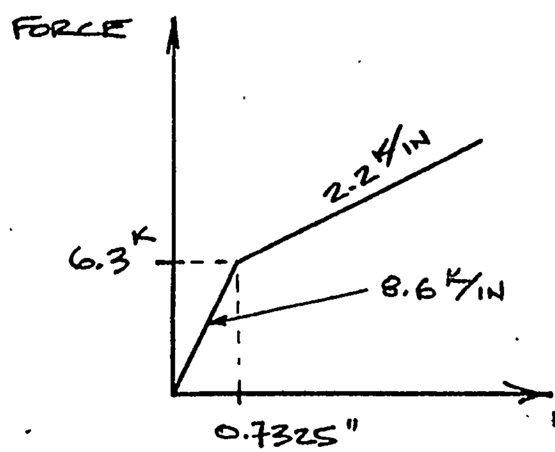
$$= \left[(188)(15)(11.3) + (188)(5.)(12.3 + 13.1 + 13.8) + (188)(6.5)(15.0) + (188)(36.5)(9.4)\right]$$

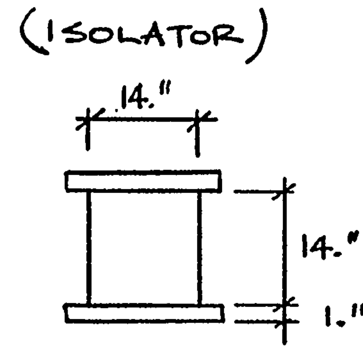
$$= (31866 + 36848 + 18330 + 64502)\ \text{lbs}$$

$$V_{WIND} = 151.5^{KIPS}$$

STEP 9: VERIFY THE DESIGN OF ISOLATOR TO ACHIEVE DESIGN PROPERTIES

ASSUME THE FOLLOWING ISOLATOR PROPERTIES PROVIDED BY SUPPLIER.





USING KIRCHEN SIMPLIFIED EQUATIONS,

(1) CHECK DAMPING AT DISPLACEMENT OF 6.5 INCHES

$$\beta = \frac{0.637\,(a_y d - d_y a)}{a d}$$

OR

$$\beta = \frac{0.637\,(F_y d - d_y F)}{F d}$$

@ $d = 6.5$ INCHES

$$F = 6.3 + 2.2\,(6.5 - 0.7325) = 19.0^{K}$$

$$\beta = \frac{0.637\left[(6.3)(6.5) - (0.7325)(19.0)\right]}{(19.0)(6.5)}$$

$$\beta = 0.139 \quad \text{OR} \quad 13.9\%$$

$\therefore$ DAMPING ASSUMPTION OF 15.% O.K.

$$\beta = \frac{0.637\left[F_y d - d_y F\right]}{F d}$$

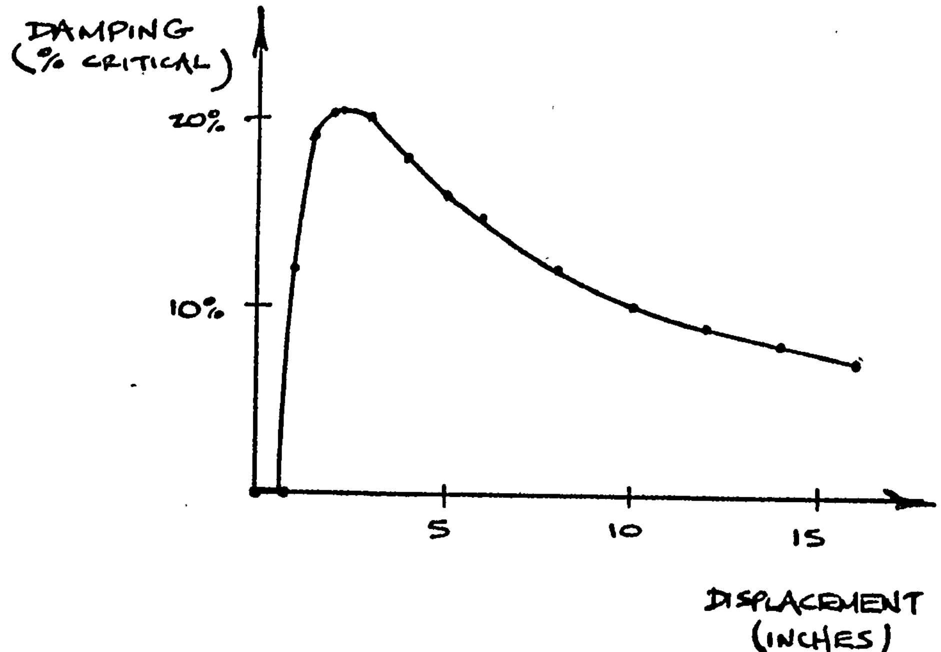
$F_y = 6.3^K$

$d_y = 0.7325$ in

$F = 8.6d \quad (\text{FOR } 0 < d < 0.7325)$

$F = \left[6.3 + (d - 0.7325)\,2.2\right] \text{ FOR } (0.7325 < d)$

d	F	β
0.1	0.9	0.0
0.7325	6.3	0.0
1.0	6.9	0.12
1.5	8.0	0.19
2.0	9.1	0.208
2.2	9.5	0.209
3.0	11.3	0.20
4.0	13.5	0.18
5.0	15.7	0.16
6.0	17.9	0.15
8.0	22.3	0.12
10.0	26.7	0.10
12.0	31.1	0.09
14.0	35.5	0.08
16.0	39.9	0.07



(2) CHECK ISOLATION PERIOD AT 6.5 INCHES

$$K = \frac{F}{d} = \left(\frac{19.0}{6.5}\right) = 2.92\ ^{K}/_{IN}$$

$$K_{TOTAL} = 32\,(2.92) = 93.5\ ^{K}/_{IN}$$

$$T = 2\pi\sqrt{\frac{W}{Kg}} = 2\pi\sqrt{\frac{4230.}{(93.5)(386.4)}} = 2.15 \text{ SECONDS}$$

∴ PERIOD ASSUMPTION OF 2.2 SECONDS O.K.

(3) CHECK WIND YIELD FORCE

$$\text{WIND YIELD} = (6.3)(32) = 201.6^{\text{KIPS}}$$

$$\text{WIND FORCE} = 151.5^{\text{KIPS}}$$

∴ SINCE WIND FORCE IS LESS THAN WIND YIELD, ISOLATOR O.K.

(4) TESTING LOADS AND DISPLACEMENT REQUIREMENTS TO CHECK ISOLATOR STABILITY BY TEST.

PER SECTION L :-

<u>CYCLIC LOAD TESTS</u>

TEST DISPLACEMENT = D_T = 9.2 INCHES

1) VERTICAL TEST LOAD = $1.2DL + 0.5LL + |E|$

2) VERTICAL TEST LOAD = $0.8\,DL - |E|$

<u>MAXIMUM LOAD TESTS FOR STABILITY</u>

TEST DISPLACEMENT = D_{TM} = 13.8 INCHES

1) VERTICAL LOAD TEST = $1.2\,DL + 0.5LL + |E|$

2) VERTICAL LOAD TEST = $0.8DL - |E|$

STEP 10:

DISTRIBUTE FORCES TO LATERAL FORCE RESISTING SYSTEM.

USING EQUATION E-7, PAGE 190-C FROM APPENDIX II.

$$F_x = \frac{V_s w_x}{\sum_{i=1}^{n} w_i}$$

$$\sum_{i=1}^{n} w_i = (790. + 1775 + 1665) = 4230.^{\text{KIPS}}$$

$$\text{ROOF SHEAR} = \left[\frac{(322.5)(790)}{4230}\right] = 60.2^{K}$$

$$\text{2ND FLOOR SHEAR} = \left[\frac{(322.5)(1775)}{4230}\right] = 135.3^{K}$$

$$\text{1ST FLOOR SHEAR} = \left[\frac{(322.5)(1665)}{4230}\right] = 127.0^{K}$$

FROM ANALYSIS, FLOOR STIFFNESSES:—

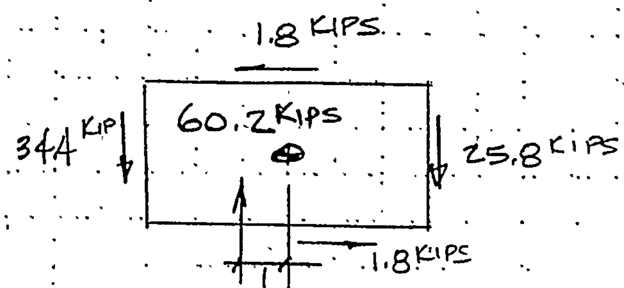
TRANSVERSE DIRECTION,

$K_{\text{2ND FLOOR}} = 38,712.$ KIPS/FT
$K_{\text{1ST FLOOR}} = 49,320.$ KIPS/FT

LONGITUDINAL DIRECTION,

$K_{\text{2ND FLOOR}} = 35,472.$ KIPS/FT
$K_{\text{1ST FLOOR}} = 48,000.$ KIPS/FT

DISTRIBUTE ROOF SHEAR, (ASSUME 8.4% ECCENTRICITY)



$$M_T = Ve = (60.2)(15.8\,\text{FT}) = 951.2^{\,\text{KFT}}$$

$$e = (0.084)(188.) = 15.8\,\text{FT}$$

$$V_{DIRECT} = (60.2)/2 = 30.1^{\,\text{KIPS}}$$

$$\Sigma(Kd^2) = \left[(38712.)(94.)^2(2) + (35472.)(42.)^2(2)\right] = 809.3\,\text{E6}\ \text{KIP.FT}$$

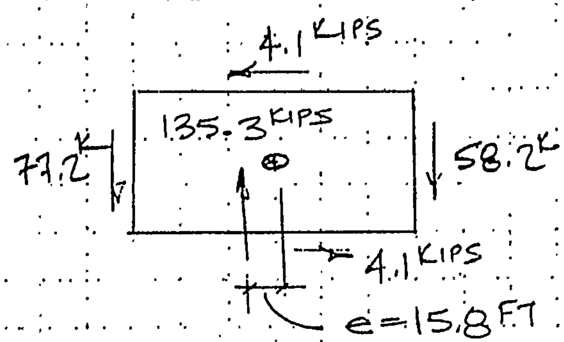
$$V_{\substack{TORSION\\LONGITUDINAL}} = M_T\,(Kd)/\Sigma(Kd^2) = \left[(951.2)(35{,}472.)(42.)/(809.3\,\text{E6})\right] = 1.8^{\,\text{KIPS}}$$

$$V_{\substack{TORSION\\TRANSVERSE}} = \left[(951.2)(38{,}712.)(94.0)/(809.3\,\text{E6})\right] = 4.3^{\,\text{KIPS}}$$

$$V_{\substack{TOTAL\\LONGITUDINAL}} = (0.0 \pm 1.8) = \pm 1.8^{\,\text{KIPS}}$$

$$V_{\substack{TOTAL\\TRANSVERSE}} = (30.1 \pm 4.3) = 34.4^{\,\text{KIPS}},\ 25.8^{\,\text{KIPS}}$$

DISTRIBUTE 2ND FLOOR SHEAR (ASSUME 8.4% ECCENTRICITY)



$$M_T = Ve$$
$$= (135.3)(15.8)$$
$$= 2137.7 \text{ KIP}\cdot\text{FT}$$

$$V_{DIRECT} = (135.3)/2$$
$$\doteq 67.7 \text{ KIPS}$$

$$\Sigma(Kd^2) = \left[(49{,}320.)(94.)^2(2) + (48{,}000)(42.)^2(2)\right]$$
$$= 1040.9\text{E}6 \text{ KIP}\cdot\text{FT}$$

$$V_{\substack{TORSION\\LONGITUDINAL}} = \left[(2137.7)(48{,}000)(42.)/1040.9\text{E}6\right]$$
$$= 4.1 \text{ KIPS}$$

$$V_{\substack{TORSION\\TRANSVERSE}} = \left[(2137.7)(49320.)(94.)/1040.9\text{E}6\right]$$
$$= 9.5 \text{ KIPS}$$

$$V_{\substack{TOTAL\\LONGITUDINAL}} = \pm 4.1 \text{ KIPS}$$

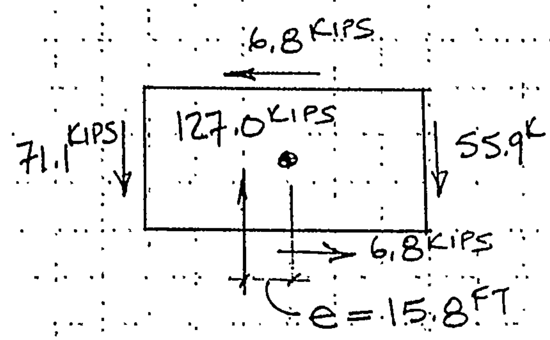
$$V_{\substack{TOTAL\\TRANSVERSE}} = (67.7 \pm 9.5) = 77.2 \text{ KIPS},\ 58.2 \text{ KIPS}$$

DISTRIBUTE 1ST FLOOR SHEAR (ASSUME 8.4% ECCENTRICITY)

– CONSERVATIVELY, CONSIDER ONLY PERIMETER ISOLATORS

$$K_{LONGITUDINAL} = 8\left(8.6^{KIPS}/_{IN}\right)(12.) = 825.6^{KIPS}/_{FT}$$

$$K_{TRANSVERSE} = 4(8.6)(12) = 412.8^{KIPS}/_{FT}$$



$$M_T = Ve = (127.0)(15.8) = 2006.6 \text{ KIP}\cdot\text{FT}$$

$$V_{DIRECT} = (127.0)/2 = 63.5 \text{ KIPS}$$

$$\Sigma(Kd^2) = \left[(825.6)(42.)^2(2) + (412.8)(94.)^2(2)\right] = 10.2^{E6} \text{ KIP}\cdot\text{FT}$$

$$V_{TORSION\ LONGITUDINAL} = \left[(2006.6)(825.6)(42.)/(10.2^{E6})\right] = 6.8 \text{ KIPS}$$

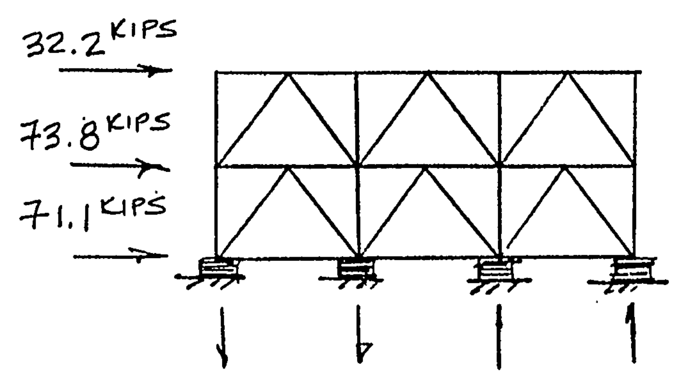
$$V_{TORSION\ TRANSVERSE} = \left[(2006.6)(412.8)(94.)/(10.2^{E6})\right] = 7.6 \text{ KIPS}$$

$$V_{\substack{TOTAL \\ LONGITUDINAL}} = \pm 6.8^{KIPS}$$

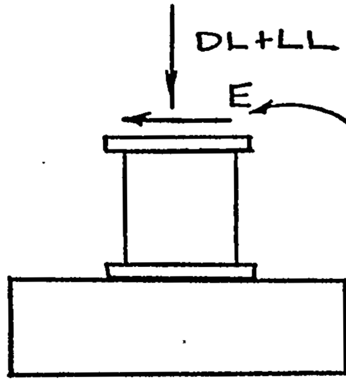
$$V_{\substack{TOTAL \\ TRANSVERSE}} = (63.5 \pm 7.6) = 71.1^{KIPS}, 55.9^{KIPS}$$

SUMMARY OF END FRAME SHEARS :—

(SIMILAR FOR LONGITUDINAL DIRECTION)



STEP 11: DESIGN ISOLATOR COMPONENT AND FOOTING



DL+LL $= 188.9^K$ (MAX) IN LONGITUDINAL DIRECTION

$= 255.9^K$ (MAX) IN TRANSVERSE DIRECTION

$E = (\text{EARTHQUAKE LOAD}) \times \left(\frac{1.8}{1.5}\right)$

FROM ANALYSIS

$$E_v = (15.0^K)\left(\frac{1.8}{1.5}\right) = 18.0^{KIPS}$$

ASSUME SPREAD FOOTING

ALLOWABLE SOIL BEARING = 5.0^{KSF}

FOR EARTHQUAKE; ALLOW 1/3 INCREASE = 6.65^{KSF}

HORIZONTAL DESIGN SHEAR

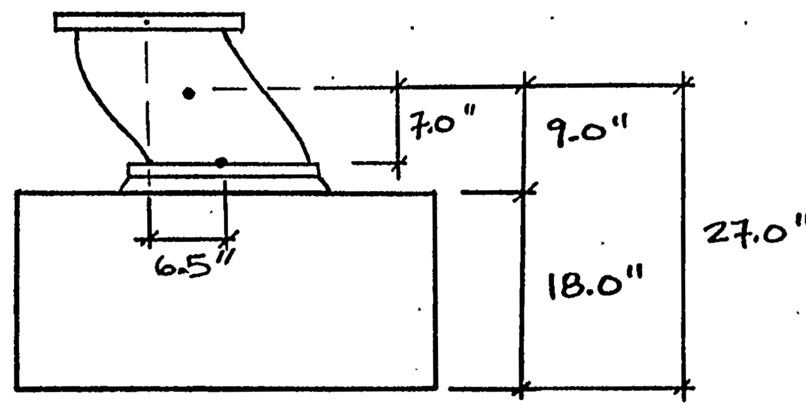
$$V_b = \frac{KD}{1.5} = \left[\frac{(2.79)(6.5)}{1.5}\right] = 12.1^{KIPS}$$

$$P = 255.9^K + 18.0^K = 273.9^{KIPS}$$

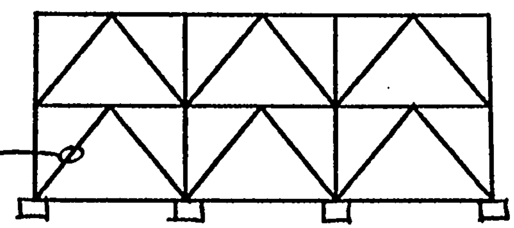
$$V = 12.1^{KIPS}$$

$$M = \left[(12.1)(27.) + (273.9)\left(\frac{6.5}{2}\right)\right]$$

$$= (326.7 + 890.2) = 1217.^{KIP \cdot IN}$$



STEP 12: BRACING DESIGN



ASSUME FROM ANALYSIS IN STEP 11

$D+L = 29.7^{KIPS}$ (LONGITUDINAL)

$D+L = 10.9^{KIPS}$ (TRANSVERSE)

$E = 24.0^{KIPS}$ (LONGITUDINAL & TRANSVERSE)

DESIGN LONGITUDINAL BRACE FOR,

$$F = \left[29.7 + 1.5\,(24.)\right] = 65.7^{KIPS}$$

DESIGN CONNECTIONS FOR,

$$F = 29.7 + 3/8\ R_w\ (24)$$
$$= \left[29.7 + \left(\tfrac{3}{8}\right)(1.8)(24.)\right]$$
$$= (29.7 + 16.2)$$
$$= 45.9^{KIPS} \longrightarrow \text{USE } \underline{65.7^{KIPS}}$$

IN ADDITION,

$$(L/r) \le \frac{720}{\sqrt{F_y}}$$

AND $F_{as} = \lambda F_a$; λ = STRESS REDUCTION FACTOR (FROM SECTION 4F, pg. 51)

STEP 13: CHECK STORY DRIFT IN FRAME FOR STEP 11 ANALYSIS.

$$\delta = \frac{0.010}{R_{WI}} = \left(\frac{0.010}{1.8}\right) = 0.00555$$

ASSUME FROM ANALYSIS (FROM STEPS 11-12), MAXIMUM INTERSTORY DRIFT = 0.2 INCH

STORY HEIGHT = 15.5 FT

ALLOWABLE DRIFT = $(15.5)(12)(0.00555) = 1.03$ INCH

$\therefore$ SINCE ACTUAL DRIFT = 0.2 INCH, DESIGN O.K.

STEP 14: CHECK ISOLATION DRIFT FOR WIND LOADS

$$\delta_{WIND} = \left(\frac{V_{WIND}}{K}\right) = \left[\frac{151.5^{K}}{(8.6)(32)}\right] = 0.55 \text{ INCH}$$

$\therefore$ SINCE WIND LOAD DRIFT IS LESS THAN ALLOWABLE DRIFT, DESIGN O.K.

STEP 15: CHECK BUILDING FIXED BASE PERIOD VERSUS ISOLATION PERIOD.

BUILDING FIXED BASE PERIOD BY ANALYSIS,

$T_{(LONGITUDINAL)}$ = 0.36 SECONDS

$T_{(TRANSVERSE)}$ = 0.40 SECONDS

RATIO OF ISOLATED PERIOD TO FIXED BASE PERIOD,

$$\frac{T_I}{T_{(LONG.)}} = \left(\frac{2.2}{0.36}\right) = 6.1$$

$$\frac{T_I}{T_{(TRANS.)}} = \left(\frac{2.2}{0.40}\right) = 5.5$$

SINCE RATIO IS GREATER THAN 3, SIMPLIFIED STATIC ASSUMPTION IS O.K.

STEP 16: RATTLESPACE AND UTILITY CONNECTIONS

PROVIDE FLEXIBILITY IN UTILITY CONNECTION AND RATTLESPACE AT LEAST EQUAL TO $D_{TM} = 13.8$ INCHES

STEP 17: ELEMENTS OF STRUCTURES AND NON-STRUCTURAL COMPONENTS.

DESIGN USING SECTION 2312(g) APPROACH WITH $I = 1.0$

$$F_p = ZIC_pW_p$$

STEP 18: OPTIMIZATION AND DYNAMIC ANALYSIS

BY PERFORMING NON-LINEAR TIME HISTORY ANALYSIS TO VERIFY RESULTS THE DESIGN REQUIREMENTS PRESENTED EARLIER CAN BE REDUCED.

BELOW THE INTERFACE:

THE TOTAL DESIGN DISPLACEMENT,

$$= 0.9 D_T = 0.9(9.2) = 8.3 \text{ INCHES}$$

THE TOTAL MAXIMUM DISPLACEMENT,

$$= 0.8 D_{TM} = 0.8(13.8) = 11.0 \text{ INCHES}$$

ABOVE THE INTERFACE:

THE DESIGN SHEAR,

REGULAR CONFIGURATION

$$= 0.60 V_S = (0.60)(322.5) = 193.5^{KIPS}$$

IRREGULAR CONFIGURATION

$$= 0.80 V_S = (0.80)(322.5) = 258.0^{KIPS}$$

STEP 19: CONSIDERATIONS IN PERFORMING DYNAMIC ANALYSIS:

TWO TYPES OF ANALYSIS,

1) RESPONSE SPECTRA
2) TIME HISTORY

RESPONSE SPECTRA,

a) 1ST MODE → HIGH DAMPING

b) OTHER MODES → LOW DAMPING

TIME HISTORY,

a) USE BETWEEN 3 AND 7 TIME HISTORIES.

b) TIME HISTORIES SHOULD BE REPRESENTATIVE OF ACTUAL EARTHQUAKES AND SCALED SO THAT THE SRSS OF THE 5.% DAMPED SPECTRA FOR THE HORIZONTAL COMPONENTS DOES NOT FALL BELOW 1.3 TIMES THE DESIGN BASIS EARTHQUAKE SPECTRA BY MORE THAN 10.% IN THE PERIOD RANGE OF T_I MINUS ONE SECOND TO T_I PLUS TWO SECONDS.

c) WITHIN 15. KM OF AN ACTIVE FAULT, TIME HISTORIES SHALL INCORPORATE NEAR FAULT EFFECTS.

DYNAMIC ANALYSIS MODEL – TYPICAL ASSUMPTIONS

1) HORIZONTAL DIAPHRAM MODEL – VERTICAL GROUND MOTION IGNORED.

2) BOTH HORIZONTAL DIRECTIONS OF GROUND MOTION CONSIDERED AND RESULTS ARE COMBINED BY SRSS FOR RESPONSE SPECTRA ANALYSIS.

3) EQUIVALENT STIFFNESS USED FOR ISOLATORS FOR RESPONSE SPECTRA ANALYSIS.

4) MODEL SHOULD INCLUDE EFFECTS OF ACTUAL AND ACCIDENTAL ECCENTRIC MASS.

COMPUTER CODES – TIME HISTORY

1) N-PAD
2) 3D-BASIS (NCEER)
3) DRAIN-2D (TWO DIMENSIONAL ANALYSIS)

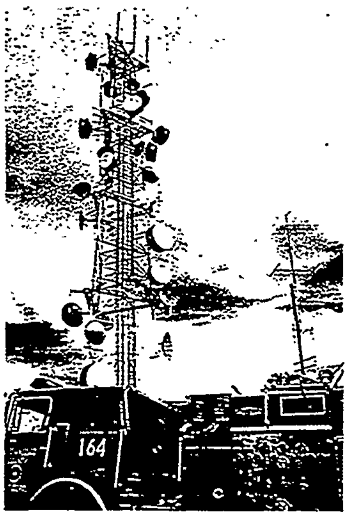
COMPUTER CODES – RESPONSE SPECTRA

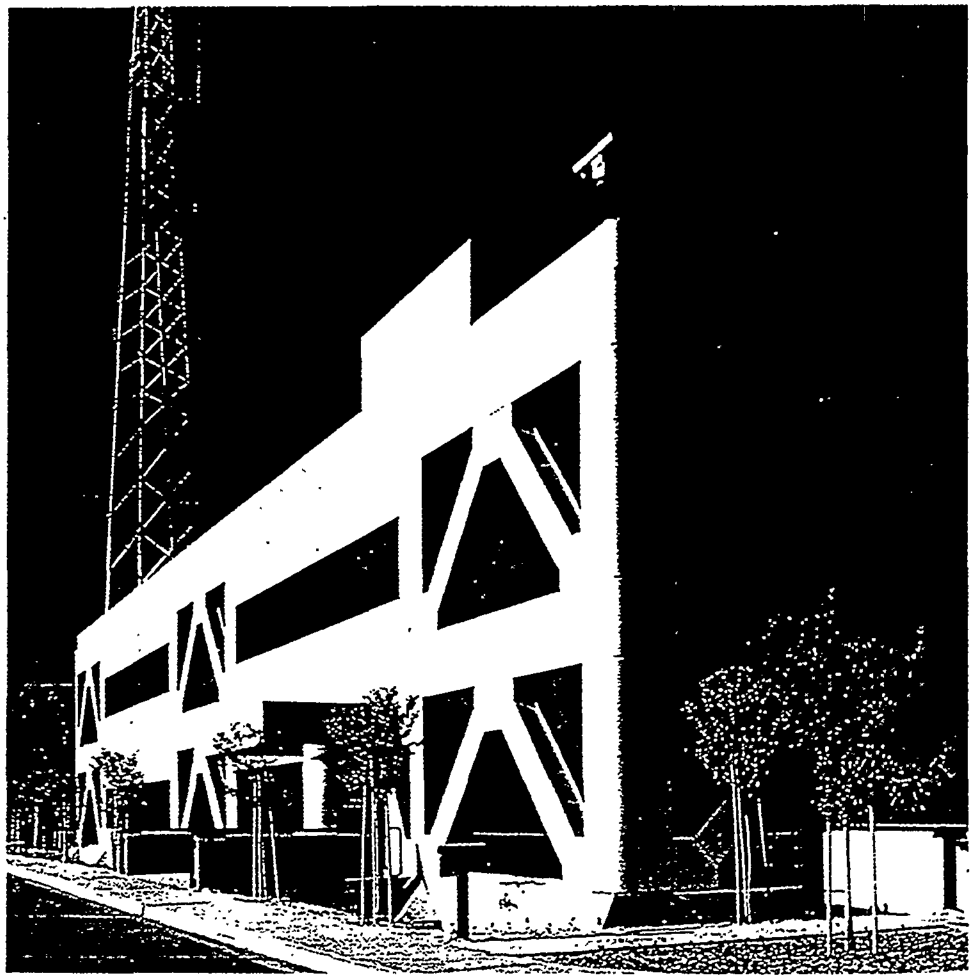
1) E-TABS
2) SK-COMBAT
3) SAP-90

SPECIAL ITEMS OF CONCERN

1) P-DELTA EFFECTS ACROSS ISOLATOR MUST BE CONSIDERED IN DESIGN AND ANALYSIS OF COLUMNS, ISOLATORS, AND FOUNDATIONS.

2) VERTICAL ACCELERATION EFFECTS MUST BE CONSIDERED IF EQUIPMENT FUNCTIONALITY IS IMPORTANT AND EQUIPMENT IS SENSATIVE TO VERTICAL VIBRATIONS IN THE 8 TO 15 CYCLES PER SECOND FREQUENCY RANGE.





Project:
Fire Command and Control System

Location:
Los Angeles County, California

Client:
Los Angeles County Fire Department

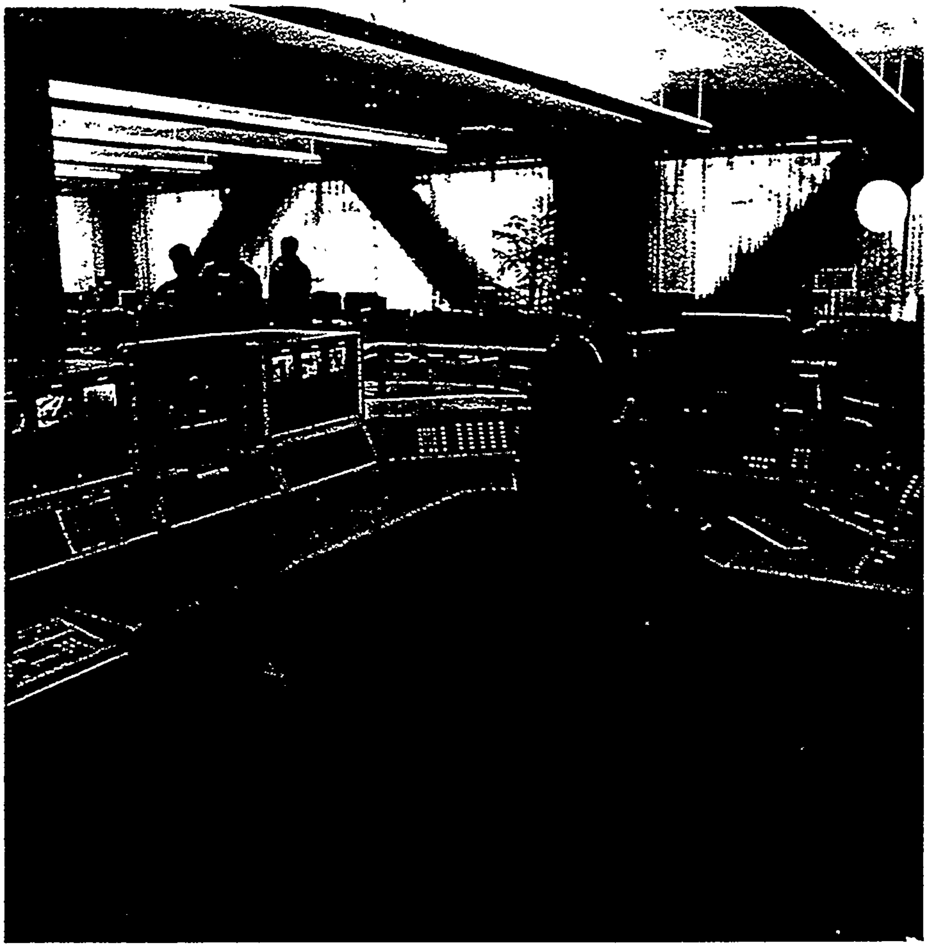
Scope:
System Design and Installation, Architectural/Engineering Design, Construction Monitoring

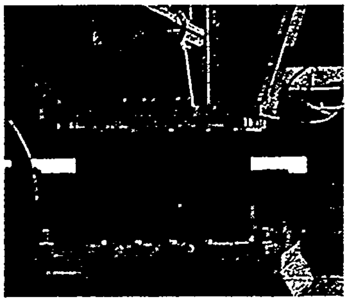
The Los Angeles County Fire Department (LACFD) instituted a four-year program to expand and modernize the capabilities of its fire and emergency communications system. When completed, the new Fire Command Control System will provide computer-aided dispatch/digital mobile radio communications to LACFD units in a 4,000 square mile area and modernize communications for over 750 department vehicles, 150 fire stations and 16 field camps. Under subcontract to Planning Research Corporation, Fluor Daniel was selected to engineer, furnish and install a major portion of the system.

Responsibilities were divided into three areas of activity: design and construction monitoring of the Fire Command and Control Facility; design, installation and testing of the UHF digital dispatch link; and modernization of command and control systems.

The 33,000 square foot Fire Command and Control Facility is the center for communications activity including 'E-911' emergency calls, voice recorders, dispatch consoles and administrative telephone services. Special design considerations included the facility's need to function during and after a natural disaster. It was designed to be fully self-contained with special features such as emergency backup power system, diverse routing for telephone feeds, and a state-of-the-art base

Over





isolator foundation system (see photo above) to allow the facility to withstand a magnitude 8.3 earthquake.

To increase the ability to handle more emergency calls over a shorter period of time and save time in the process, a new digital UHF mobile radio dispatch data link was designed and installed by Fluor Daniel. The system, controlled from the Fire Command and Control Facility's operations center, provides dispatch/data communications with fire stations, remote fire camps, and LACFD vehicles equipped with new mobile digital terminals.

Approximately 23 major command and control subsystems were also upgraded, including VHF voice radio, various emergency radios, private branch exchange (PBX) switch, Enhanced 911, alarm and security, paging and a complete emergency standby power system.

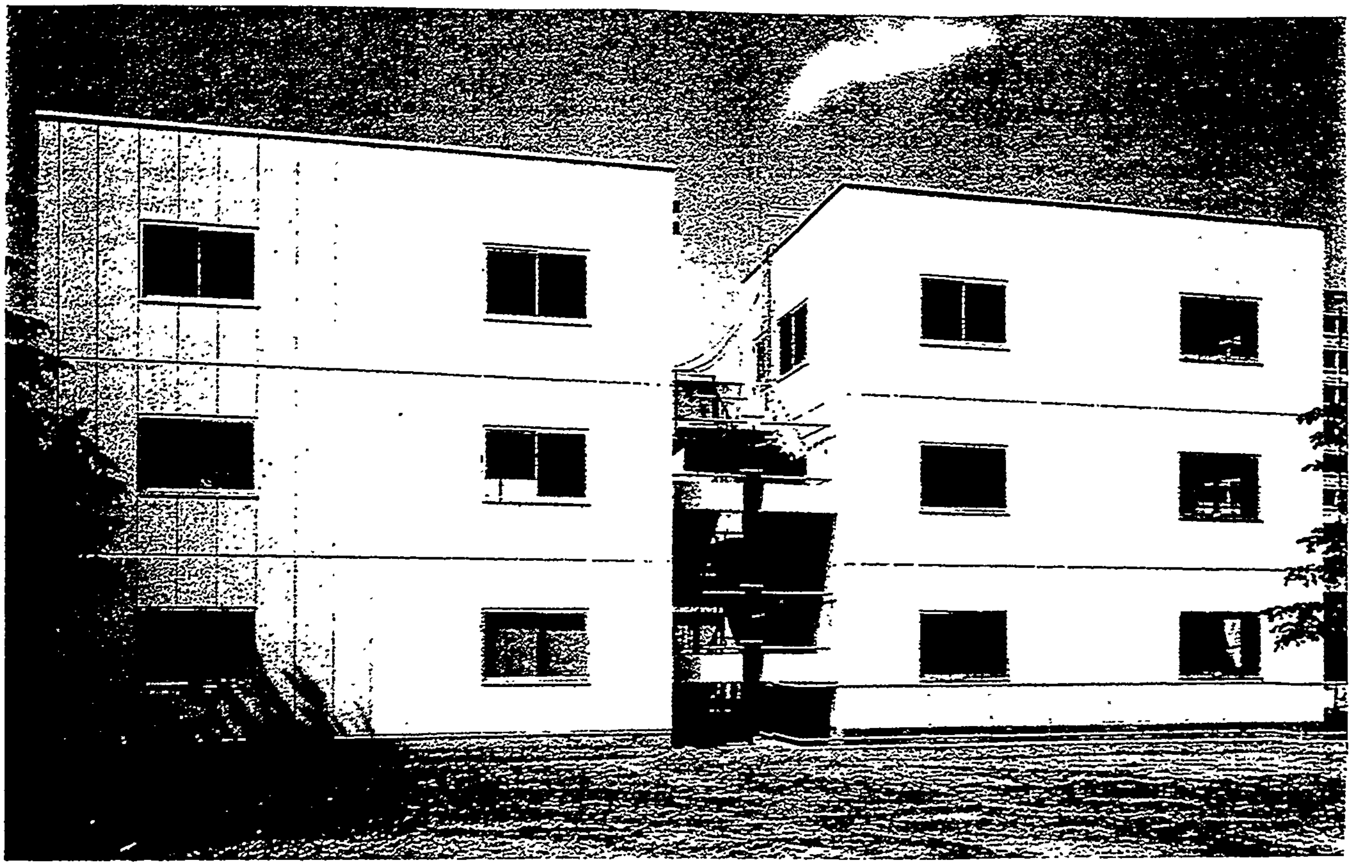
Department of Energy Short Course
on Seismic Base Isolation

DESCRIPTION OF STUDENT DESIGN PROBLEM

Earthquake Engineering Research Center
James Kelly
Ian Aiken
Peter Clark

August 13, 1992

Structure to be Considered for Design Problem



Full Scale Demonstration Buildings in Sendai, Japan
Left: Fixed-Base Building Right: Base-Isolated Building

0.4 Scale Model Structure Constructed and Tested at EERC
This Structure is used for the Student Design Problem

Tohoku University Twin Test Buildings

The test buildings are a joint project between Tohoku University and Shimizu Corporation. Construction of the buildings by Shimizu was completed in May 1986. The buildings are full-sized three-story, reinforced-concrete structures, and the dimensions and construction method of the superstructures were exactly the same for both buildings.

One test building is base isolated, and the other has an ordinary fixed-base foundation. The foundation details of the isolated building are such that it relatively easy to change bearing systems. To-date, four different isolation systems have been installed under the building. Both buildings (isolated and fixed-base) are comprehensively instrumented, and a large number of (relatively low-level) earthquakes have been recorded.

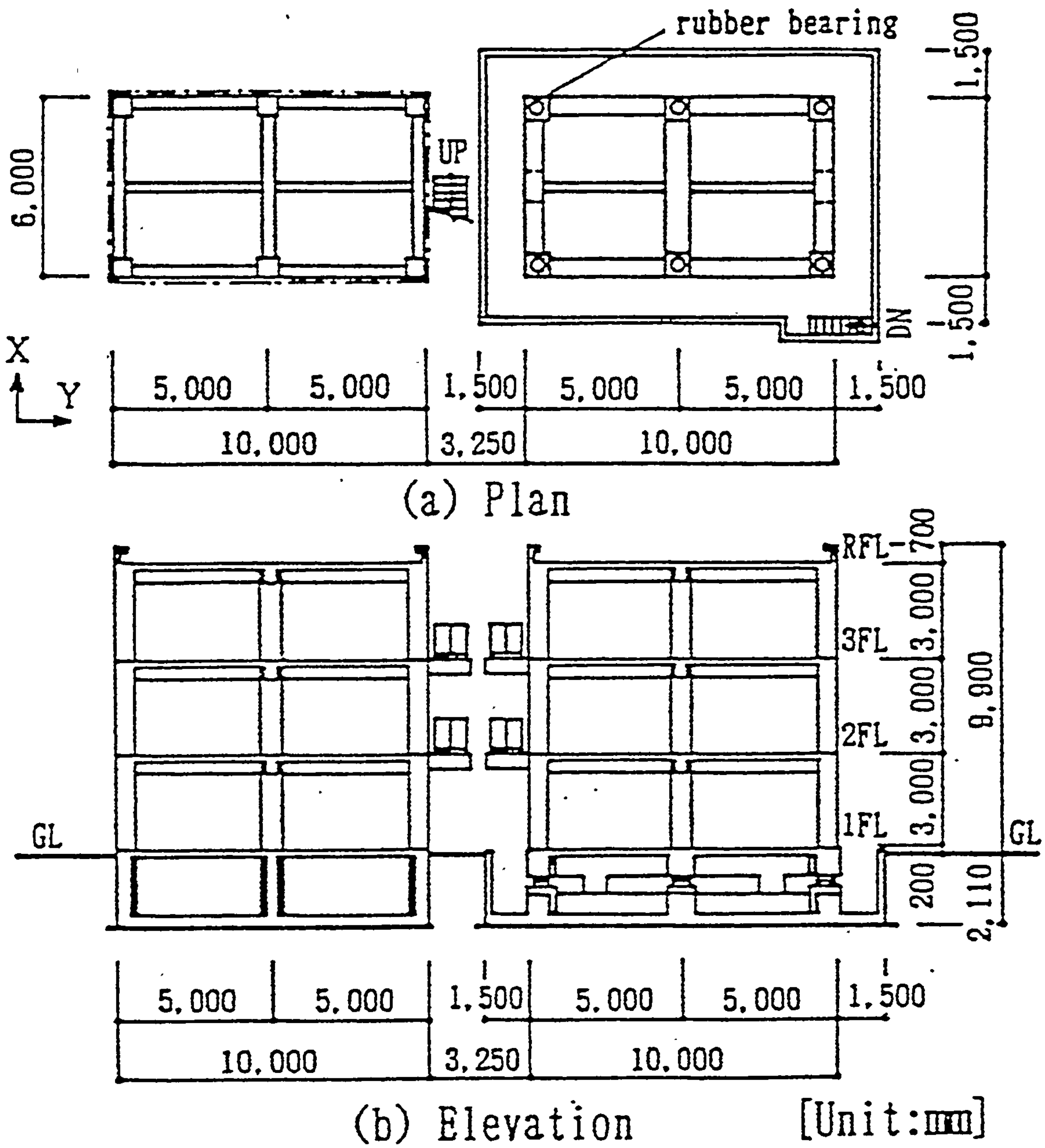


Fig. 2 Plan and elevation of test buildings

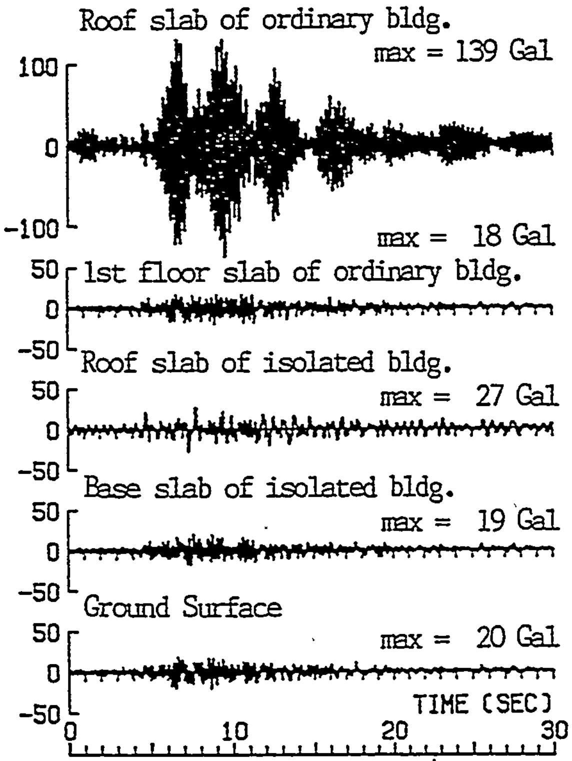


Fig. 8 Time history for X-direction of the earthquake on October 4, 1987

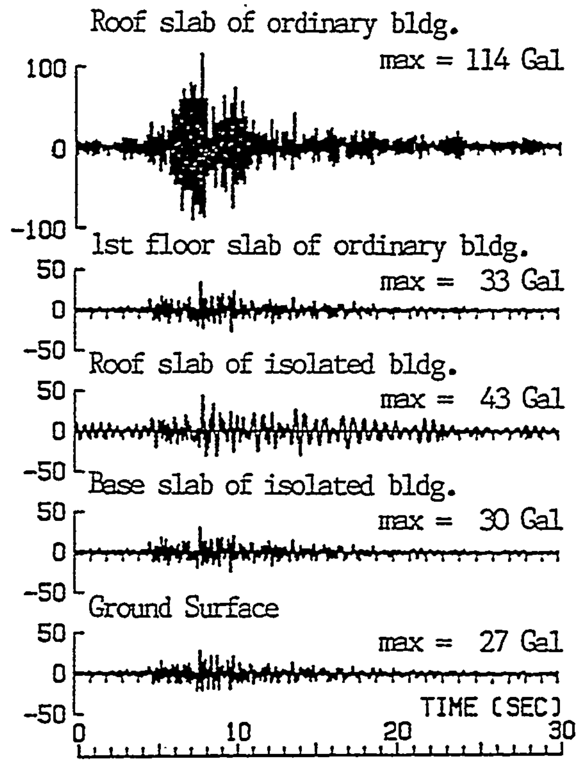


Fig. 9 Time history for Y-direction of the earthquake on October 4, 1987

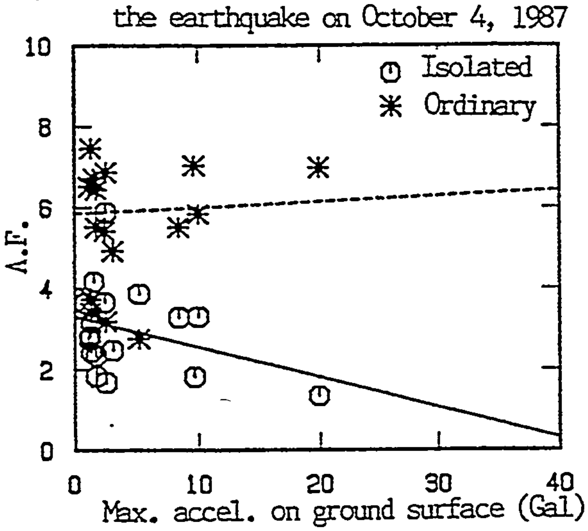


Fig. 10 Amplification factor in X-direction

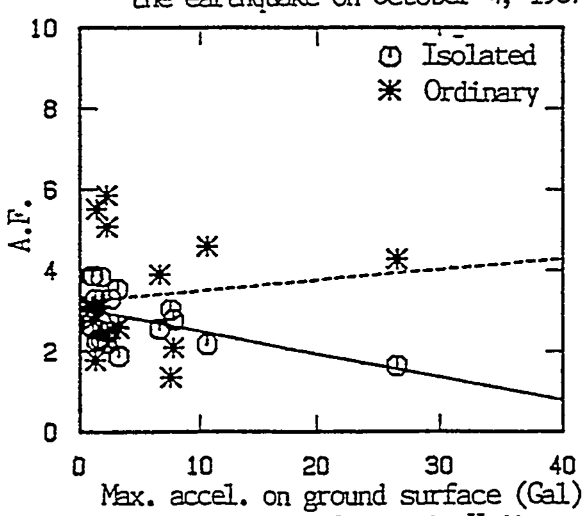
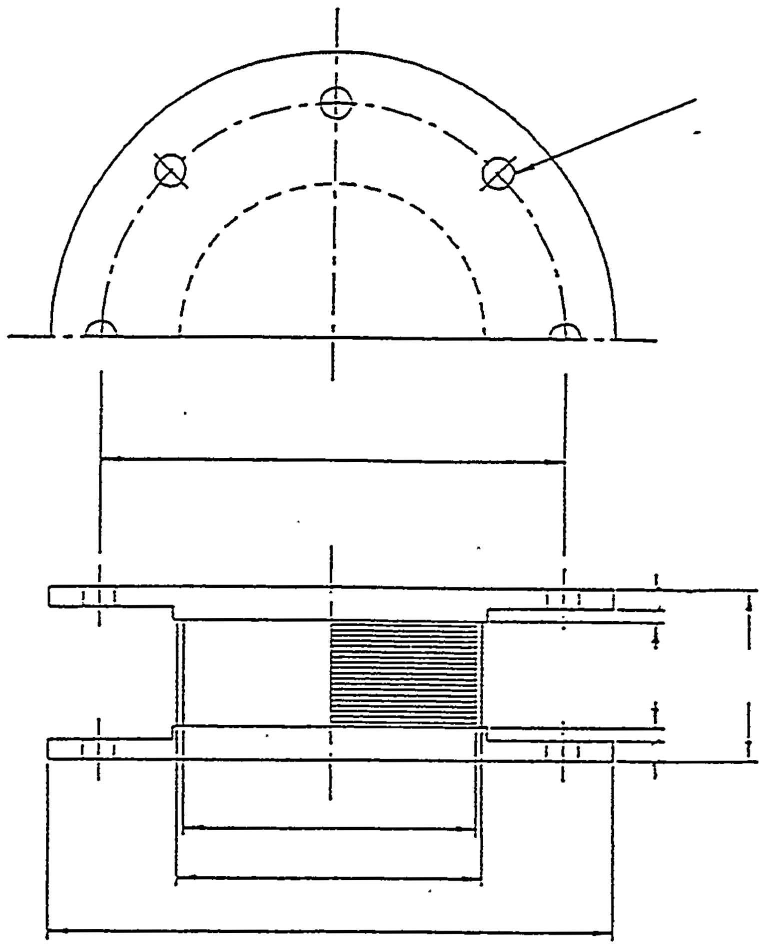
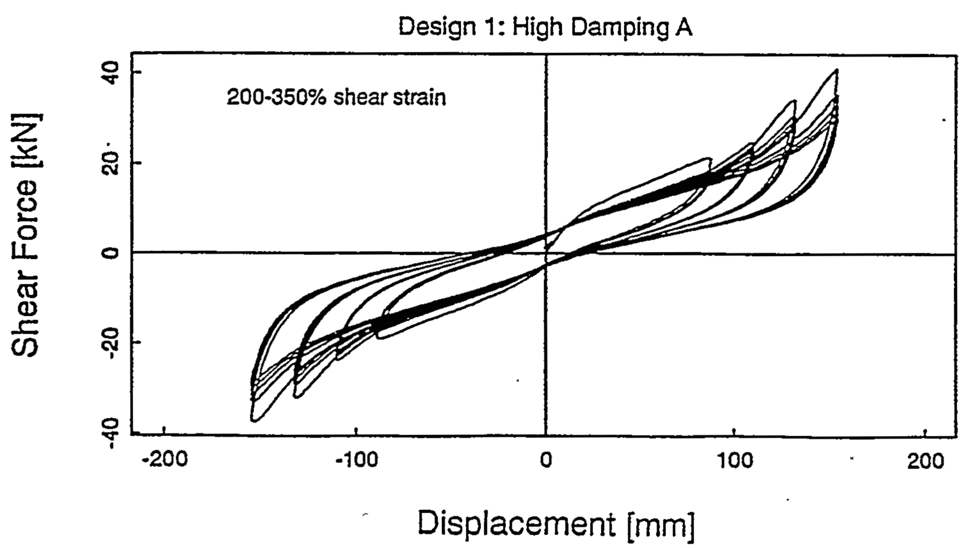


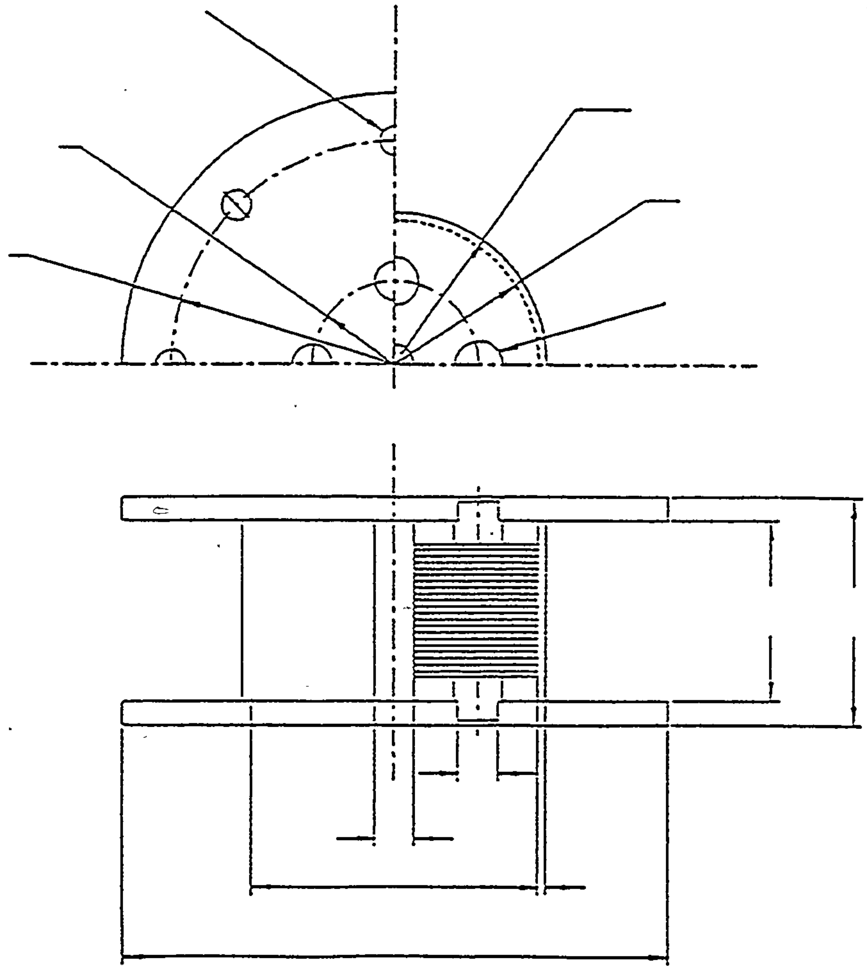
Fig. 11 Amplification factor in Y-direction

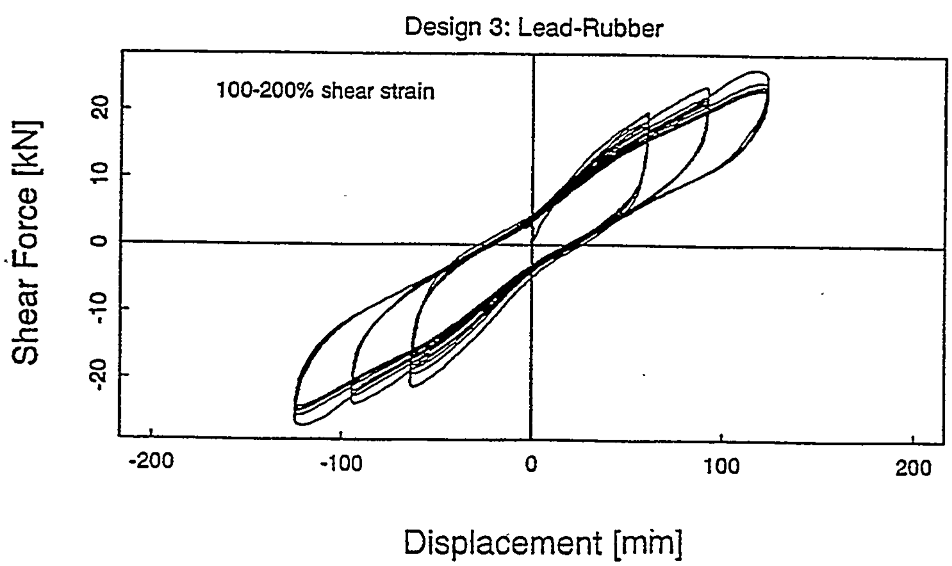
Bearing design 1: high-damping A





Bearing design 3: lead-rubber



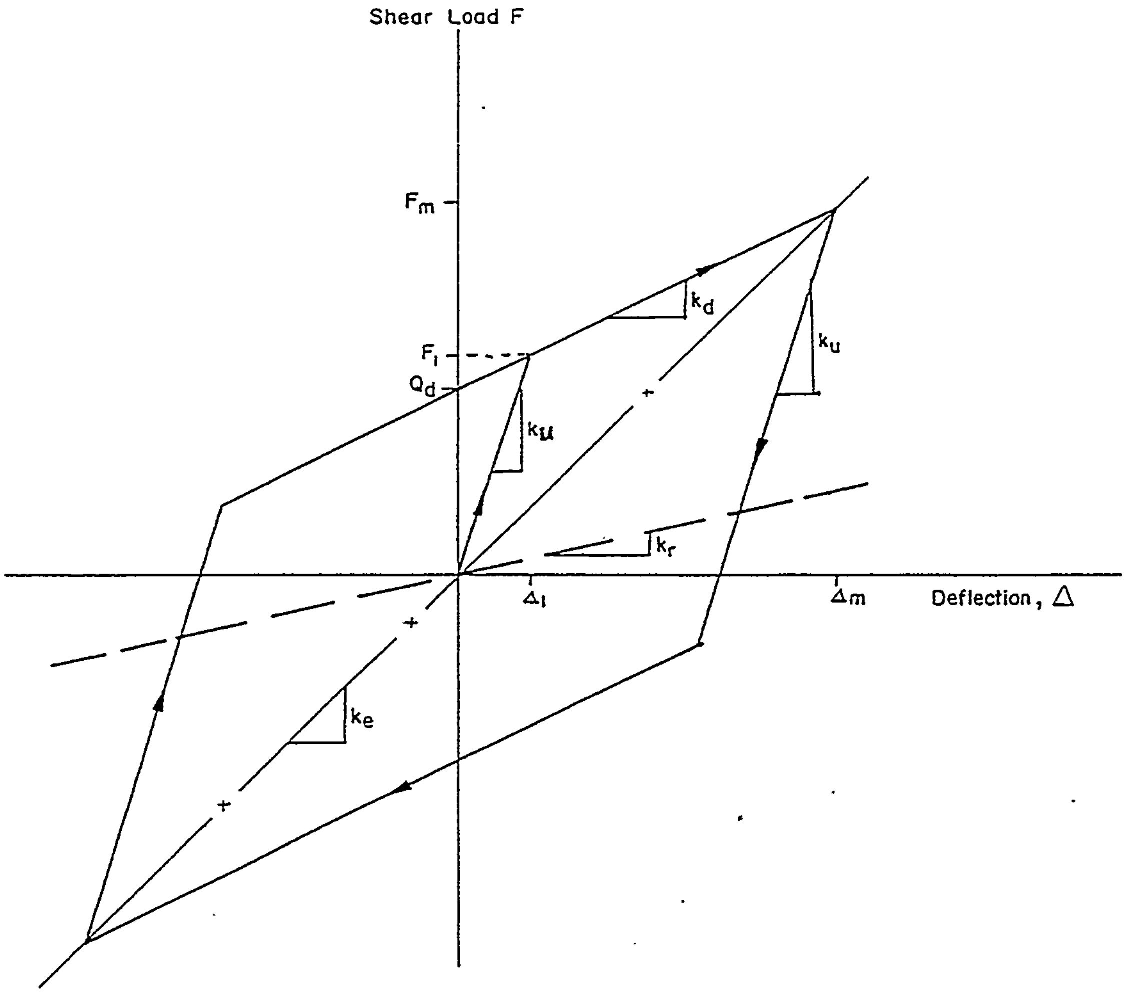


Ministry of Works
and Development

DESIGN OF LEAD-RUBBER BRIDGE BEARINGS

Civil Engineering Division

C D P 818/A: 1983



Empirical Force-Deformation Relationship:

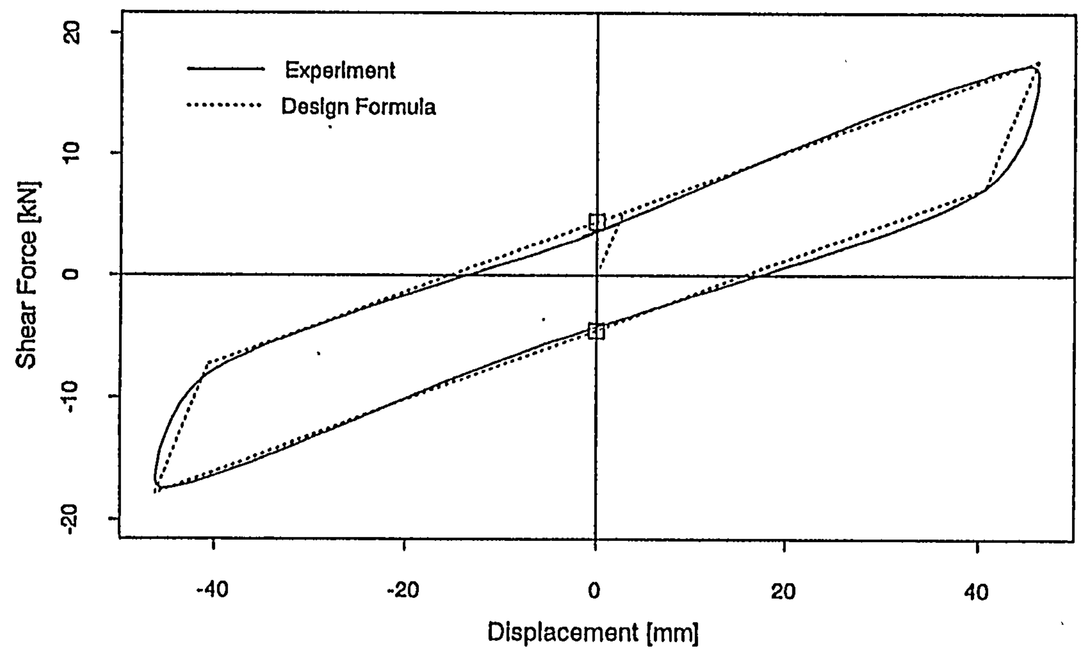
$$k_d = k_r \left[1 + 12 \frac{A_p}{A_r} \right]$$

$$k_r = \frac{GA_r}{T_r}$$

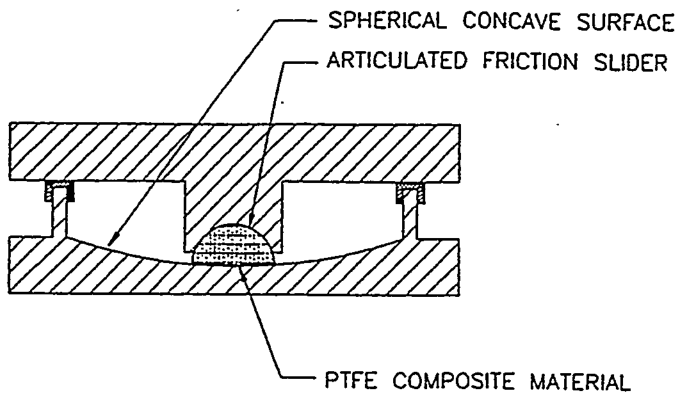
$$k_u = 6.5\, k_d$$

$$Q_d = F_1 \left(1 - \frac{k_d}{k_u} \right) = 0.85\, F_1$$

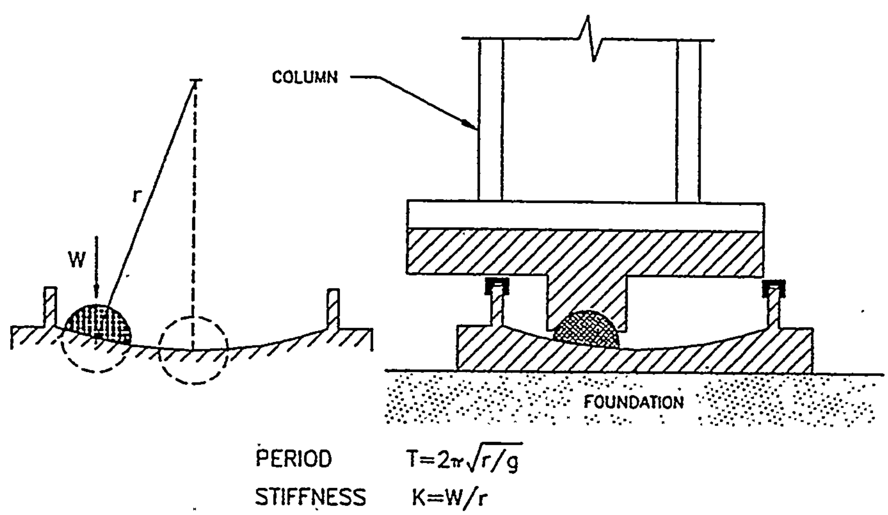
$$F_1 = f_{pb} A_p \quad (f_{pb} = 9\,\text{MPa}$$
$= 1300$ psi



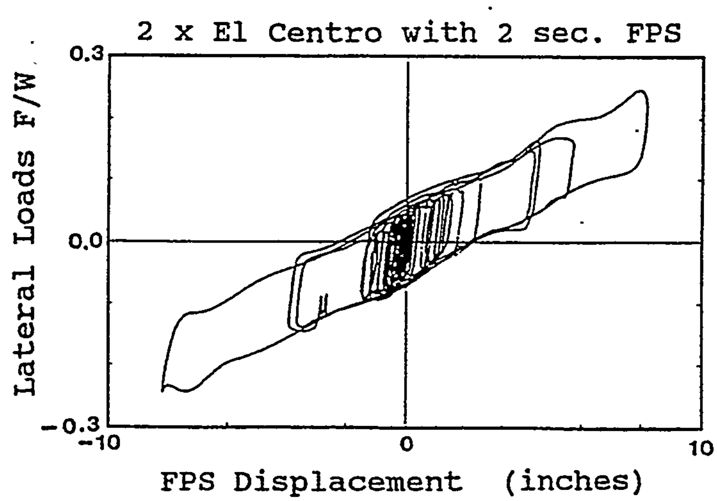
Experimental and Design Formula Hysteresis Loops,
Lead-Rubber Bearings, 75 Percent Shear Strain



FPS Isolator Section



Friction Pendulum Concept



FPS Hysteretic Response

UNIFORM BUILDING CODE™

1991 Edition

Division III

EARTHQUAKE REGULATIONS FOR SEISMIC-ISOLATED STRUCTURES

NOTE: This is a new division

General

Sec. 2370. Every seismic-isolated structure and every portion thereof shall be designed and constructed in accordance with the requirements of this division and the applicable requirements of Chapter 23, Part III.

The lateral force-resisting system and the isolation system shall be designed to resist the deformations and stresses produced by the effects of seismic ground motions as provided in this division.

Where wind forces prescribed by Chapter 23, Part II, produce greater deformations or stresses, such loads shall be used for design in lieu of the deformations and stresses resulting from earthquake forces.

Definitions

Sec. 2371. The definitions of Section 2331 and the following apply to the provisions of this division:

DESIGN-BASIS EARTHQUAKE is defined in Section 2335 (b).

DESIGN DISPLACEMENT is the design-basis earthquake lateral displacement, excluding additional displacement due to actual and accidental torsion, required for design of the isolation system.

EFFECTIVE DAMPING is the value of equivalent viscous damping corresponding to energy dissipated during cyclic response of the isolation system.

EFFECTIVE STIFFNESS is the value of the lateral force in the isolation system, or an element thereof, divided by the corresponding lateral displacement.

ISOLATION INTERFACE is the boundary between the upper portion of the structure, which is isolated, and the lower portion of the structure, which moves rigidly with the ground.

ISOLATION SYSTEM is the collection of structural elements which includes all individual isolator units, all structural elements which transfer force between elements of the isolation system, and all connections to other structural elements.

The isolation system also includes the wind-restraint system if such a system is used to meet the design requirements of this section.

ISOLATOR UNIT is a horizontally flexible and vertically rigid structural element of the isolation system which permits large lateral deformations under design seismic load. An isolator unit may be used either as part of or in addition to the weight-supporting system of the building.

MAXIMUM CREDIBLE EARTHQUAKE is the maximum level of earthquake ground shaking which may ever be expected at the building site within the known geological framework. This intensity may be taken as the level of earthquake ground motion that has a 10 percent probability of being exceeded in a 250-year time period.

TOTAL DESIGN DISPLACEMENT is the design-basis earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for design of the isolation system, or an element thereof.

TOTAL MAXIMUM DISPLACEMENT is the maximum credible earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for verification of the stability of the isolation system, or elements thereof, design of building separations, and vertical load testing of isolator unit prototypes.

WIND-RESTRAINT SYSTEM is the collection of structural elements which provide restraint of the seismic-isolated structure for wind loads. The wind-restraint system may be either an integral part of isolator units or may be a separate device.

Symbols and Notations

Sec. 2372. The symbols and notations of Section 2332 and the following provisions apply to the provisions of this division:

B = numerical coefficient related to the effective damping of the isolation system as set forth in Table No. A-23-W.

b = the shortest plan dimension of the structure, in feet, measured perpendicular to d.

D = design displacement, in inches, at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Formula (74-1).

D_T = total design displacement, in inches, of an element of the isolation system including both translational displacement at the center of rigidity, D, and the component of torsional displacement in the direction under consideration, as specified in Section 2374 (c) 3.

D_{TM} = total maximum displacement, in inches, of an element of the isolation system, including both translational displacement at the center of rigidity and the component of torsional displacement in the direction under consideration, as prescribed by Formula (74-4).

d = the longest plan dimension of the structure, in feet.

e = the actual eccentricity, in feet, measured in plan between the center of mass of the structure above the isolation interface and the center of rigidity of the isolation system, plus accidental eccentricity, in feet, taken as 5 percent of d.

$F-$ = maximum negative force in an isolator unit during a single cycle of prototype testing at a displacement amplitude of $\Delta-$.

F^-_{max} = maximum negative force in an isolator unit for all cycles of prototype testing at a common displacement amplitude of $\Delta-$.

F^-_{min} = minimum negative force in an isolator unit for all cycles of prototype testing at a common displacement amplitude of $\Delta-$.

$F+$ = maximum positive force in an isolator unit during a single cycle of prototype testing at a displacement amplitude of $\Delta+$.

F^+_{max} = maximum positive force in an isolator unit for all cycles of prototype testing at a common displacement amplitude of $\Delta+$.

F^+_{min} = minimum positive force in an isolator unit for all cycles of prototype testing at a common displacement amplitude of $\Delta+$.

k_{eff} = effective stiffness of an isolator unit, as prescribed by Formula (81-1).

k_{max} = maximum effective stiffness of the isolation system at the design displacement in the horizontal direction under consideration.

k_{min} = minimum effective stiffness of the isolation system at the design displacement in the horizontal direction under consideration.

N = numerical coefficient related to the proximity of the building or structure to active faults as set forth in Table No. A-23-V.

R_{wI} = numerical coefficient related to the type of lateral force-resisting system above the isolation interface as set forth in Table No. A-23-X for seismic-isolated structures.

S_I = numerical coefficient for site-soil profile as set forth in Table No. A-23-U for seismic-isolated structures.

T_I = period of seismic-isolated structure, in seconds, in the direction under consideration, as prescribed by Formula (74-2).

V_b = the total lateral seismic design force or shear on elements at or below the isolation interface, as prescribed by Formula (74-5).

V_s = the total lateral seismic design force or shear on elements above the isolation interface, as prescribed by Formula (74-6).

W = the total seismic dead load defined in Section 2334 (a). For design of the isolation system, W is the total seismic dead load weight of the structure above the isolation interface.

y = the distance, in feet, between the center of rigidity of the isolation system rigidity and the element of interest, measured perpendicular to the direction of seismic loading under consideration.

$\Delta+$ = maximum positive displacement of an isolator unit during each cycle of prototype testing.

$\Delta-$ = *maximum negative displacement of an isolator unit during each cycle of prototype testing.*

β = effective damping of the isolation system, as prescribed by Formula (81-2).

Criteria Selection

Sec. 2373. (a) **Basis for Design.** The procedures and limitations for the design of seismic-isolated structures shall be determined considering zoning, site characteristics, *occupancy, configuration, structural system and height in accordance* with Section 2333, except as noted below.

(b) **Stability of the Isolation System.** The stability of the vertical load-carrying elements of the isolation system shall be verified by analysis and test, as re*quired, for lateral seismic displacement equal to the total maximum displacement.*

(c) **Occupancy Categories.** The importance factor, I, for a seismic-isolated building shall be taken as 1.0 regardless of occupancy category.

(d) **Configuration Requirements.** Each structure shall be designated as being regular or irregular on the basis of the structural configuration above the isolation interface.

(e) **Selection of Lateral Response Procedure. 1. General.** Any seismic-isolated structure may be, and certain seismic-isolated structures defined below shall be, designed using the dynamic lateral response procedure of Section 2375.

2. **Static analysis.** The static lateral response procedure of Section 2374 may be used for design of a seismic-isolated structure, provided:

A. The structure is located at least 15 kilometer (km) from all active faults.

B. The structure is located on a soil profile with a site factor of S_1 or S_2.

C. The structure is located in Seismic Zone No. 3 or 4.

D. The structure above the isolation interface is equal to or less than four stories, or 65 feet, in height.

E. The isolated period of the structure, T_I, is equal to or less than 3.0 seconds.

F. The isolated period of the structure, T_I, is greater than 3 times the elastic, fixed-base period of the structure above the isolation interface, as determined by *Formula (12-5) of Section 2334.*

G. The structure above the isolation interface is of regular configuration.

H. The isolation system does not limit the total maximum displacement to less than 1.5 times the total design displacement.

I. The isolation system is defined by all of the following attributes:

(i) The effective stiffness of the isolation system at the design displacement is greater than one third of the effective stiffness at 20 percent of the design displacement.

(ii) The isolation system is capable of producing a restoring force, as specified in Section 2377 (b) 4.

(iii) The isolation system has force-deflection properties which are independent of the rate of loading.

(iv) The isolation system has force-deflection properties which are independent of vertical load and bilateral load.

3. **Dynamic analysis.** The dynamic lateral response procedure of Section 2375 shall be used for design of seismic-isolated structures as specified below:

A. **Response spectrum analysis.** Response spectrum analysis may be used for design of a seismic-isolated structure, provided:

(i) The structure is located on a soil profile with a site factor of S_1, S_2 or S_3.

(ii) The isolation system *is defined by all of the attributes specified in Section* 2373 (e) 2 H and I.

B. **Time-history analysis.** Time-history analysis may be used for design of any seismic-isolated structure and shall be used for design of all seismic-isolated struc*tures not meeting the criteria of Section 2373 (e) 3 A.*

C. **Site-specific design spectra.** Site-specific ground motion spectra of the design-basis earthquake and the maximum credible earthquake, developed in accordance with Section 2335 (b), shall be used for design and analysis of all seismic-isolated structures as specified below:

(i) The structure is located on a soil profile with a site factor of S_3 or S_4.

(ii) The structure is located within 15 km of an active fault.

(iii) The structure is located in Seismic Zone No. 1, 2A or 2B.

(iv) The isolated period of the structure, T_I, is greater than 3.0 seconds.

Static Lateral Response Procedure

Sec. 2374. (a) **General.** Except as provided in Section 2375, every seismic-isolated structure, or portion thereof, shall be designed and constructed to resist minimum earthquake displacements and forces as specified by this section and the *applicable requirements of Section 2334.*

(b) **Deformation Characteristics of the Isolation System.** Minimum lateral earthquake design displacements and forces on seismic-isolated structures shall be based on the deformation characteristics of the isolation system.

The deformation characteristics of the isolation system shall explicitly include the effects of the wind-restraint system if such a system is used to meet the design requirements of this document.

The deformation characteristics of the isolation system shall be based on *properly* substantiated tests performed in accordance with Section 2381.

(c) **Minimum Lateral Displacements. 1. Design displacement.** The isolation system shall be designed and constructed to withstand minimum lateral earthquake displacements which act in the *direction of each of the main horizontal axes* of the structure in accordance with the formula:

$$D = 10ZNS_IT_I/B \qquad (74\text{-}1)$$

2. **Isolated-structure period.** The isolated-structure period, T_I, shall be deter*mined using the deformational characteristics of the isolation system in accor-*

dance with the formula:

$$T_I = 2\pi\sqrt{\frac{W}{k_{min}g}} \qquad (74\text{-}2)$$

3. **Total design displacement.** The total design displacement, D_T, of elements of the isolation system shall include additional displacement due to actual and accidental torsion calculated considering the spatial distribution of the lateral stiffness of the isolation system and the most disadvantageous location of mass eccentricity.

The total design displacement, D_T, of elements of an isolation system with uniform spatial distribution of lateral stiffness shall not be taken as less than that prescribed by the formula:

$$D_T = D\ [1 + y[12e/(b^2 + d^2)]] \qquad (74\text{-}3)$$

The total design displacement, D_T, may be taken as less than the value prescribed by Formula (74-3), but not less than 1.1 times D, provided the isolation system is shown by calculation to be configured to resist torsion accordingly.

4. **Total maximum displacement.** The total maximum displacement, D_{TM}, required for verification of isolation system stability in the most critical direction of horizontal response shall be calculated in accordance with the formula:

$$D_{TM} = 1.5\ D_T \qquad (74\text{-}4)$$

(d) **Minimum Lateral Forces. 1. Structural elements at or below the isolation interface.** The isolation system, the foundation, and all structural elements below the isolation interface shall be designed and constructed to withstand a minimum lateral seismic force, V_b, using all of the appropriate provisions for a nonisolated structure where:

$$V_b = \frac{k_{max}D}{1.5} \qquad (74\text{-}5)$$

2. **Structural elements above the isolation interface.** The structure above the isolation interface shall be designed and constructed to withstand a minimum shear force, V_s, using all of the appropriate provisions for a nonisolated structure where:

$$V_s = \frac{k_{max}D}{R_{wI}} \qquad (74\text{-}6)$$

The R_{wI} factor shall be based on the type of lateral force-resisting system used for the structure above the isolation system.

3. **Limits on V_s.** The value of V_s shall not be taken as less than the following:

A. The lateral seismic force required by Chapter 23, Part III, for a fixed-base structure of the same weight, W, and a period equal to the isolated period, T_I.

B. The base shear corresponding to the design wind load.

C. The lateral seismic force required to fully activate the isolation system (e.g., the yield level of a softening system, the ultimate capacity of a sacrificial wind-restraint system or the static friction level of a sliding system).

(e) **Vertical Distribution of Force.** The total force shall be distributed over the height of the structure above the isolation interface in accordance with the formula:

$$F_x = \frac{V_s w_x}{\sum_{i=1}^{n} w_i} \qquad (74\text{-}7)$$

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution at the level. Stresses in each structural element shall be calculated as the effect of force, F_x, applied at the appropriate levels above the base.

(f) **Drift Limits.** The maximum interstory drift ratio of the structure above the isolation system shall not exceed $0.010/R_{wI}$.

Dynamic Lateral Response Procedure

Sec. 2375. (a) **General.** As required by Section 2373, every seismic-isolated structure, or portion thereof, shall be designed and constructed to resist earthquake displacements and forces as specified in this section and the applicable requirements of Section 2335.

(b) **Isolation System and Structural Elements below the Isolation Interface.** The total design displacement of the isolation system shall not be taken as less than 90 percent of D_T as specified by Section 2374 (c) 3.

The total maximum displacement of the isolation system shall not be taken as less than 80 percent of D_{TM} as prescribed by Formula (74-4).

The design lateral shear force on the isolation system and structural elements below the isolation interface shall not be taken as less than 90 percent of V_b as prescribed by Formula (74-5).

(c) **Structural Elements above the Isolation Interface.** The design lateral shear force on the structure above the isolation interface, if regular in configuration, shall not be taken as less than 80 percent of $k_{max}D/R_{wI}$, or less than the limits specified by Section 2374 (d) 3.

> EXCEPTION: The design lateral shear force on the structure above the isolation interface, if regular in configuration, may be taken as less than 80 percent of $k_{max}D/R_{wI}$, but not less than 60 percent of $k_{max}D/R_{wI}$, provided time-history analysis is used for design of the structure.

The design lateral shear force on the structure above the isolation interface, if irregular in configuration, shall not be taken as less than $k_{max}D/R_{wI}$, or less than the limits specified by Section 2374 (d) 3.

> EXCEPTION: The design lateral shear force on the structure above the isolation interface, if irregular in configuration, may be taken as less than $k_{max}D/R_{wI}$, but not less than 80 percent of $k_{max}D/R_{wI}$, provided time-history analysis is used for design of the structure.

(d) **Ground Motion. 1. Design spectra.** Properly substantiated, site-specific spectra are required for design of all structures with an isolated period, T_I, greater than 3.0 seconds, or located on a soil type profile of S_3 or S_4, or located within 15 km of an active fault or located in Seismic Zone No. 1, 2A or 2B. Structures not requiring site-specific spectra shall be designed using spectra based on Figure No. 23-3 of Chapter 23, Part III.

A design spectrum shall be constructed for the design-basis earthquake. This design spectrum shall not be taken as less than the normalized response spectrum given in Figure No. 23-3 of Chapter 23, Part III, for the appropriate soil type, scaled by the seismic zone coefficient.

EXCEPTION: If a site-specific spectrum is calculated for the design-basis earthquake, then the design spectrum may be taken as less than 100 percent, but not *less than 80 percent of the normalized response spectrum given in* Figure No. 23-3 of Chapter 23, Part III, the appropriate soil type, scaled by the seismic zone coefficient.

A design spectrum shall be constructed for the maximum credible earthquake. This *design spectrum shall not be taken as less than 1.25 times the design basis* earthquake spectrum. This design spectrum shall be used to determine the total maximum displacement for testing of the stability of the base-isolation system.

2. **Time histories.** Pairs of horizontal ground motion time-history components shall be selected *from not less than three recorded events. These motions shall be* scaled such that the square root sum of the squares (SRSS) of the 5 percent-damped spectrum of the scaled horizontal components does not fall below *1.3 times the 5* percent-damped spectrum of the design-basis earthquake (or maximum credible earthquake) by *more than 10 percent in the period range of T_I,* as determined by Formula (74-2), for periods from T_I minus 1.0 seconds to T_I, plus 2.0 seconds.

The *duration of the time histories shall be consistent* with the magnitude and source characteristics of the design-basis earthquake (or maximum credible earthquake).

Time histories developed for sites within 15 km of a major active fault shall incorporate near-fault phenomena.

(e) **Mathematical Model. 1. General.** The mathematical models of the isolated structure, including the isolation system, the lateral force-resisting system and other structural elements, shall conform to Section 2335 (c) and to the requirements of Subsections 2 and 3 below.

2. **Isolation system.** The isolation system shall be modeled using deformational characteristics developed and verified by test in accordance with the requirements of Section 2374 (b).

The isolation system shall be modeled with sufficient detail to:

A. Account for the spatial distribution of isolator units,

B. Calculate translation, in both horizontal directions, and torsion of the structure above the isolation interface, *considering the most disadvantageous location* of mass eccentricity,

C. Assess *overturning/uplift forces on individual isolator units*, and

D. Account for the effects of vertical load, bilateral load and/or the rate of loading if the force deflection properties of the isolation system are dependent on one or more of these attributes.

3. **Isolated structure. A. Displacement.** *The maximum* displacement of each floor and the total design displacement and total maximum displacement across the isolation system shall be calculated using *a model of the isolated structure* which incorporates the force-deflection characteristics of nonlinear elements of the isolation system and the lateral force-resisting system.

Isolation systems with nonlinear elements include, but are not limited to, systems which do not meet the criteria of *Section 2373 (e) 2 I.*

Lateral force-resisting systems with nonlinear elements include, but are not limited to, irregular *structural systems designed for a lateral force less than $k_{max}D/R_{wI}$* and regular structural systems designed for a lateral force less than 80 percent of $k_{max}D/R_{wI}$.

B. **Forces and displacements in key elements.** Design forces and displace*ments in key elements of the lateral force-resisting system* may be calculated using a linear elastic model of the isolated structure, provided:

(i) *Pseudo-elastic* properties assumed for nonlinear isolation system components are based on the maximum effective stiffness of the isolation system.

(ii) All key elements of the lateral force-resisting system are linear.

(f) **Description of Analysis Procedures. 1. General.** A *response spectrum* analysis or a time-history analysis, or both, shall be performed in accordance with Section 2335 (d) and (e) and the requirements of this section.

2. **Input earthquake.** The design-basis earthquake shall be used to calculate the total design displacement of the *isolation system and the lateral forces and dis*placements of the isolated structure. *The maximum credible earthquake shall be* used to calculate the total maximum *displacement of the isolation system.*

3. **Response spectrum analysis.** Response spectrum analysis shall be performed using a damping value *equal to the effective damping of the isolation sys*tem or 30 percent of critical, whichever is less.

Response *spectrum analysis used* to determine the total design displacement and the total maximum displacement shall include simultaneous excitation of the *model by 100 percent of the most critical direction of ground motion and 30 percent* of the ground motion on the orthogonal axis.

4. ***Time-history* analysis.** Time-history analysis shall be performed with at least three appropriate pairs of horizontal time-history components, as defined in Section 2375 (d) 2.

Each pair of time histories shall be applied simultaneously to the model, considering *the most disadvantageous location* of mass eccentricity.

The maximum response of the parameter of interest calculated by the three time-history analyses shall be used for design.

(g) **Design Lateral Force. 1.** ***Structural elements*** **at or below the isolation interface.** The isolation system, the foundation and all structural elements below the isolation interface shall be designed using all of the *appropriate provisions* for a nonisolated structure and the forces obtained from the dynamic analysis reduced by a factor of 1.5.

2. **Structural elements above the isolation interface.** Structural elements above the isolation interface shall be designed using all of the appropriate provisions *for a nonisolated structure and the forces obtained from the dynamic analysis* reduced by a factor of R_{wI}. The R_{wI} factor shall be based on the type of lateral force-resisting system used for the structure above the isolation system.

3. **Scaling of results.** When the factored lateral shear force on structural elements, determined using either response spectrum or time-history analysis, is less than the minimum level prescribed by Section 2375 (a), then all response parameters, including member forces and moments shall be adjusted upward proportionally.

(h) **Drift Limits.** Maximum interstory drift corresponding to the design lateral force shall not exceed the following limits:

1. The maximum interstory drift ratio of the structure above the isolation system, calculated by response spectrum analysis, shall not exceed $0.015/R_{wI}$.

The maximum interstory drift ratio of the structure above the isolation system, calculated by time-history analysis considering the force-deflection characteristics of nonlinear elements of the lateral force-resisting system, shall not exceed $0.020/R_{wI}$.

The secondary effects of the maximum credible earthquake lateral displacement (delta) of the structure above the isolation system combined with gravity forces shall be investigated if the interstory drift ratio exceeds $0.010/R_{wI}$.

Lateral Load on Elements of Structures and Nonstructural Components Supported by Structures

Sec. 2376. (a) **General.** Parts or portions of an isolated structure, permanent nonstructural components and the attachments to them, and the attachments for permanent equipment supported by a structure shall be designed to resist seismic forces and displacements as prescribed by this section and the applicable requirements of Section 2336.

(b) **Forces and Displacements. 1. Components at or above the isolation interface.** Elements of seismic-isolated structures and nonstructural components, or portions thereof, which are at or above the isolation interface, shall be designed to resist a total lateral seismic force equal to the maximum dynamic response of the element or component under consideration divided by a factor of 1.5.

EXCEPTION: Elements of seismic-isolated structures and nonstructural components, or portions thereof, may be designed to resist total lateral seismic force as prescribed by Formula (12-10) of Section 2336.

2. **Components which cross the isolation interface.** Elements of seismic-isolated structures and nonstructural components, or portions thereof, which cross the isolation interface shall be designed to withstand the total maximum displacement.

3. **Components below the isolation interface.** Elements of seismic-isolated structures and nonstructural components, or portions thereof, which are below the isolation interface shall be designed and constructed in accordance with the requirements of Section 2336.

Detailed Systems Requirements

Sec. 2377. (a) **General.** The isolation system and the structural system shall comply with the requirements of Section 2337 and the material requirements of Chapters 24 through 28. In addition, the isolation system shall comply with the detailed system requirements of this section and the structural system shall comply with the detailed system requirements of this section and the applicable portions of Section 2337.

(b) **Isolation System. 1. Environmental conditions.** In addition to the requirements for vertical and lateral loads induced by wind and earthquake, the isolation system shall be designed with consideration given to other environmental conditions including aging effects, creep, fatigue, operating temperature and exposure to moisture or damaging substances.

2. **Wind forces.** Isolated structures shall resist design wind loads at all levels above the isolation interface in accordance with the general wind design provisions. At the isolation interface, a wind restraint system shall be provided to limit lateral displacement in the isolation system to a value equal to that required between floors of the structure above the isolation interface.

3. **Fire resistance.** Fire resistance for the isolation system shall meet that required for the building columns, walls or other structural elements.

4. **Lateral restoring force.** The isolation system shall be configured to produce a restoring force such that the lateral force at the total design displacement is at least $0.025W$ greater than the lateral force at 50 percent of the total design displacement.

EXCEPTION: The isolation system need not be configured to produce a restoring force, as required above, provided the isolation system is capable of remaining stable under full vertical load and accommodating a total maximum displacement equal to the greater of either 3.0 times the total design displacement or $36ZNS_I$ inches.

5. **Displacement restraint.** If the isolation system is configured to limit the total maximum displacement to less than 1.5 times the total design displacement, then the isolation system shall be designed for lateral forces corresponding to the maximum credible earthquake and the structure above the isolation system shall be checked for stability and ductility demand of the maximum credible earthquake. The total maximum displacement required for vertical load testing of isolator unit prototypes, building separations and verification of isolation system stability shall be calculated using time-history analysis, but shall not be taken as less than 1.2 times the total design displacement.

6. **Vertical load stability.** The isolation system shall provide a factor of safety of three for vertical loads (dead load plus live load) in its laterally undeformed state. It shall also be designed to be stable under the full design vertical loads at a horizontal displacement equal to the total maximum displacement.

7. **Overturning.** The factor of safety against global structural overturning at the isolation interface shall not be less than 1.0 for required load combinations. All gravity and seismic loading conditions shall be investigated, except that seismic forces for overturning calculations shall be based on the maximum base shear for the superstructure; and W shall be used for the vertical restoring force.

Local uplift of individual elements is permitted provided the resulting deflections do not cause overstress or instability of the isolator units or other building elements.

8. **Inspection and replacement.** Access for inspection and replacement of all components of the isolation system shall be provided.

9. **Quality control.** A quality control testing program for isolator units shall be established by the engineer responsible for the structural design.

(c) **Structural System. 1. Horizontal distribution of force.** A horizontal diaphragm or other structural elements shall provide continuity above the isolation interface and shall have adequate strength and ductility to transmit forces (due to nonuniform ground motion) from one part of the building to another.

2. **Building separations.** Minimum separations between the isolated building and surrounding retaining walls or other fixed obstructions shall not be less than the total maximum displacement.

Nonbuilding Structures

Sec. 2378. Nonbuilding structures shall be designed in accordance with the requirements of Section 2338 using design displacements and forces calculated in accordance with Section 2374 or 2375.

Foundations

Sec. 2379. Foundations shall be designed and constructed in accordance with the requirements of Chapter 29 using design forces calculated in accordance with Section 2374 or 2375.

Design and Construction Review

Sec. 2380. (a) **General.** A design review of the isolation system and related test programs shall be performed by an independent engineering team including persons licensed in the appropriate disciplines, experienced in seismic analysis methods and the theory and application of seismic isolation.

(b) **Isolation System.** Isolation system design review shall include, but not be limited to, the following:

1. Review of site-specific seismic criteria, including the development of site-specific spectra and ground motion time histories, and all other design criteria developed specifically for the project.
2. Review of the preliminary design, including the determination of the total design displacement of the isolation system design displacement and lateral force design level.
3. Overview and observation of prototype testing (Section 2381).
4. Review of the final design of the entire structural system and all supporting analyses.
5. Review of the isolation system quality control testing program [Section 2377 (b) 9].

Required Tests of Isolation System

Sec. 2381. (a) **General.** The deformation characteristics and damping values of the isolation system used in the design and analysis of seismic-isolated structures shall be based on the following tests of a selected sample of the components prior to construction.

The isolation system components to be tested shall include the wind restraint system if such systems are used in the design.

The tests specified in this section are for establishing and validating the design properties of the isolation system, and shall not be considered as satisfying the manufacturing quality control tests of Section 2377 (b) 9.

(b) **Prototype Tests. 1. General.** Prototype tests shall be performed separately on two full-size specimens of each type and size of isolator unit of the isolation system. The test specimens shall include the wind restraint system, as well as individual isolator units, if such systems are used in the design. Specimens tested shall not be used for construction.

2. **Record.** For each cycle of tests the force-deflection and hysteretic behavior of the test specimen shall be recorded.

3. **Sequence and cycles.** The following sequence of tests shall be performed for the prescribed number of cycles at a vertical load equal to the typical or average DL on all isolator units of a common type and size:

A. Twenty fully reversed cycles of loading at a lateral force corresponding to the wind design force.

B. Three fully reversed cycles of loading at each of the following increments of the total design displacement: 0.25, 0.50, 0.75 and 1.0.

C. $15S_I/B$, but not less than ten, fully reversed cycles of loading at 1.0 times the total design displacement and a vertical load equal to DL.

If an isolator unit is also a vertical load-carrying element, then Item B of the sequence of cyclic tests specified above shall be performed for two additional vertical load cases:

$$(1)\quad 1.2DL + 0.5LL + |E|$$

$$(2)\quad 0.8DL - |E|$$

where DL, LL and E are as defined in Chapter 23, Part III. In these tests, the combined vertical load shall be taken as the typical or average downward force on all isolator units of a common type and size.

4. **Units dependent on loading rates.** If the force-deflection properties of the isolator units are dependent on the rate of loading, then each set of tests specified in Section 2381 (b) 3 shall be performed dynamically at three different rates of loading. The frequency of loading shall correspond to $^1/_2$, 1 and 2 times the inverse of the isolated structure period, defined as a cycle of response at the total design displacement.

EXCEPTION: Properly documented dynamic tests of scaled specimens may be used to quantify the properties of rate-dependent systems, in lieu of the above specified dynamic tests. The scaled specimens shall be representative of full-scale prototype specimens.

The force-deflection properties of an isolator unit shall be considered to be dependent on the rate of loading if there is *greater than a ± 10 percent change in the* effective stiffness at the design displacement for either a factor of two increase or a factor of two decrease in the rate of loading, where the frequency of loading is taken as the inverse of the isolated structure period.

5. **Units dependent on bilateral load.** If the force-deflection properties of the isolator units are dependent on bilateral load, then the tests specified in Section 2381 (b) 3 and 4 shall be augmented to include bilateral load at increments of the total *design displacement* 0.25 *and* 1.0, 0.50 *and* 1.0, 0.75 *and* 1.0, and 1.0 and 1.0.

EXCEPTION: Properly documented dynamic tests of scaled specimens may be used to quantify the properties of direction dependent systems, in lieu of the above specified tests. The scaled specimens shall be representative of full-scale prototype specimens.

The force-deflection properties of an isolator unit shall be considered to be dependent on bilateral load, if the bilateral and unilateral force-deflection properties have greater than a 10 percent difference in *effective stiffness at the design displacement.*

6. **Downward vertical load.** The vertical load-carrying elements of the isolation system shall be statically tested for maximum and minimum downward vertical load, at the total maximum displacement. In these tests, the combined vertical load of $1.2DL + 1.0LL + |E|$ shall be taken as the maximum downward force, and the combined vertical load of $0.8DL - |E|$ shall be taken as the minimum downward force, on any one isolator unit of a common type and size.

7. **Sacrificial wind-restraint systems.** If a sacrificial wind-restraint system is *to be utilized, then the ultimate capacity shall be established by test.*

8. **Testing similar units.** The prototype tests are not required if an isolator unit is of *similar size and of the same type and material as the prototype isolator unit* that has been previously tested using the specified sequence of tests.

(c) **Determination of Force-deflection Characteristics.** The force-deflection characteristics of the isolation system shall be based on the cyclic load test results for each fully reversed cycle of loading.

The effective stiffness of an isolator unit shall be calculated for each cycle of loading as follows:

$$k_{eff} = \frac{F^+ - F^-}{\Delta^+ - \Delta^-} \tag{81-1}$$

where $F+$ and $F-$ are the maximum positive and maximum negative forces, respectively, and $\Delta+$ and $\Delta-$ are the maximum positive and maximum negative test displacements, respectively.

If the minimum effective stiffness is to be determined, then F^+_{min} and F^-_{min} shall be used in the equation.

If the maximum effective stiffness is to be determined, then F^+_{min} and F^-_{max} shall be used in the equation.

(d) **System Adequacy.** The performance of the test specimens shall be assessed as adequate if the following conditions are satisfied:

1. The force-deflection plots of all tests specified in Section 2381 (b) have a *positive incremental force-carrying capacity.*

2. For each increment of test displacement specified in Section 2381 (b) 3 B and for each *vertical load case specified in Section 2381 (b) 3:*

A. There is no greater than a 10 percent difference between the effective stiffness at each of the *three cycles of test and the average value of effective stiffness* for each test specimen.

B. There is no greater than a 10 percent difference in the average value of effective stiffness of the two test specimens of a common type and size of the isolator unit over the required three cycles of test.

3. For each specimen there is no greater than a ± 20 percent change in the initial effective stiffness of each test specimen over the $15S_l/B$, but not less than 10, cycles of test specified in Section 2381 (b) 3 C.

4. For each specimen there is no greater than a 20 percent decrease in the initial effective damping over for the $15S_l/B$, but not less than 10, cycles of test specified in Section 2381 (b) 3 C.

5. All *specimens of vertical load-carrying* elements of the isolation system remain stable at the total maximum displacement for static load as prescribed in Section 2381 (b) 6.

(e) **Design Properties of the Isolation System. 1. Effective stiffness.** The minimum and maximum effective stiffnesses of the isolation system shall be determined as follows:

A. The value of k_{min} shall be based on the minimum effective stiffnesses of individual isolator units as established by the cyclic tests of Section 2381 (b) 3 B at a displacement amplitude equal to the design displacement.

B. The value of k_{max} shall be based on the maximum effective stiffnesses of individual isolator units as established by the cyclic tests of Section 2381 (b) 3 B at a displacement *amplitude equal to the design displacement.*

C. For isolator units that are found by the tests of Section 2381 (b) 3, 4 and 5 to have force-deflection characteristics which vary with vertical load, rate of loading or bilateral load, respectively, the value of k_{max} shall be increased and the value of k_{min} shall be decreased, as necessary, to bound the effects of measured variation in effective stiffness.

2. **Effective damping.** The effective damping (β) of the isolation system shall be calculated as:

$$\beta = \frac{1}{2\pi}\left[\frac{\text{Total Area}}{k_{max}D^2}\right] \tag{81-2}$$

where the total area shall be taken as the sum of the areas of the hysteresis loops of all isolator units and the hysteresis loop area of each isolator unit shall be taken as the minimum area of the three hysteresis loops established by the cyclic tests of Section 2381 (b) 3 B at a displacement amplitude equal to the design displacement.

(Sections 2382 through 2389 are reserved.)

TABLE NO. A-23-U
SITE COEFFICIENTS FOR SEISMIC-ISOLATED STRUCTURES

SOIL PROFILE TYPE[1]	S_i FACTOR
S_1	1.0
S_2	1.5
S_3	2.0
S_4	2.7

[1]See Table No. 23-J for a description of soil profile types and related requirements.

TABLE NO. A-23-V
NEAR-FIELD RESPONSE COEFFICIENT, N

	CLOSEST DISTANCE, d_F, TO AN ACTIVE FAULT[1,2]		
	$d_F > 15$km	$d_F = 10$km	$d_F < 5$km
N	1.0	1.2	1.5

[1]Active faults shall be established from properly substantiated geotechnical data (e.g., most recent mapping of active faults by the United States Geological Survey or the California Division of Mines and Geology).
[2]The near-field response coefficient shall be based on linear interpolation for values of the closest distance, d_F, to an active fault other than those given.

TABLE NO. A-23-W
DAMPING COEFFICIENT, β

	EFFECTIVE DAMPING (Percentage of Critical)[1,2]						
	<2%	5%	10%	20%	30%	40%	>50%
β	0.8	1.0	1.2	1.5	1.7	1.9	2.0

[1]The damping coefficient shall be based on the effective damping of the isolation system determined in accordance with the requirements of Section 2381 (e).
[2]The damping coefficient shall be based on linear interpolation for effective damping values other than those given.

TABLE NO. A-23-X
STRUCTURAL SYSTEMS ABOVE THE ISOLATION INTERFACE

BASIC STRUCTURAL SYSTEM[1]	LATERAL LOAD-RESISTING SYSTEM—DESCRIPTION	R_{wi}[5]	H[2]
A. Bearing wall system	1. Light-framed walls with shear panels		
	a. Plywood walls for structures three stories or less	2.6	65
	b. All other light-framed walls	2.2	65
	2. Shear walls		
	a. Concrete	2.6	160
	b. Masonry	2.6	160
	3. Light-steel-framed bearing walls with tension-only bracing	1.6	65
	4. Braced frames where bracing carries gravity loads		
	a. Steel	1.8	160
	b. Concrete[3]	1.5	—
	c. Heavy timber	1.5	65
B. Building frame system	1. Steel eccentric-braced frame (EBF)	3.0	240
	2. Light-framed walls with shear panels		
	a. Plywood walls for structures three stories or less	2.6	65
	b. All other light-framed walls	2.2	65
	3. Shear walls		
	a. Concrete	3.0	240
	b. Masonry	3.0	160
	4. Concentric-braced frames		
	a. Steel	2.2	160
	b. Concrete[3]	2.6	—
	c. Heavy timber	2.6	65
C. Moment-resisting frame system	1. Special moment-resisting frames (SMRF)		
	a. Steel	3.0	N.L.[4]
	b. Concrete	3.0	N.L.
	2. Concrete intermediate moment-resisting frames (IMRF)[6]	2.2	—
	3. Ordinary moment-resisting frames		
	a. Steel	1.8	160
	b. Concrete[3]	1.5	—
D. Dual system	1. Shear walls		
	a. Concrete with SMRF	3.0	N.L.
	b. Concrete with concrete IMRF[4,6]	2.6	160
	c. Masonry with SMRF	3.0	160
	d. Masonry with concrete IMRF[3]	2.6	—
	2. Steel EBF with steel SMRF	2.6	N.L.
	3. Concrete-braced frames		
	a. Steel with steel SMRF	2.2	N.L.
	b. Concrete with concrete SMRF[3]	2.2	—
	c. Concrete with concete IMRF[3]	1.8	—
E. Undefined systems	See Section 2333 (i) 2	—	—

[1]Basic structural systems are defined in Section 2333 (f).
[2]H = height limit applicable to Seismic Zones Nos. 3 and 4. See Section 2333 (g) for exceptions.
[3]Prohibited in Seismic Zones Nos. 3 and 4.
[4]N.L. = No limit.
[5]See Section 2334 (c) for combination of structural system.
[6]Prohibited in Seismic Zones Nos. 3 and 4, except as permitted in Section 2338 (b).

SCALING

Scale Factor: $L = 1/2.5$ (geometric)

Scaling of different quantities:

Parameter	Prototype / Model
length	L
time	$\sqrt{L}$
mass	L^2
displacement	L
velocity	$\sqrt{L}$
acceleration	1
stress	1
strain	1
force	L^2
area	L^2
moment of inertia	L^4

DESIGN PROBLEM - SCALING

$\text{Length}_p = 2.5 \times \text{Length}_m$

implication : bearing displacements

$\text{Period}_p = \sqrt{2.5} \times \text{Period}_m$

SEAOC/UBC design displacement equation transforms :

prototype : $D_p = \dfrac{10\,NZST_p}{B}$

model : $D_M = \dfrac{10}{\sqrt{2.5}} \dfrac{NZST_m}{B}$

$= \dfrac{6.32\,NZST_m}{B}$

SIMPLIFYING ASSUMPTIONS FOR DESIGN PROBLEM

$N = 1.0$
$S = 1.0$

HDR System

G - will be given for Bridgestone KL301 compound
$B = 1.2$ (ie. $\xi = 10\%$)
$\gamma_{max} \leq 200\%$
$p \approx 500$ psi
$T = 1.26 - 1.58$ sec (or 2 - 2.5 sec for prototype)

Lead-Rubber System

$G = 85$ psi (constant)
lead plug dia. $= \frac{1}{6} \times$ bearing dia.
$B = 1.2$
$\gamma_{max} \leq 125\%$
$p \approx 750$ psi
$T = 1.26 - 1.58$ sec

Friction System

$\mu = 0.05 - 0.20$
$T = 1.26 - 1.90$ sec (or 2 - 3 sec for prototype)
$B = 1.2$

SUGGESTED DESIGN SEQUENCE

Given : N, S

Select : Z

: B

: T (remember - this should be scaled × $\frac{1}{\sqrt{2.5}}$)

Calculate : $D = \frac{10\,NZST}{\sqrt{2.5}\;B}$ (D_T, D_{TM})

Select : $\gamma_m \Rightarrow t_r$

: G

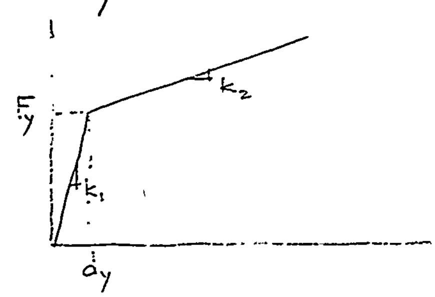
: $p \Rightarrow A$

Calculate : $k = \frac{GA}{t_r}$, $T = 2\pi\sqrt{\frac{M}{K}}$

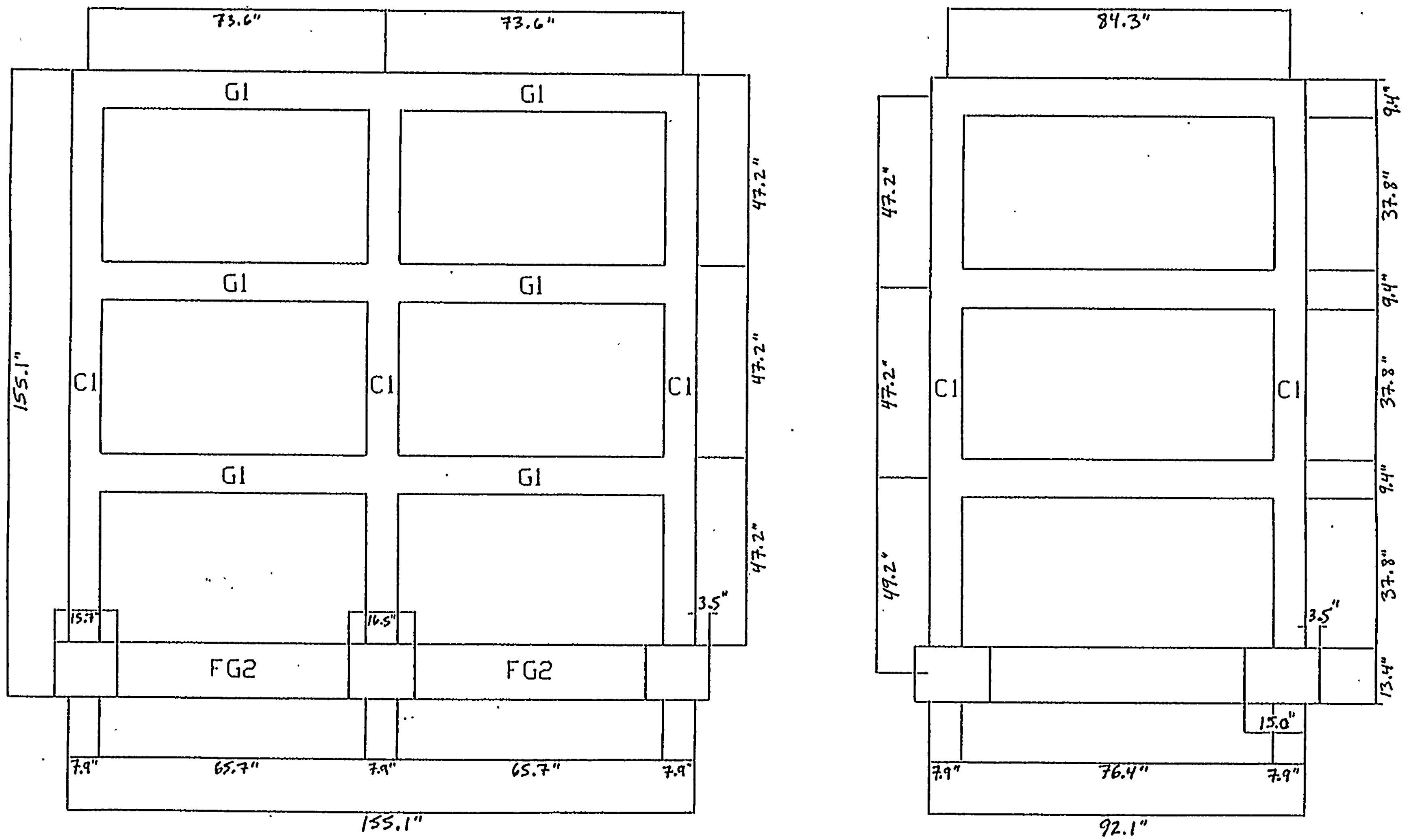
Check calculated T against design T

If necessary, modify p (and hence A) and/or t_r, re-check

For the HDR and Lead-Rubber systems, 3D-BASIS requires that you define

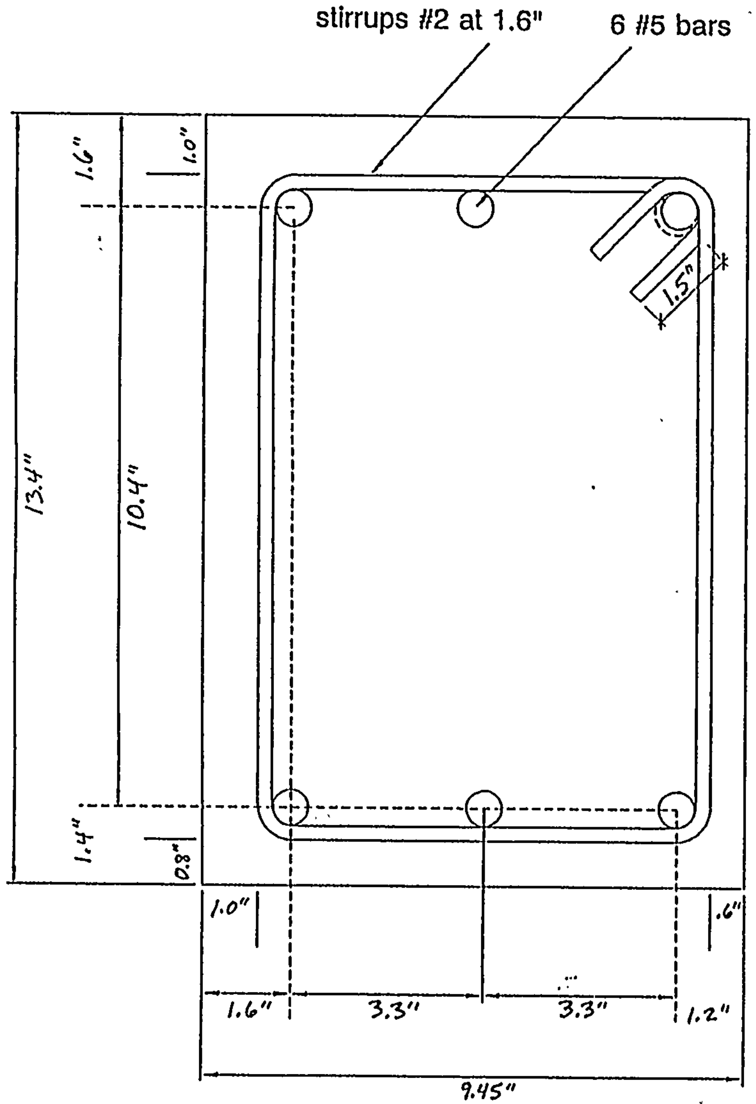


by specifying d_y, F_y, and $\alpha = \frac{k_2}{k_1}$

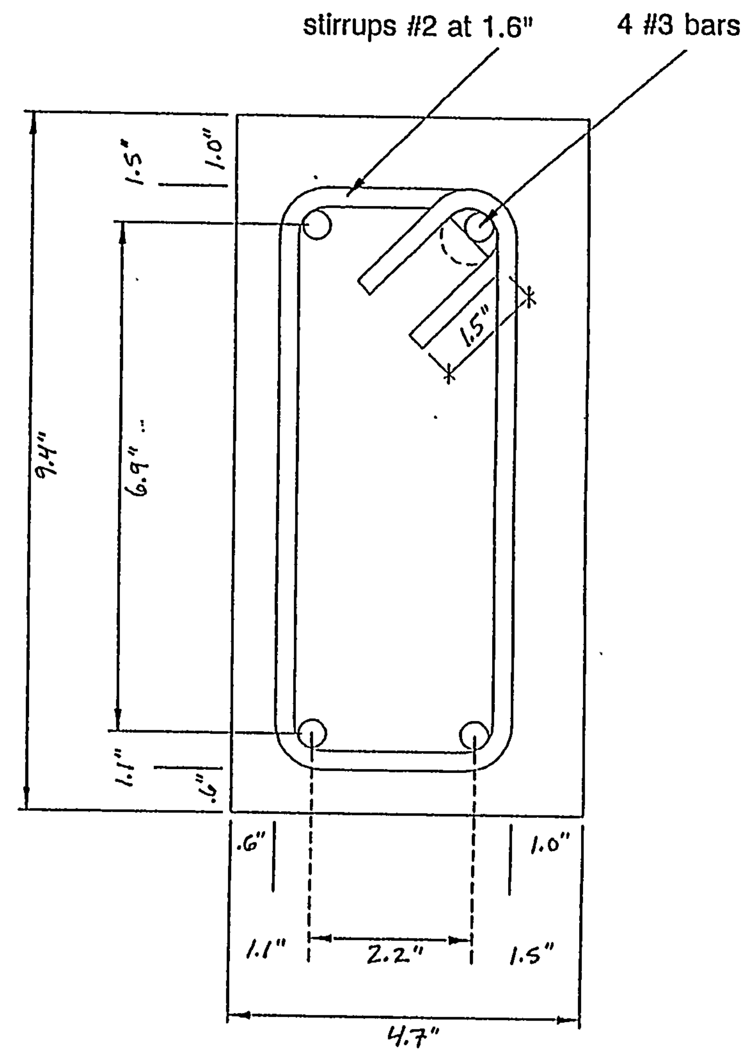


Transverse and Longitudinal Sections of Test Structure

(all dimensions in inches)

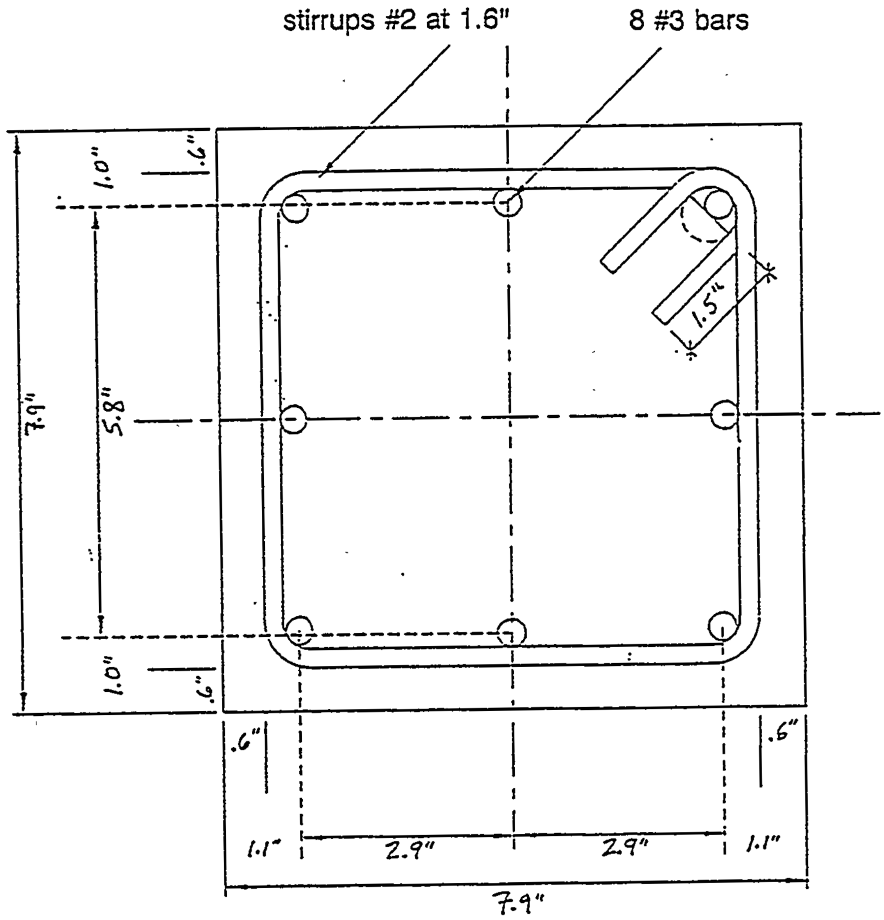


Floor Girder 2 (FG2)



Girder 1 (G1)

Cross Sections of Longitudinal Beam Elements
(all dimensions in inches)



Cross Section of Columns

MATERIAL PROPERTIES

The materials used in the construction of the model were selected to match as closely as possible those in the actual building.

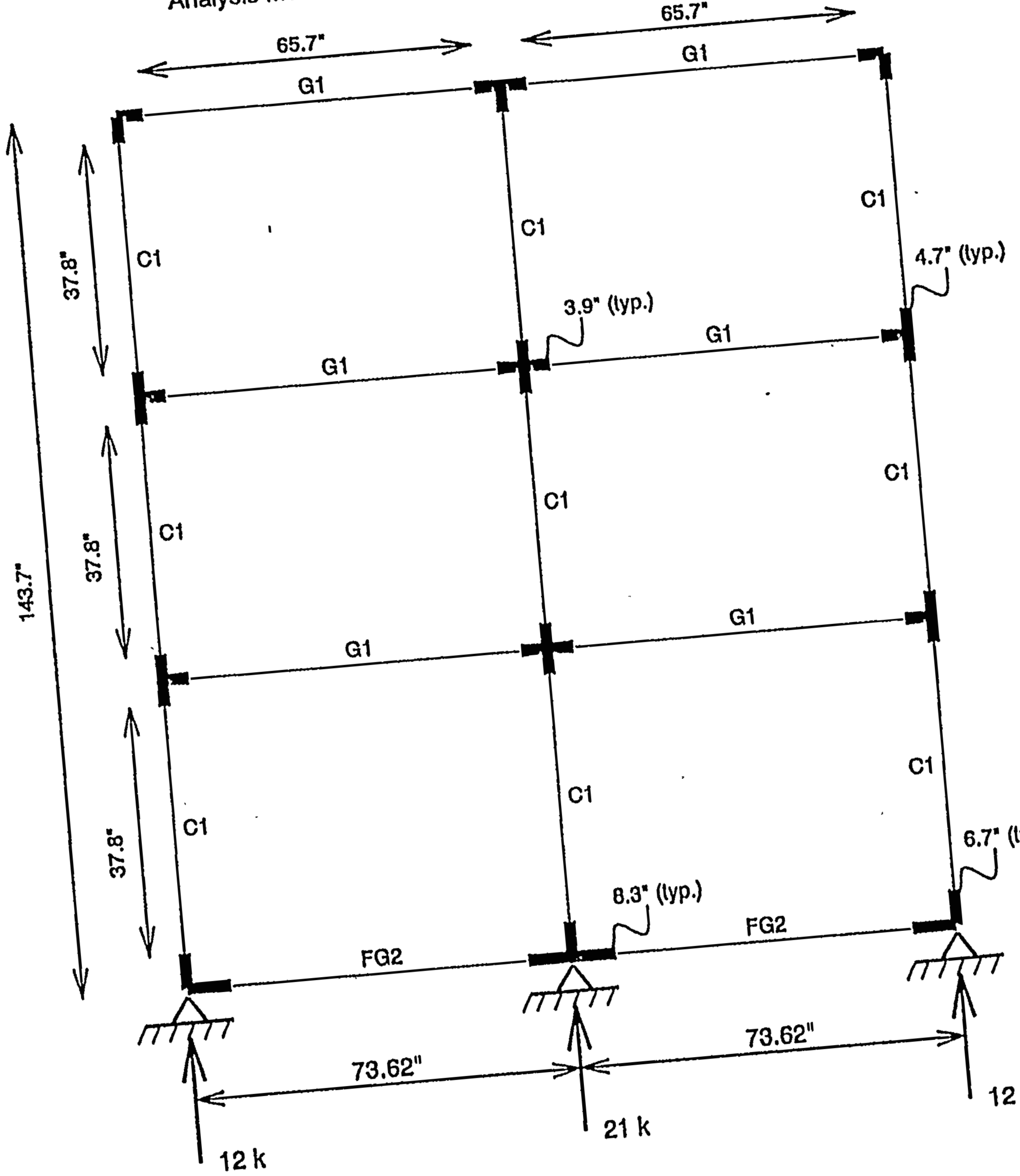
CONCRETE

- Assume $f'_c = 3600$ psi $\rightarrow$ $f_r = 7.5\sqrt{f'_c} = 450$ psi
- For E, use ACI 8.5.1: $E = 57{,}000\sqrt{f'_c}$ for normal-weight concrete.
  $\rightarrow E_c = 3.42\times10^6$ psi

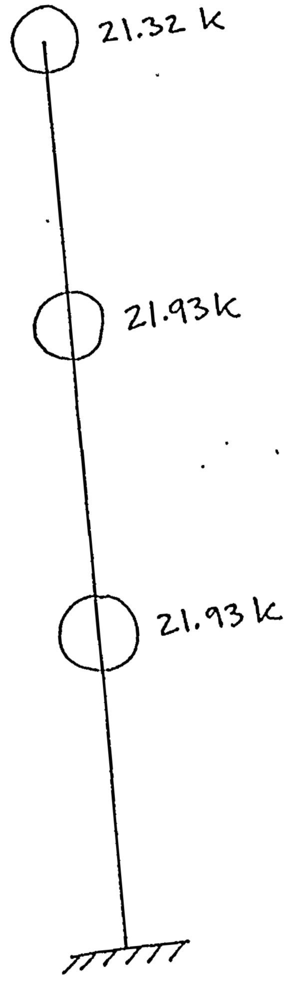
STEEL

- Assume $f_y = 50{,}000$ psi $\rightarrow$ 50 ksi steel
- Assume $E_c = 29{,}000$ksi

Analysis Model with Rigid-End Offsets and Column Loads

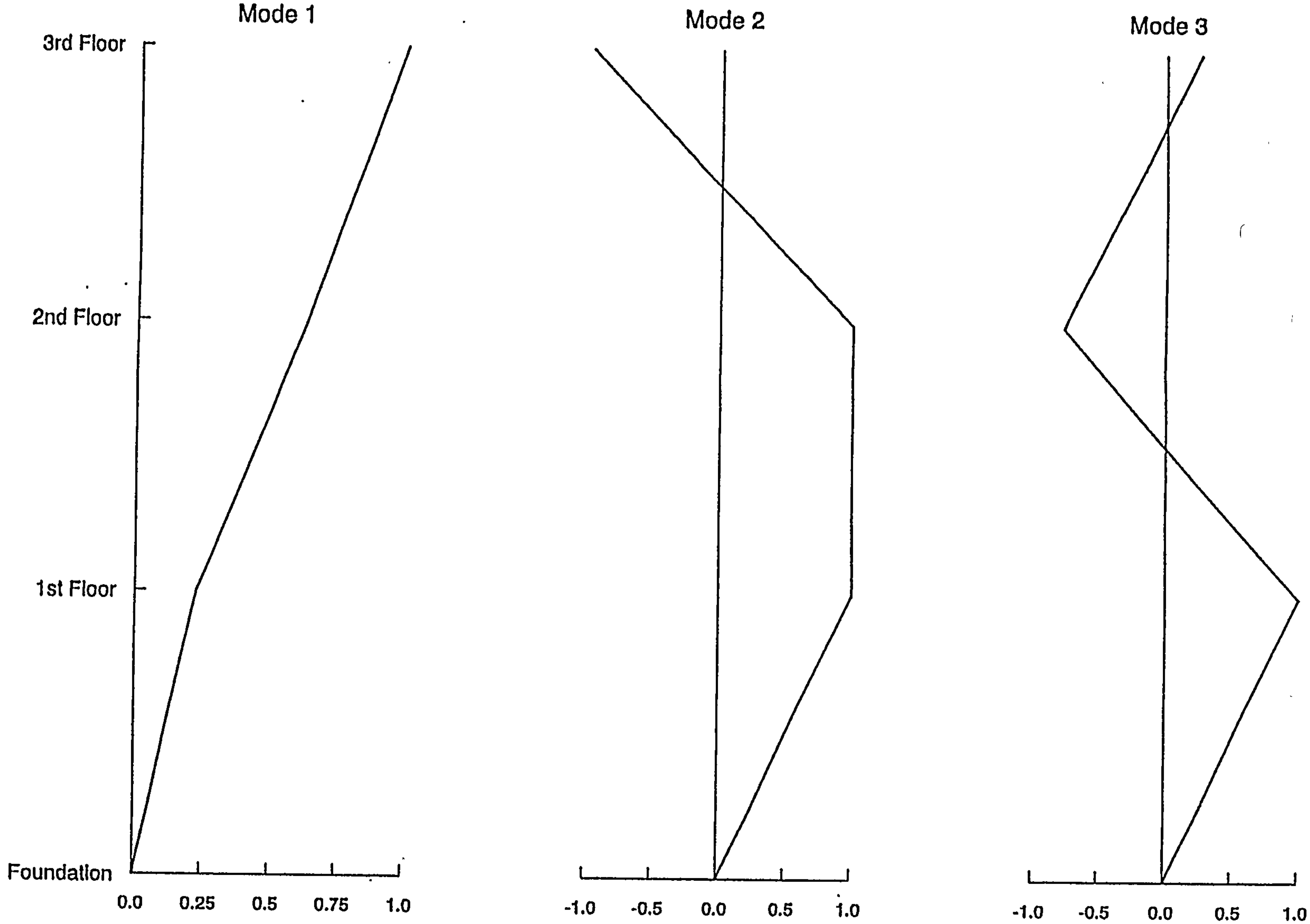


Floor Weights used
for Eigenvalue Calculation



(base weight is 24.83k)

Mode Shapes from Eigenvalue Analysis



$\omega_1^2 = 868.38$ (rad/sec)^2
$f_1 = 4.69$ Hz
$T_1 = 0.21$ seconds

$\omega_2^2 = 13721$ (rad/sec)^2
$f_2 = 18.64$ Hz
$T_2 = 0.054$ seconds

$\omega_3^2 = 62270$ (rad/sec)^2
$f_3 = 39.72$ Hz
$T_3 = 0.025$ seconds

GROUND MOTION

El Centro 1940 modified to be SI spectrum compatible

$$Z \ (\text{or N.Z.S}) = \text{EPA}$$

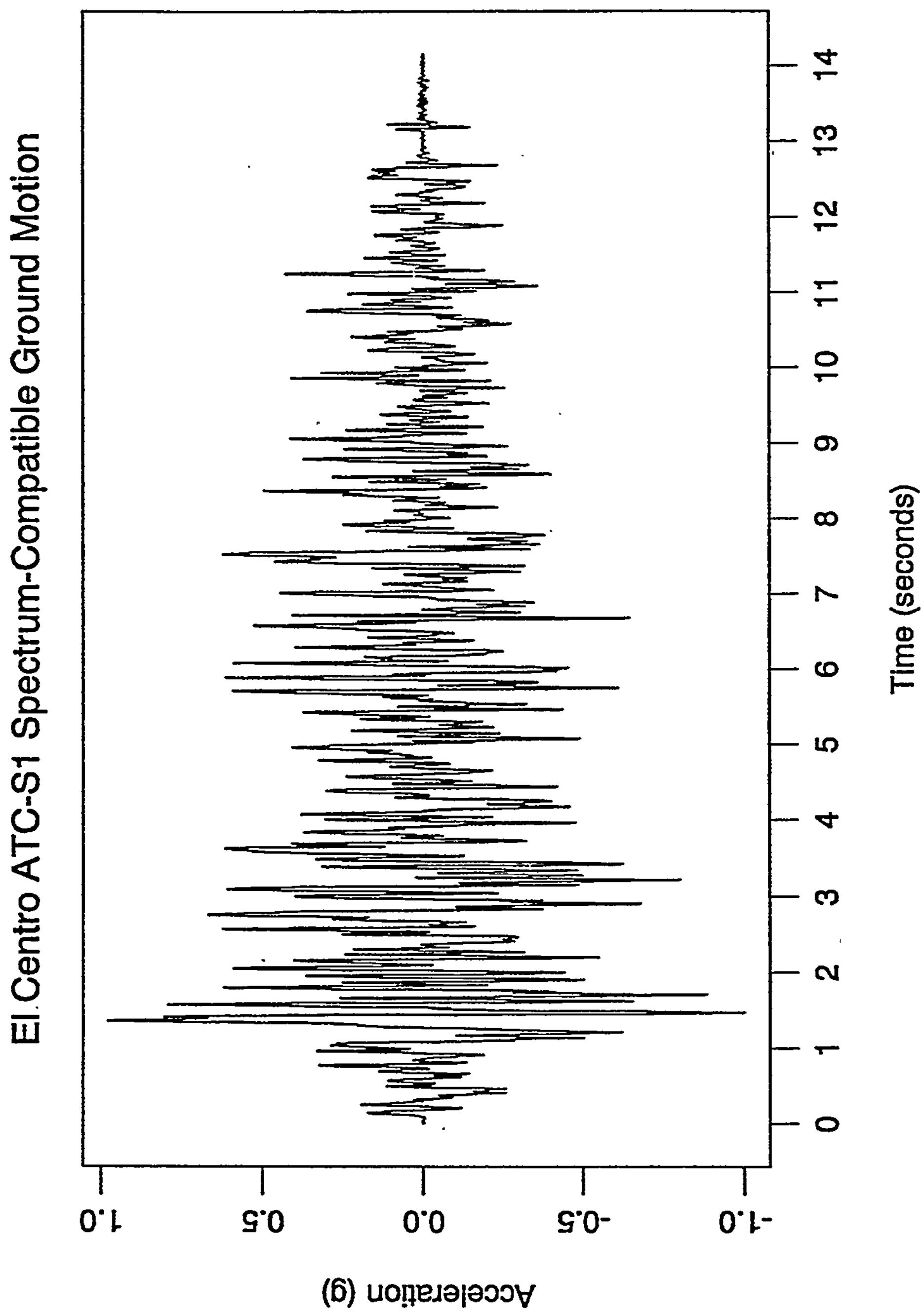
EPA := effective peak acceleration
is a mean over 0.1 – 0.5 second range of
the 5%-damped acceleration response spectrum

assumption : MCE = 1.5 × DBE

			3D-BASIS load factor
Zone 3	DBE :	EPA = 0.3	115.8
	MCE :	EPA = 0.45	144.75
Zone 4	DBE :	EPA = 0.4	154.4
	MCE :	EPA = 0.6	231.6

Ground motion data-file is normalized such that
the PGA = 1 g

Scale (load) factors to scale the normalized signal
to the appropriate level for analysis are shown above



El Centro ATC-S1 Spectrum-Compatible Ground Motion
Acceleration (g)
1.0
0.5
0.0
-0.5
-1.0
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
Time (seconds)

NATIONAL CENTER FOR EARTHQUAKE
ENGINEERING RESEARCH
State University of New York at Buffalo

3D-BASIS
Nonlinear Dynamic Analysis of Three-Dimensional Base Isolated Structures: Part II

by

S. Nagarajaiah, A. M. Reinhorn and M. C. Constantinou
Department of Civil Engineering
State University of New York at Buffalo
Buffalo, New York 14260

Technical Report NCEER-91-0005

February 28, 1991

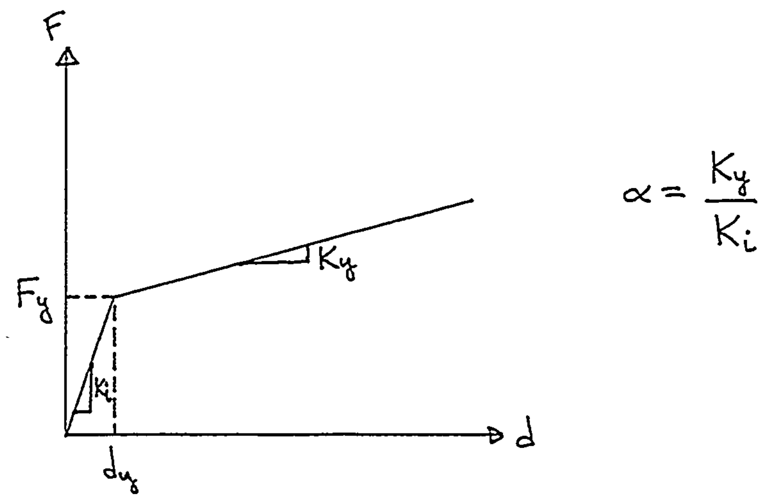
This research was conducted at the State University of New York at Buffalo and was partially supported by the National Science Foundation under Grant No. ECE 86-07591.

3D-BASIS (NCEER/SUNY-Buffalo)

- Computer program developed specifically for analyzing three-dimensional response of base-isolated structures
- Models bidirectional behavior of both elastomeric and sliding bearings
- Individual isolators are modeled explicitly by nonlinear elements
- Superstructure is assumed to remain elastic — building characteristics are entered in one of two forms:
  - — Full 3-D representation (eigenvalues and eigenvectors)
  - — Shear building representation (story masses and stiffnesses)
- Five different isolation elements are available:
  - — Single linear-elastic element with viscous damping representing a complete isolation system
  - — Linear-elastic element (for individual isolators)
  - — Viscous damping element
  - — Hysteretic element for elastomeric bearings, LRB, or certain types of damping devices
  - — Hysteretic element for sliding bearings

Input Information for Isolation Elements

- Hysteretic element for elastomeric and LRB requires post-yield to pre-yield stiffness ratio (α), yield force (F_y), and yield displacement (d_y)



- Hysteretic element for sliding bearings requires maximum coefficient of sliding friction (μ_{max}), difference between the maximum and minimum friction coefficients ($\delta\mu$), a constant controlling the transition between maximum and minimum friction coefficients ($a \approx 0.9$), the yield (or breakaway) displacement (d_y), and the axial force on the bearing (F_N)

```
Tohoku University Isolated Building - Reduced Scale
units: kips-inches
2       3       6       3
0.01    0.001   2000    20      1
0.5     0.25
1       0.005    2831    0       107.77
868.38
13721.1
62269.84
3.510 0 0 2.232 0 0 0.80791 0 0
-2.2447 0 0 2.5160 0 0 2.5256 0 0
0.87144 0 0 -2.5087 0 0 3.251772 0 0
0.0552
0.0568
0.0568
100000
100000
100000
0.04
0.04
0.04
0       0
0       0
0       0
150.39
103.15
55.91
0.0
0       0       0       0       0
0.0643      100000
0       0       0       0       0
# Delete these comment lines and replace them
# with your isolation system parameters.
# There should be two lines for each of the 6
# isolators under the test building.
-73.62  -42.13
-73.62  42.13
0       -42.13
0       42.13
73.62   -42.13
73.62   42.13
1       5      1 0 0 0
0 0
0 0
0 0
0 0
0 0
0 0
```

LOAD FACTORS

	DBE	MCE
Zone 3	115.8	144.75
Zone 4	154.4	231.6

3D-BASIS - OUTPUT

What to look for ?

Maximum response values of :

- isolator displacement (and shear strain %)
- isolator force
- base shear (and calculate V/W)
- interstory drift (%)

3D-BASIS

INPUT FORMAT FOR ISOLATION ELEMENTS

C.5 Isolation Element Data

(i). Data for NP isolation elements to be given using the elements in C.5.1, C.5.2, C.5.3 and C.5.4.

(ii). The following indices are used to identify the element type in the isolation system. INELEM(NP,2) described below is used in all the subsequent sections and will not be described in the subsequent sections.

INELEM(K,1:2)
= Indices for the isolation element K indicating its type and whether it is a uniaxial or biaxial element.

INELEM(K,1) = 1 for a uniaxial element in the X direction

INELEM(K,1) = 2 for a uniaxial element in the Y direction

INELEM(K,1) = 3 for a biaxial element

INELEM(K,2) = 1 for a linear elastic element

INELEM(K,2) = 2 for a viscous element

INELEM(K,2) = 3 for a hysteretic element
for elastomeric bearing or steel damper

INELEM(K,2) = 4 for a hysteretic element
for sliding bearing

C.5.1 Linear Elastic Element

One card

INELEM(K,1:2) INELEM(K,1) can be either 1,2 or 3
INELEM(K,2) = 1
(Refer to C.5 for further details).

One card

PS(K,1),PS(K,2)

PS(K,1) = Shear stiffness in the X direction for biaxial element or uniaxial element in the X direction (leave blank if the uniaxial element is in the Y direction only).

PS(K,2) = Shear stiffness in the Y direction for biaxial element or uniaxial element in the Y direction (leave blank if the uniaxial element is in the X direction only).

Note: 1. Biaxial element means elastic stiffness in both X and Y direcitons (no interaction between forces in the X and Y direction).

C.5.2 Viscous Element

One card

INELEM(K,1:2) INELEM(K,1) can be either 1,2 or 3
INELEM(K,2) = 2
(Refer to C.5 for further details).

One card

PC(K,1),PC(K,2)

PC(K,1) = Damping coefficient in the X direction for biaxial element or uniaxial element in the X direction (leave blank if the uniaxial element is in the Y direction only).

PC(K,2) = Damping coefficient in the Y direction for biaxial element or uniaxial element in the Y direction (leave blank if the uniaxial element is in the X direction only).

Note: 1. Biaxial element means damping in both X and Y direcitons (no interaction between forces in the X and Y direction).

C.5.3 Hysteretic Element for Elastomeric Bearings/Steel Dampers

One card

INELEM(K,1:2) INELEM(K,1) can be either 1,2 or 3
INELEM(K,2) = 3
(Refer to C.5 for further details).

One card

ALP(K,I),YF(K,I),YD(K,I),I=1,2

ALP(K,1) = Post-to-preyielding
stiffness ratio;
YF(K,1) = Yield force;
YD(K,1) = Yield displacement;
in the X direction
for biaxial element or uniaxial
element in the X direction
(leave blank if the uniaxial element
is in the Y direction only).

ALP(K,2) = Post-to-preyielding
stiffness ratio;
YF(K,2) = Yield force;
YD(K,2) = Yield displacement;
in the Y direction
for biaxial element or uniaxial
element in the Y direction
(leave blank if the uniaxial element
is in the X direction only).

C.5.4 Hysteretic Element for Sliding Bearings

One card

INELEM(K,1:2) INELEM(K,1) can be either 1,2 or 3
INELEM(K,2) = 4
(Refer to C.5 for further details).

One card

(FMAX(K,I),DF(K,I),PA(K,I),YD(K,I),I=1,2),FN(K)

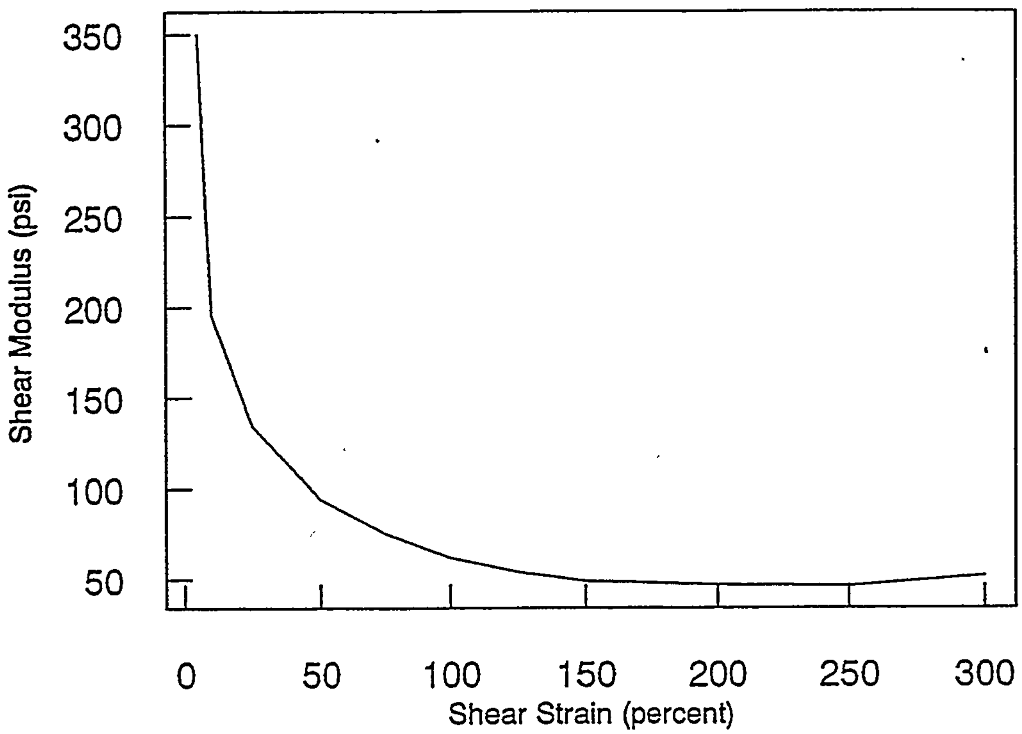
FMAX(K,1) = Maximum coefficient
of sliding friction;
DF(K,1) = Difference between

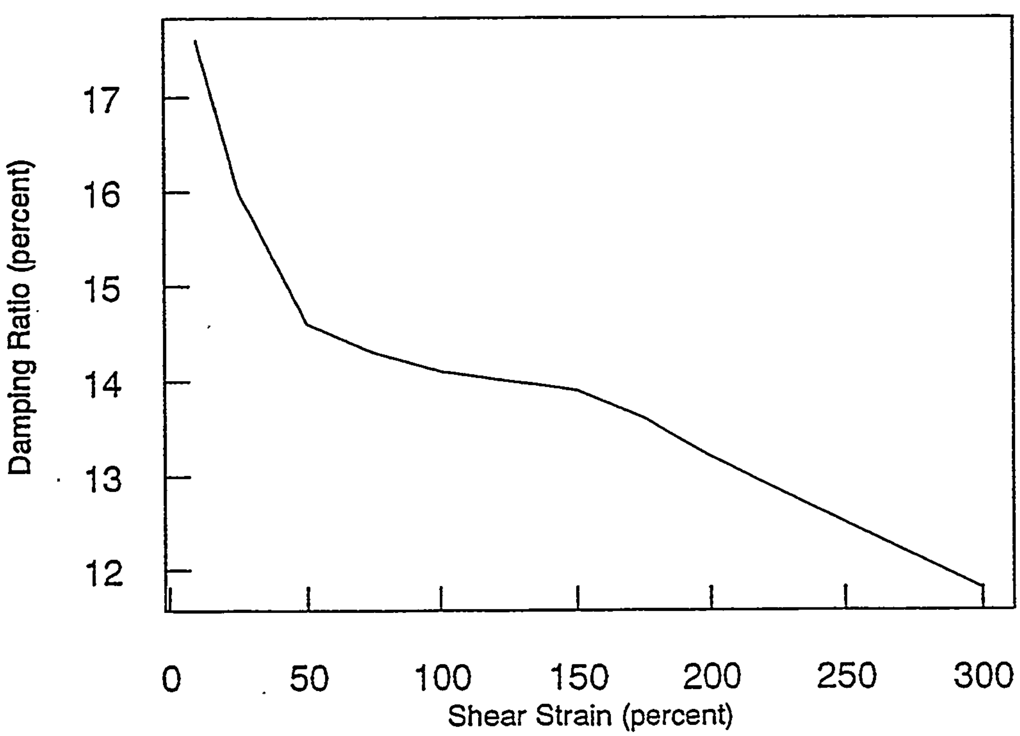
the maximum and minimum
coefficient of sliding friction;
PA(K,1) = Constant which controls the
transition of coefficient of sliding
friction from maximum to minimum value;
in the X direction
for biaxial element or uniaxial
element in the X direction
(leave blank if the uniaxial element
is in the Y direction only).

FMAX(K,2) = Maximum coefficient
of sliding friction;
DF(K,2) = Difference between
the maximum and minimum
coefficient of sliding friction;
PA(K,2) = Constant which controls the
transition of coefficient of sliding
friction from maximum to minimum value;
in the Y direction
for biaxial element or uniaxial
element in the Y direction
(leave blank if the uniaxial element
is in the X direction only).

FN(K) = Initial normal force at the
sliding interface.

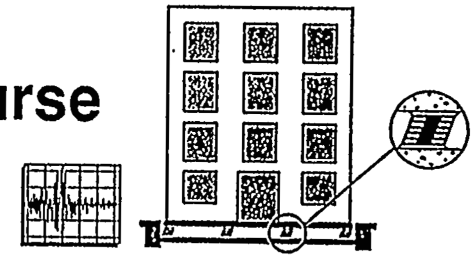
Damping and Shear Modulus Relationships for KL301





NSSP
Nuclear Systems Safety Program

Department of Energy Short Course on Seismic Base Isolation



FUTURE DIRECTIONS IN THE DOE

Stanley C. Sommer
Lawrence Livermore National Laboratory
Nuclear Systems Safety Program

August 14, 1992
Berkeley Marina Marriott
Berkeley, California

The DOE has been Studying the Role of Seismic Base Isolation for its Many Facilities.

- Argonne National Laboratory (ANL)

  Extensive study for liquid metal-cooled reactor (LMR)
  Program for DOE New Production Reactors (NPRs)
  Elastomer Testing Facility (ETF) for rubber samples
  Joint efforts with Shimizu Corporation of Japan

- Energy Technology Engineering Center (ETEC)

  Tests of elastomer bearings in support of ANL

- Lawrence Livermore National Laboratory (LLNL)

  Workshop on Seismic Base Isolation for DOE Facilities
  DOE Short Course on Seismic Base Isolation

Argonne National Laboratory has been Deeply Involved in Seismic Isolation for over Ten Years.

- Development of computer codes for isolators and isolated structures
- Implementation of test program for isolator bearings
- Testing of elastomer compounds for bearings
- Studying effects of long period ground motions
- Determination of effects of aging, temperature, and other environmental factors on isolation systems
- Development of nuclear-grade design and fabrication specifications for isolators

Several Reactor Designs have Considered Seismic Base Isolation.

- Advanced Liquid Metal-Cooled Reactors

  Thin-walled construction
  - Higher thermal performance and lower cost
  - Difficult to resist earthquake loads

  Standardization was primary concern
  Current reference design uses seismic isolation

- DOE New Production Reactors

  Important and unique facility
  Assure continuity of operation
  - Minimum downtime after major seismic event
  - Return to full operation at reasonable cost

SCS DOE Base Isolation Short Course August 14,1992 Future Frame 4 of 10

ANL Performed Tests of Bearing Designs at the EERC.

- Tests have Several Purposes

  Study performance of bearing properties
  Compare performance of fixed-base versus isolated
  Study effects of real earthquakes on isolated structure

- Several Types of Bearings were Tested

  High shear modulus, high damping bearings
  - Same shape factor as for GE PRISM design

  Low shear modulus, high damping bearings
  - Try to improve response for small earthquakes

  Bridgestone high damping bearings
  - Low shear modulus
  - High shear modulus

Technology Base Started at ANL is being Expanded throughout the DOE.

- Technology Transfer to Increase Knowledge Base

  Workshop on Seismic Base Isolation for DOE Facilities
  DOE Short Course on Seismic Base Isolation

- Main Topics

  Concepts, issues, and applications of isolation
  Engineering details and characteristics of systems
  Design process involving base isolation

DOE Community Needs Guidance Concerning the Use of Seismic Base Isolation.

- **Utilize Results of ANL Efforts**

  Detailed testing procedures and requirements
  Tools for evaluating isolation systems and structures

- **Prepare Guidance Similar to that Done by US Navy**

  <u>Issues Related to the Selection of U.S. Navy Buildings for Base Isolation</u> - Gary Hart
  <u>Seismic Design Criteria for Base Isolated U.S. Navy Essential Buildings</u> - Gary Hart and Rami Elhassan

- **Develop Policy and Procedures for Base Isolation**

  Techniques (performance categories) in UCRL-15910

As with any New Technology, Care is Required for Implementing Base Isolation.

- Structural Requirements of Facility
- Cost Concerns
- Performance and Functionality Requirements
- Level of Risk
- Advantages and Disadvantages of Different Systems
- Safety Margins

LLNL/DOE Appreciates your Participation in the Short Course.

- Feedback for LLNL

  Short Course evaluation form
  Role of seismic base isolation in the DOE

- Materials

  Short Course binder

The Short Course will Conclude with a Panel Discussion about the Future of Base Isolation.

- Panel Members

James R. Hill -- Future in the Department of Energy
James M. Kelly -- Future in Research and Testing
Charles A. Kircher -- Future in Code Design

- Possible Topics of Discussion

Role of base isolation in the DOE for the next 5 years
Amplification in building with low seismic input
Lack of recorded performance of buildings at high seismic input
Accuracy of hysteresis loops in predicting damping