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FINAL REPORT ON THE INTERNATIONAL WORKSHOP ON THE USE OF RUBBER-BASED BEARINGS FOR THE EARTHQUAKE PROTECTION OF BUILDINGS

by

JAMES M. KELLY

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This workshop on the use of rubber-based bearings for the base isolation of buildings was held in Shantou City, Guangdong Province in the People's Republic of China during May 17-19, 1994. The report summarizes the workshop and includes six reports submitted by participants.

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Final Report on the International Workshop on the Use of Rubber-Based Bearings for the Earthquake Protection of Buildings

Shantou City
Guangdong Province
P.R. China
May 17-19, 1994

by

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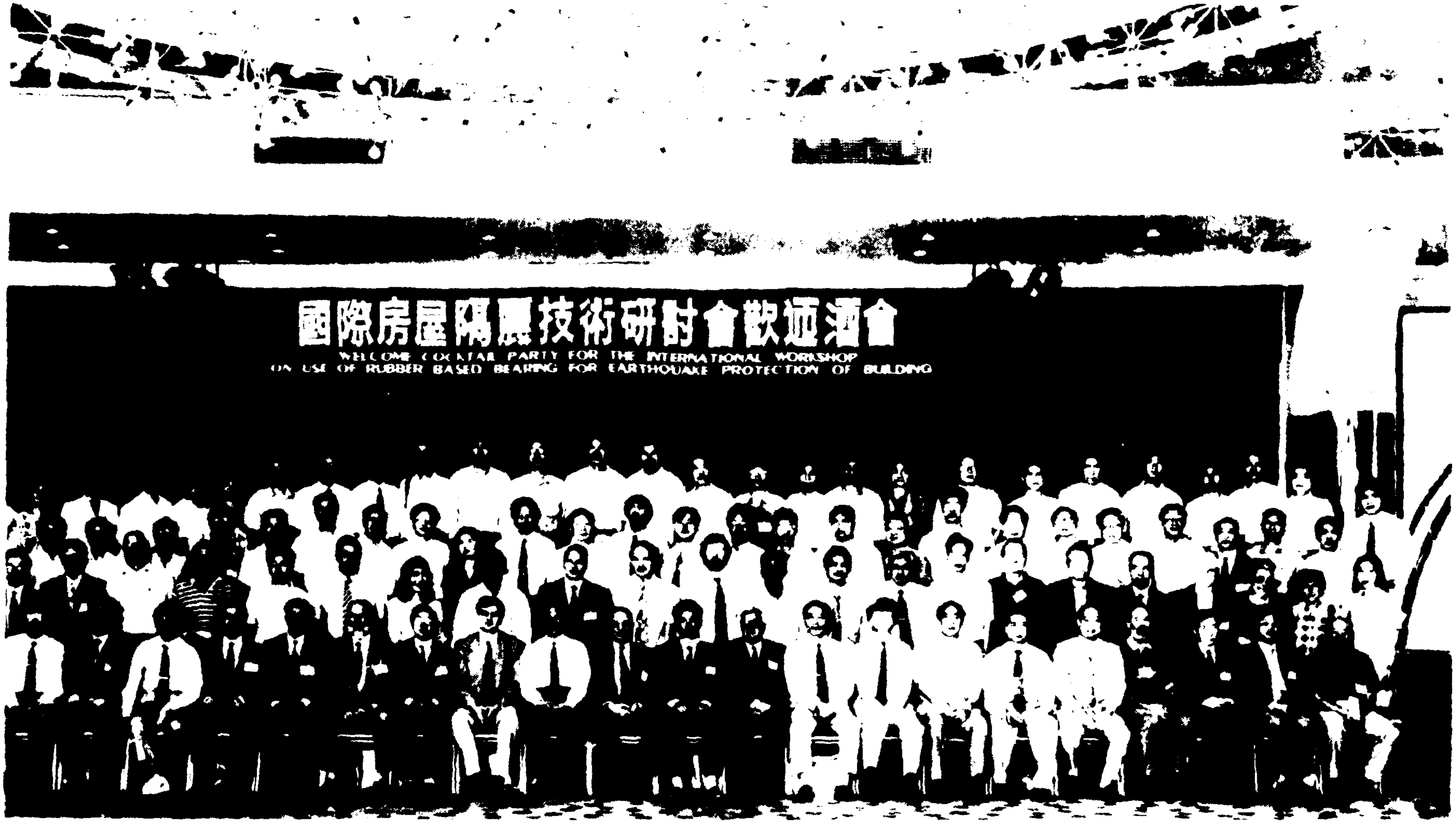

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FOR MEMENTO: REPRESENTATIVES OF IWADBI MAY 1994 SHANTOU CHINA

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Table of Contents

List of Figures

1. Introduction

The concept of base isolation as an innovative means of creating earthquake-resistant structural systems was met initially with a great deal of skepticism by the engineering community. Today, however, it is on the cutting edge of seismic resistance engineering, as evidenced by the rapidly increasing number of buildings, both for new construction and retrofit, using this technique. It is now generally accepted that a base-isolated building's performance will be superior to a conventional fixed-base building in moderate or strong earthquakes. In the structures in which it has been used to date, the major benefits have been to reduce the effects of seismic forces on contents and internal equipment, more than justifying the increased cost of isolated construction.

A basic problem in designing earthquake-resistant low- to medium-rise structures is that their fundamental frequency of vibration is in the range of frequencies where the earthquake energy is the strongest. This means that the building acts as an amplifier of the ground vibrations, with the floor accelerations increasing over the height of the building. These amplified accelerations cause stresses in the frame and interstory drifts, which may damage the columns between floors. In addition, the amplified accelerations at each floor act on the contents and occupants of the floor and can cause severe damage to these contents even when no damage occurs to the structure itself.

The goal of the seismic design should be to reduce the accelerations in buildings to below the level of the ground accelerations. To do this the building must be flexible. Flexibility in a structural frame may cause windows to fall out due to wind loads, partition walls to crack, and floors to vibrate under foot. For a low- or medium-rise building, the necessary flexibility can only be achieved by using base isolation at the foundation level.

Developments in rubber technology over the past twenty-five years have made the idea of base isolation a practical reality. Rubber bearings are now used almost everywhere as thermal expansion bearings for bridges. The mechanical roller or rocker bearings that had been used previously had a number of problems associated with them: they were vulnerable to attacks by salt, had a tendency to lock-up, and performed badly in earthquakes. The rubber bearings that replaced them are inexpensive, durable, and reliable, some now having been in service for over twenty-five years. Many buildings in Europe and the United Kingdom have been built on rubber bearings to isolate them from vibrations from underground railways, and these bearings have performed well over substantial periods of time.

Bearings used to isolate structures from earthquake loads were patterned directly after these thermal expansion bearings. Both types of bearings are manufactured in the same way, the only differences between them are in the proportions of rubber to steel and the deformations for which they are designed.

Rubber bearings that are used in anti-seismic applications offer the simplest method of isolation. Relatively easy to manufacture, isolation bearings are made by vulcanization bonding of sheets of rubber to thin steel reinforcing plates. The bearings are very stiff in the vertical direction, but very flexible in the horizontal direction. Under seismic loading, they act to isolate the building from the horizontal components of the earthquake ground movement; in addition, they isolate the building from high-frequency vertical vibrations that are produced by underground railways and local traffic.

the effectiveness of the isolation system.

Most base-isolated superstructures in Japan are reinforced concrete and up to six stories in height. There are six buildings above six stories with composite steel-reinforced concrete structural systems and there are three wooden-frame structure applications. Roughly half of the buildings approved use natural rubber (low damping) isolators with additional damping components such as steel bars, external lead bars, or frictional elements. Lead-rubber bearings and high-damping rubber bearings constitute most of the remaining systems. There are three applications of sliding systems (PTFE and stainless steel sliders), with laminated neoprene rubber springs to provide restoring forces. To date, all rubber isolators used in Japan have been circular, most about 400 to 600 mm in diameter; the smallest isolators in use are 200 mm bearings under a two-story wooden house and the largest are 1.5 m in diameter. The Building Center of Japan design requirements tend to lead to isolation periods (at large displacements) that cluster around 2.5 sec, although there are four examples of buildings with periods greater than 3.0 sec.

One of the largest base-isolated buildings in Japan is the C-1 building in Fuchu City, near Tokyo. The building is a computer center for an insurance company; it is seven stories tall with a penthouse and has a total floor area of 37,846 m^2. The superstructure has a composite structural system and the isolation systems consists of lead-plug bearings with diameters that range from 1100 mm to 1500 mm. The isolators are laid out on a 15 m^2 grid, resulting in large isolator loads that range from 500 tons to 1600 tons. The isolators were manufactured by Bridgestone Rubber Corporation for Oiles Corporation, which inserted the lead plugs and provided the isolators to the construction contractor, Shimizu Corporation.

The design was based on the standard 25 cm/sec and 50 cm/sec ground motions, but an additional check of the performance at a 75 cm/sec velocity ground motion input was carried out to verify the characteristics of the system under extreme earthquake loading. The natural period of the system at the second level of input was 3.0 sec. In contrast with the usual Japanese practice, the elastomer strain at level 2 input (50 cm/sec) is only 100%, corresponding to a horizontal displacement of 24 cm.

Another interesting Japanese base isolation project is a complex of three buildings in Nagoya City for the Chubu Electric Power Company. An administrative center for Chubu Electric, the center consists of three structurally-independent buildings. The central building is a conventional fixed-base structure. The two flanking buildings are identical six-story composite structures with a floor area of 6800 m^2. Both buildings are base isolated with the east building, which was constructed by Kajima Corporation, using lead-rubber bearings, and the west building, a project of Shimizu Corporation, using high-damping rubber bearings. The design requirements for the two base-isolated buildings were identical.

As is usual Japanese practice, the systems were designed for two levels of seismic input. The horizontal displacement for both designs is around 30 cm at the higher level, and both systems provide a period of around 2.5 sec. When the shear strain in the rubber exceeds 0.5, the effective stiffness and effective damping of both types of isolators are the same. The damping factor is around 15%. Three sizes of each type of isolator were used, corresponding to bearing loads of 300, 450 and 600 tons. The lead-rubber bearings are 800, 1000 and 1100 mm in diameter with 240 mm total thickness of rubber, and the high-damping rubber bearings are 750, 900 and 1000 mm in diameter with 202 mm total thickness of rubber. It is planned to study and compare the response of the buildings with these two different types of isolators to earthquake excitation.

The largest base-isolated building in the world at the present time is the West Japan Postal Computer Center, which is located in Sanda, Kobe Prefecture, Japan. It is 47,000 m^2 in floor area, six stories in height, and is supported on 120 elastomeric isolators with a number of additional steel and lead dampers. The isolated period is 3.9 sec. The building is located approximately 30 km from the epicenter of the January 17, 1995 Hyogo-ken-Nambu (Kobe) earthquake and experienced severe ground motion. The peak ground acceleration was 400 cm/sec^2 (under the isolators) and was reduced by the isolation system to 127 cm/sec^2 at the sixth floor. The preliminary estimate of the displacement of the isolators is around 20 cm. There was no damage to the isolated building, but a fixed-base building adjacent to the computer center suffered some damage.

1.2 Base Isolation in New Zealand

The first base-isolated building in New Zealand was the William Clayton building in Wellington [1]. Completed in 1981, it was the first building in the world to be isolated on lead-rubber bearings. Since its completion, three other base-isolated buildings have been built in New Zealand; two of these structures (Union House, Auckland, and Wellington Central Police Station) are isolated using the sleeved-pile approach. The Union House is a twelve-story reinforced concrete braced frame. Displacement control is provided by an additional damping system based on the elastic-plastic deformation of mild steel-tapered plates. The Wellington Central Police Station is a ten-story reinforced concrete braced frame structure and displacement control is effected by lead-extrusion dampers [2]. The National Museum of New Zealand in Wellington, currently under construction, is isolated with 142 lead-rubber bearings and 36 teflon pads under shear walls.

A printing press building in Petone near Wellington, has been built on lead-rubber isolators [3]. The purpose of the isolation system is to protect the printing presses which are very large and brittle pieces of equipment.The presses are made of cast-iron and are equivalent in height to a four-story building. The building structure surrounds and is connected to the press and the entire system is isolated at the base.

A major isolation retrofit project has recently been completed. The New Zealand Parliament House, a masonry bearing wall structure originally completed in 1922, and one other building have been isolated using more than 514 lead-rubber bearings [4].

1.3 Applications of Base Isolation in Italy

Base isolation is being actively studied in Italy under the auspices of the National Working Group on Seismic Isolation, Gruppo de Lavoro Isolamento Sismico (GLIS). GLIS has a wide membership comprised of researchers and practitioners; it has organized several workshops and is preparing design guidelines for isolation systems.

Several buildings have been built in Italy using base isolation. One of these is the new Administration Center of the National Telephone Company (SIP), a complex of five seven-story buildings in Ancona. A second base-isolated building project is under construction in Ancona for the Ministry of Defense. A design for base-isolated standardized prefabricated switch houses, also for SIP, has been developed by Giuliani [5]. A number of these are to be located in highly-seismic areas. A pilot project on the retrofit of a historic building is under construction in the village of Frigento in southern Italy. The simple masonry church of St. Peter is to be restored using high-damping rubber bearings in addition to other structural strengthening [6].

1.4 Demonstration Projects

The emphasis in most base isolation applications up to this time has been on large structures with sensitive or expensive contents, but there is increasing interest in applying this technology to public housing, schools, and hospitals in developing countries where the replacement cost due to earthquake damage could be a significant part of the GNP. Several projects are under way for such applications. The challenge in this context is to develop low-cost isolation systems that can be used in conjunction with vernacular methods of construction, such as masonry block and lightly-reinforced concrete frames. The United Nations Industrial Development Organization (UNIDO) has partially financed a joint effort between the Malaysian Rubber Producers' Research Association (MRPRA) of the United Kingdom and Earthquake Engineering Research Center (EERC) at the University of California at Berkeley to research and promote the use of elastomeric bearings for base-isolated demonstration buildings in developing countries.

To date, a number of base-isolated demonstration projects have been completed, are currently under construction, or are in the planning phase. In most cases, an identical structure of fixed-base construction was built adjacent to the isolated building to compare their seismic behavior during earthquakes. There are completed demonstration projects in Reggio Calabria, Italy, Santiago, Chile, Guangdong Province, P.R. China, and in Pelabuhan Ratu, Indonesia. A feasibility study is currently underway on constructing a demonstration building in Armenia.

The first project to build a public housing facility on a natural rubber isolation system was completed in Italy in 1989. Two buildings, identically constructed except that one is isolated and the other is not, were built in the town of Squillace Marina in Calabria, a highly-seismic province in southern Italy. The buildings are three stories in height with a complete basement. The structural system of both buildings is a reinforced concrete frame. The isolators in the isolated building were placed at the top of the foundation and under the framed structure. The isolators are natural rubber multilayer bearings with diameters of either 400 mm or 500 mm. The design natural period of the isolated building is 1.75 sec, as compared with the period of the fixed-base building that is around 0.2 sec. These buildings were built as a demonstration project with support provided by the Italian government. Some dynamic tests have been carried out on the two buildings [7].

A similar demonstration project was carried out in Santiago, Chile with support from the Chilean research agency FONDECYT and the Ministry of Housing. Two four-story housing units, each containing four apartments, are located in a low-income housing project in Santiago. Completed in 1993, the buildings are identical in construction except that one is isolated. Both buildings have been instrumented by the Department of Civil Engineering at the University of Chile and records of a few small earthquakes have been obtained.

The first stories of both buildings are built with reinforced concrete walls and the upper stories are confined masonry. The floors consist of 10 cm-thick reinforced concrete slabs and the buildings have wooden roofs. The isolated building has eight natural rubber multilayer bearings and a seismic gap of 20 cm is provided around the building. The design natural period of the isolated structure is around 2.0 sec, and the fundamental period of the fixed-base building is around 0.1 sec.

The natural rubber isolators for this project were produced in Santiago. The isolators themselves were relatively inexpensive as they were manufactured locally, but due to the extremely low total cost of the building and the fact that it was not possible to reduce the strength of the structure, the use of isolation resulted in an increase in the total cost of the isolated structure. The total cost, however, is of the order of $10,000 (U.S.) per unit with four units in each building, and thus is still considered low-cost housing.

The UNIDO-sponsored demonstration building in Shantou City, Guangdong Province, P.R. China is the next example of a base-isolated building for public housing. It will be described in more detail in section 4 of this report. It is useful to point out, however, that because of the isolation system, it was possible to reduce the framing of this building as compared to that of the fixed-base companion building, thereby producing a cost savings in the construction cost of the structure. Even when including the cost of the isolators, the construction cost of the isolated building was identical to the fixed-base building. The demonstration building was completed in May 1994 and its opening to the public was the occasion of the International Workshop on the Use of Rubber-Based Buildings for the Earthquake Protection of Buildings.

The latest example of a low-cost demonstration project is a four-story housing block in S.W. Java, Indonesia. The construction of the building was entirely funded by UNIDO through a grant to MRPRA and was completed in October 1994. Located on a tea and rubber estate just outside the coastal community of Pelabuhan Ratu, the building, a reinforced concrete frame with masonry block infill, has eight two-bedroom apartments. The frame is carried on sixteen high-damping natural rubber bearings. There are two types of bearings comprised of two different compounds; however, they are the same size, 330 mm in diameter and 275 mm tall, and the target design period is 2.0 sec.

2. International Workshop on the Use of Rubber-Based Bearings for the Earthquake Protection of Buildings

The completion and dedication of the UNIDO-sponsored demonstration building in Shantou City was the occasion for this workshop held in Shantou City on 17-19 May 1994. The workshop was sponsored by UNIDO, the United Nations Development Program (UNDP), and by the National Natural Science Foundation of China. It was organized by members of the South China Construction University, Guangzhou, P.R. China with the assistance of the Shantou City Government.

An international steering committee was formed comprised of the following members:

J.M. Kelly (U.S.A.), Chairman
M. Youssef (UNIDO), Co-Chairman
T.-C. Pan (Singapore), Co-Chairman
S.F. Stiemer (Canada)
A. Martelli (Italy)
T. Fujita (Japan)
H. Tada (Japan)
W.-S. Cheng (P.R. China)
F.-L. Chu (P.R. China)
X.-Q. Na (P.R. China)
J. Liu (P.R. China)
X.-K. Wang (P.R. China)
L.-L. Xie (P.R. China)
Y.-X. Ye (P.R. China)
B. Zhu (P.R. China)
E. Csorba (UNIDO)
F.-L. Zhou, (P.R. China) Executive Chairman
K.N.G. Fuller (U.K.), Co-Chairman
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M. Sarrazin Arellano (Chile)
H. Akiyama (Japan)
M. Izume (Japan)
W.H. Robinson (New Zealand)
W.-X. Cheng (P.R. China)
J.-J. Jiang (P.R. China)
Q.-Z. Liang (P.R. China)
X.-Q. Qiu (P.R. China)
S.-Y. Wu (P.R. China)
X.-Z. Xin (P.R. China)
X.-Y. Zhou (P.R. China)
A.H. Muhr (U.K.)
S.-C. Liu (U.S.A.)

Listed below are the members of the local organizing committee:

Z.-Z Huang, Chairman
X. Sishi, Co-Chairman
H.-Y. Zhou, Co-Chairman
S.-X. Zeng, Co-Chairman & Secretary General
Y.-M. Chen
W.-H. Huang, Co-Chairman
L. Wei, Co-Chairman
Z.-C. Lie, Co-Chairman
X.-Y. Chang
L.-Z. Cheng
S.-X. Ding

H.-M. Gui	D.-Y. Hao
C.-K. Hua	S. Jia
K. Kun	Y.-L. Liang
C.-H. Lin	C.-J. Lin
W.-M. Lin	Z.-G. Liu
Y.-H. Peng	C-.M. Shang
X-.M. Song	W.-L. Wang
Q.-L. Xian	L.-B. Yu
Z.-G. Yun	L.-X. Zhao
G.-L. Zhang	

The workshop was extremely well attended, attracting participants from many countries. UNIDO sponsored a number of participants from developing countries with high seismicity. In addition to the many participants from all parts of P.R. China, there were participants from Canada, England, many from Italy, Japan, Korea, New Zealand, and Singapore. The participant list is included in this report as Appendix A. The range of papers presented at the workshop was very broad, reflecting the worldwide interest in the development of this new technology.

In addition to the presentations, the Shantou City Government hosted a dedication and reception at the site of the two buildings where it was possible to view the isolators and the accommodations. Shantou City is extremely interested in developing an isolator manufacturing facility and has agreed to provide land and financial support for this facility with the intention of providing natural rubber isolators for base-isolated buildings throughout P.R. China.

While the scope of the workshop initially seemed to be somewhat restricted to those who had some interest in rubber bearing technology, in fact, the workshop proved to be a good opportunity to review progress in base isolation in general, both in practice and research, with more emphasis than usual on practical applications. The discussions after many of the presentations were vigorous and helpful, and generally much more interesting than occurs in a typical workshop. This was an important conference for the Chinese, and while many of their presentations did not seem to be technically at the levels of the others, it was exciting to see a whole new group of academics approaching base isolation as a new subject. It was also extremely important that the U.S. was able to support a significant group of participants: if this group had not been there, their absence would have been noted.

In discussions at the workshop (for example, among the New Zealand, Italian, and U.S. participants) it seems clear that base isolation is still impeded by over-conservative attitudes. For example, in the U.S. the number of bureaucratic mandates (i.e., feasibility studies, peer reviews, plant and site inspections) that an engineer must satisfy in order to isolate a structure make it remarkable that anyone does a base-isolated project. In addition, while base isolation provisions are now in the Uniform Building Code, the requirements are so conservative that the potential advantages of using base isolation (reduced-design requirements in the superstructure) are lost.

As another example of this over-reaching conservatism, a New Zealand presenter explained that every bearing for a new building (the Wellington Museum) had been tested. This may be of great technical interest, but it represents an impediment to the increased application of base isolation technology. As a comparison, supposing that every steel wide flange member had to be tested, where would steel construction be? Unless bearings become a catalog commodity with certified characteristics and allied to reasonably simple design and analysis procedures that promote the benefits of base isolation, this technology will remain difficult to implement and restricted to a few projects a year.

In contrast, the Chinese engineering and research community have used base isolation to isolate ordinary, low-cost housing, as opposed to western base-isolated projects that have been to date expensive, large-scale structures. Additionally, the Chinese have exploited the advantages of base isolation by reducing superstructure costs, so that the total project costs are not increased. Also, they have the opportunity to make their bearings a commodity and to offer them to other developing countries with the potential to market them in the developed world. Because of conservatism in the west, the European, U.S., and Japanese proponents of base isolation have been forced to promote base isolation as an expensive but high-performing system to a very limited set of users.

3. National Science Foundation of the United States Sponsored Participants

The National Science Foundation (NSF) of the United States sponsored a team of participants to this workshop. The members of the team included:

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4. UNIDO Demonstration Buildings in Shantou City

The isolated and non-isolated demonstration buildings in Shantou City are eight stories high and have commercial space at the ground floor level with three three-bedroom apartments on each upper floor. The front elevation and a typical section of the isolated buildings are shown in Figs. 1a, 1b, 2a and 2b. The building height is 2416 m, the ground floor for commercial use is 3.6 m high and the upper stories have a height of 3.0 m. The building is a reinforced concrete frame with masonry infill, 10.3 m × 24.3 m. The beams are 200 mm wide × 500 mm deep and support a 80 mm reinforced concrete slab. The columns are 350 mm wide with a thickness that varies from 550 mm at the first two floors, to 450 mm at the next two floors, and to 350 mm for the top four stories. A photograph of the demonstration building is shown in Fig. 3.

The seismic requirements for Shantou City, based on the Chinese code 9BJ11-89, use a Design Basis Earthquake (DBE) (10% probability of exceedance in 50 years), with a peak ground acceleration of 0.2g. The Maximum Credible Earthquake (MCE), according to 9BJ11-89, is to be taken as twice the DBE.

The target period for the structure is 2.0 sec, and for the 5% damped spectrum developed for the project shown in Fig. 4, the spectral displacement is 140 mm. Assuming that the damping is around 10%, the design displacement for the bearings is taken to be 120 mm. By code the MCE displacement is 240 mm, but when the nonlinearity of the isolator compound is taken into account, it is not exactly twofold, but is closer to 220 mm.

The two types of bearings are designed to have the same dimensions; this is achieved by using two different rubber compounds with different shear moduli. One bearing is located under each column, except for the heaviest column which is supported by two bearings comprised of the softer compound. The connection between the bearing and the structure is done using recess plates. A 20 mm-thick plate with a hole the diameter of the bearing is bolted to the foundation plate and the bearing sits within this hole. There is an identical recess plate with the same configuration at the top of the bearing.

4.1 Shantou/Hume Test Bearings

The bearings used in the demonstration building in Shantou City were manufactured by Hume Industries of Kuala Lumpur, Malaysia under the supervision of Dr. C.T. Loo to MRPRA specifications. These bearings are comprised of two different types of rubber compound: type I is a soft compound and type II is a hard compound. The two different types of bearings have the same dimensions, but different properties in order to be able to accommodate the variation of the column loads. These high-damping natural rubber compounds, developed by MRPRA for this project, are filled with carbon black, thus they have a significantly lower shear modulus, yet retain the loss factor and the elongation to break. The high-damping characteristics of the rubber resulted in low-cost, lighter and more stable bearings, even under low vertical pressure.

The type I bearings are made of a soft compound with a shear modulus of 0.50 MPa at 100% strain, while the type II bearings are made of a hard compound with a shear modulus of 0.79 MPa at 100% strain. The bearings are circular with a shape factor ($S = \phi/4t$) of $S = 10$, which is rela-

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At strains that exceed the design level strains, the elastomer exhibits a strain hardening effect. This will reduce the displacements if an earthquake of unanticipated level occurs. At the highest level of cyclic shear strain achieved in the test program, the maximum shear stress for the soft compound was 1.41 MPa and for the hard compound 2.55 MPa: the design pressure is 3.61 MPa and 5.64 MPa, respectively, and the ratio of these maximum shear stresses to the design pressures is 0.39 and 0.45, respectively. In a beyond-design-basis earthquake, the base shear to which the superstructure would be subjected would lie between these values. The type of superstructure used for this building is such that significant yielding would be expected in the lateral force-resisting system at this level of base shear, i.e., a softening of the system, increasing the energy dissipation in the frame and increasing the period of the superstructure. Consequently, the superstructure will have to absorb a larger fraction of the overall displacement than it would if it remained elastic and stiff. This will reduce the displacement demand on the bearings, assuring that in the case of an earthquake of unanticipated magnitude, the bearings will not be the weak link in the overall structural system.

5. Base-Isolated Buildings in P.R. China

The Peoples' Republic of China is a highly-seismic region of the world, and Chinese engineers have a long history of earthquake engineering research. For economic reasons, none of the western approaches to base isolation have been used in P.R. China prior to the rubber isolators used in the UNIDO demonstration building. The common approach has been to adopt isolation systems where the isolation mechanism is based purely on sliding friction. This is the simplest isolation approach and there has been a great deal of theoretical analysis of sliding systems and some experimental work by Chinese researchers [8]

The idea of using a sliding joint as the isolation system for low-cost housing is an attractive one because a sliding type of isolation system can be easily incorporated into conventional building designs. In the aftermath of the 1976 Tangshan earthquake, it was observed that a number of masonry block buildings in which the reinforcement was not carried through to the foundation remained standing, while the majority of adjacent buildings in which it did, collapsed. In the buildings where it was not carried through, a horizontal crack was observed at the base of the wall with a residual offset of around 6 cm.

As a result of these observations, the approach adopted in P.R. China up to the present time uses a separation layer between floor beams and the foundation walls. A thin layer of specially screened sand is spread on this surface and the building is constructed on this sand layer. Three small one-story buildings have been built using this technique and one four-story brick dormitory in Beijing for the Strong Motion Observatory Center. This technique is certainly cost-effective, but its efficacy has not been established, and it is unlikely that it will be widely used in the future.

The demonstration project in Shantou City which uses natural rubber isolators as a seismic-resistant design strategy, has sparked widespread interest throughout P.R. China. As part of the UNIDO project, rubber technologists from a local rubber manufacturing company were sent to the headquarters of MRPRA in the United Kingdom and trained in the manufacture of multilayer elastomeric isolators for the purpose of having the Shantou facility become a central manufacturer of bearings for buildings in other parts of P.R. China.

The Shantou City government has expressed considerable enthusiasm in promoting this manufacturing facility. It has been estimated that the Chinese authorities are planning an extensive program of building new public housing that is estimated to be of the order of 600,000,000 sq. m of housing per year for the next ten years. If only 10% of these proposed structures use elastomeric isolators, the demand for bearings will exceed 30,000 per year. The manufacturing facility in Shantou currently has the capacity to produce only 300 isolators per year. Accordingly, the Shantou City government is proposing to develop a new manufacturing center to provide the isolators needed for this building program, and has donated 2 hectares located near the International Airport of Shantou City for this purpose. A complete facility, comprising a manufacturing shop, testing laboratory, product store, training center, and research, development and design office, will be built on this land.

A second base-isolated building using natural rubber isolators manufactured in Shantou City is now under construction in the city. The number of base-isolated buildings in other parts of the country is increasing rapidly. Appendix D is a list of the base-isolated buildings that have been completed, under construction, or in the design phase as of January 1995.

6. Conclusions

When the Foothill Communities Law and Justice Center was completed in 1985, it was the first base-isolated building in the United States. It was visited by many architects and structural engineers, and it created a widespread interest in this innovative approach to the earthquake-resistant design of buildings. Since its completion, this technique has been used in a great many buildings, bridges, and industrial structures all over the United States. The interest generated by the building led to the development of code requirements for base isolation design, culminating in regulations for base-isolated buildings first published in the 1991 Uniform Building Code with further revisions in the 1994 edition.

The Foothill Communities Law and Justice Center was the catalyst for changing the standard design approach to earthquake-resistant design in the United States. The United Nations Industrial Development Organization (UNIDO) base-isolated demonstration building in Shantou City, Guangdong Province is a similar outstanding example of a source for change. Because many parts of P.R China are highly seismic and there is a large demand for low-cost public housing in these areas, this project aims to provide safe and affordable housing. The first base-isolated building in P.R. China to use natural rubber isolators, and the largest base-isolated dwelling that has been completed worldwide, the demonstration building is a truly pioneering project. As a result of this project's success, additional Chinese base-isolated projects are in the design stage, under construction, or completed. Furthermore, it will be the catalyst developing of national code requirements for base isolation design and for the development of a new and potentially highly-profitable industry - the manufacturing of natural rubber isolation bearings. This project will have a global impact as well; it has shown that it is possible to use base isolation technology to construct inexpensive dwellings with increased seismic safety.

The emphasis in most base isolation applications up to this time has been on large structures with sensitive or expensive contents, but there is increasing interest in applying this technology to public housing, schools, and hospitals in developing countries where the replacement cost due to earthquake damage could be a significant part of the GNP. The challenge in this context is to develop low-cost isolation systems that can be used in conjunction with vernacular methods of construction, such as masonry block and lightly-reinforced concrete frames. The Shantou City demonstration project has shown that it is possible to meet both of these goals.

The demonstration building, the first building in China to use elastomeric bearings, is intended to be an example of this new method of earthquake-resistant design. The completion of this project will lead to the widespread use of this new and cost-effective method, increasing the seismic safety of public housing in P.R. China. It is hoped that success of this project will lead to the widespread use of base isolation technology in other earthquake-prone developing countries where seismic-resistant low-cost housing is needed.

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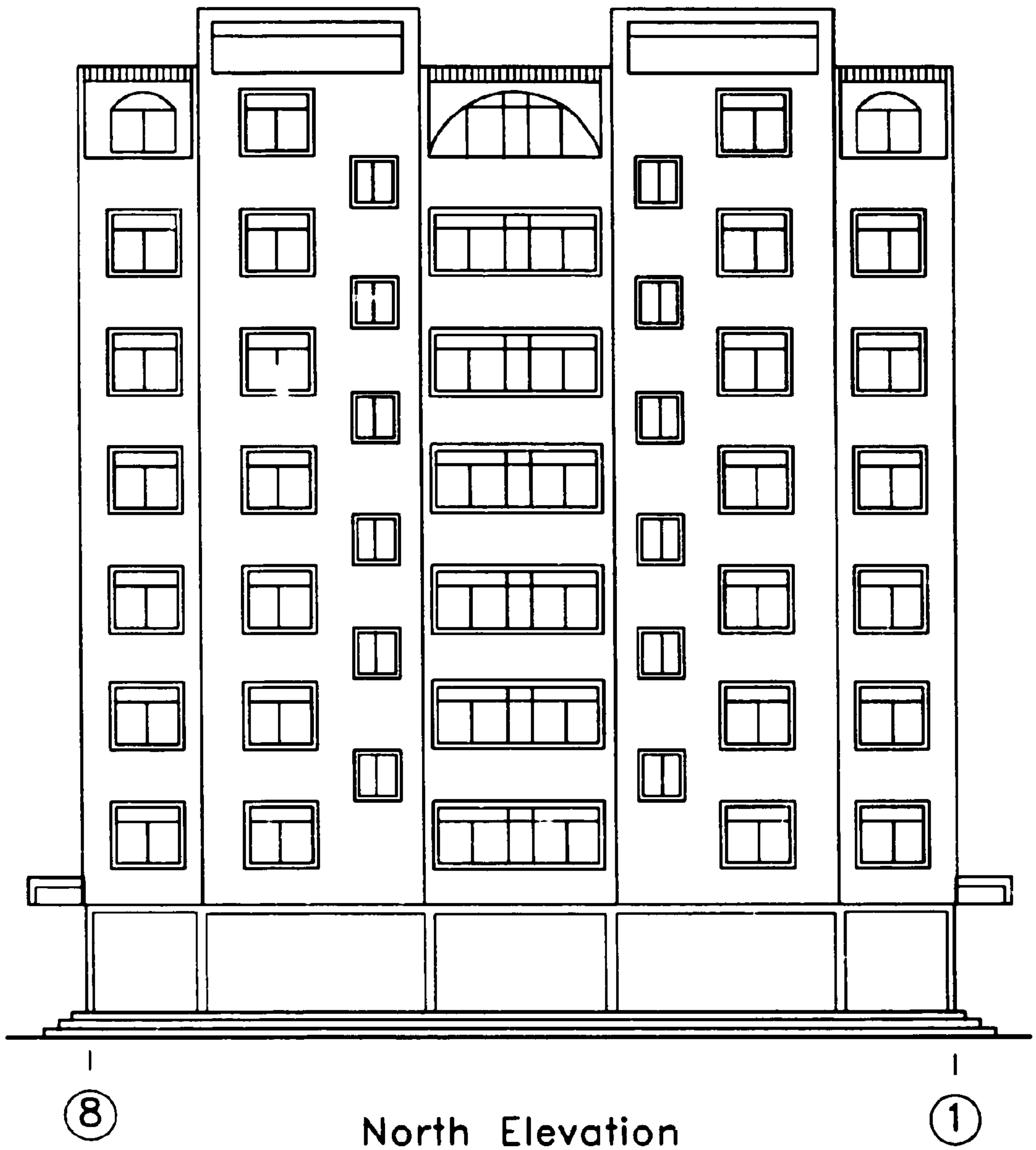

Figure 1a: **North Elevation of Shantou Demonstration Building**

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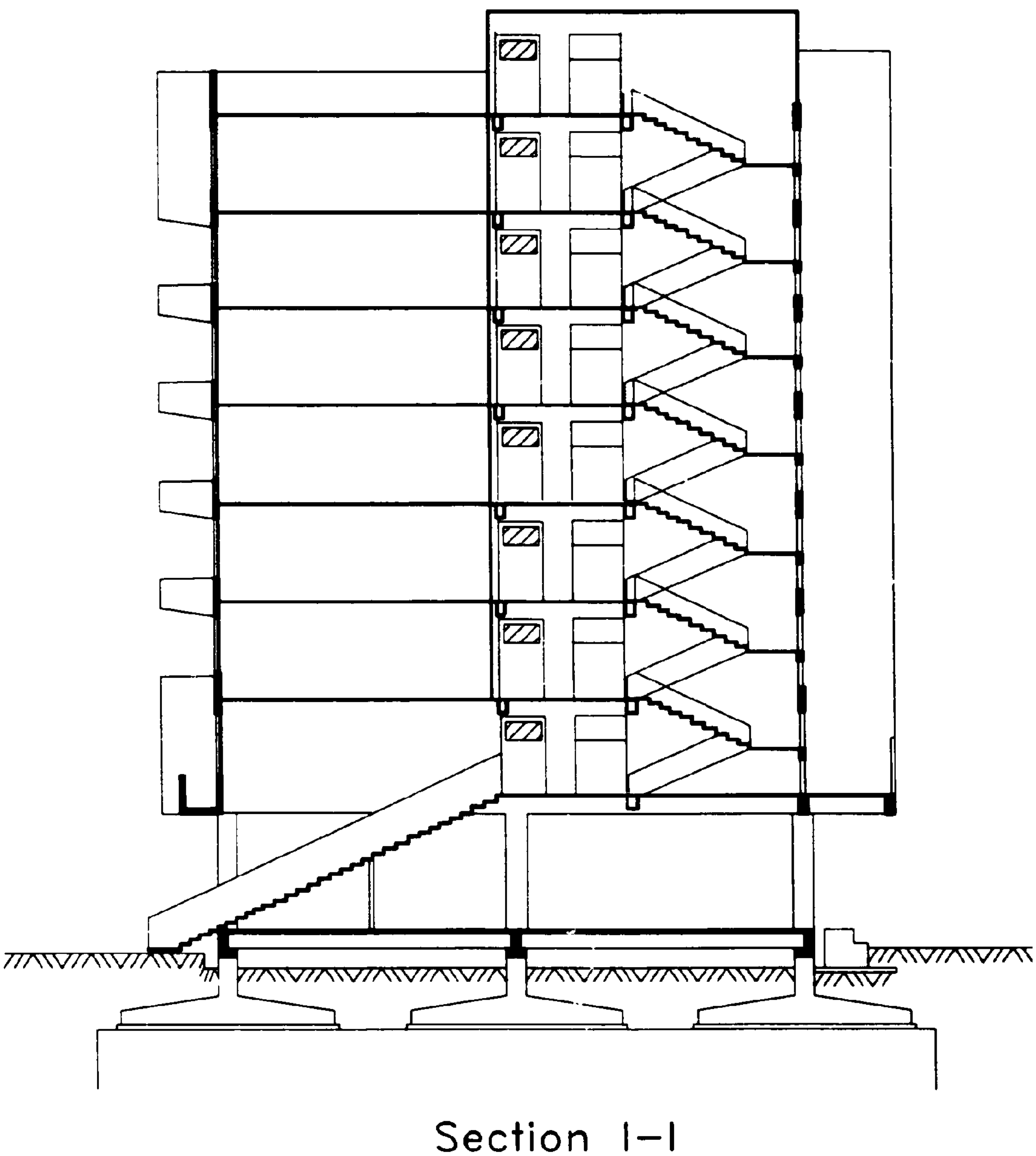


Figure 2a: **Section 1-1 of Shantou Demonstration Building**

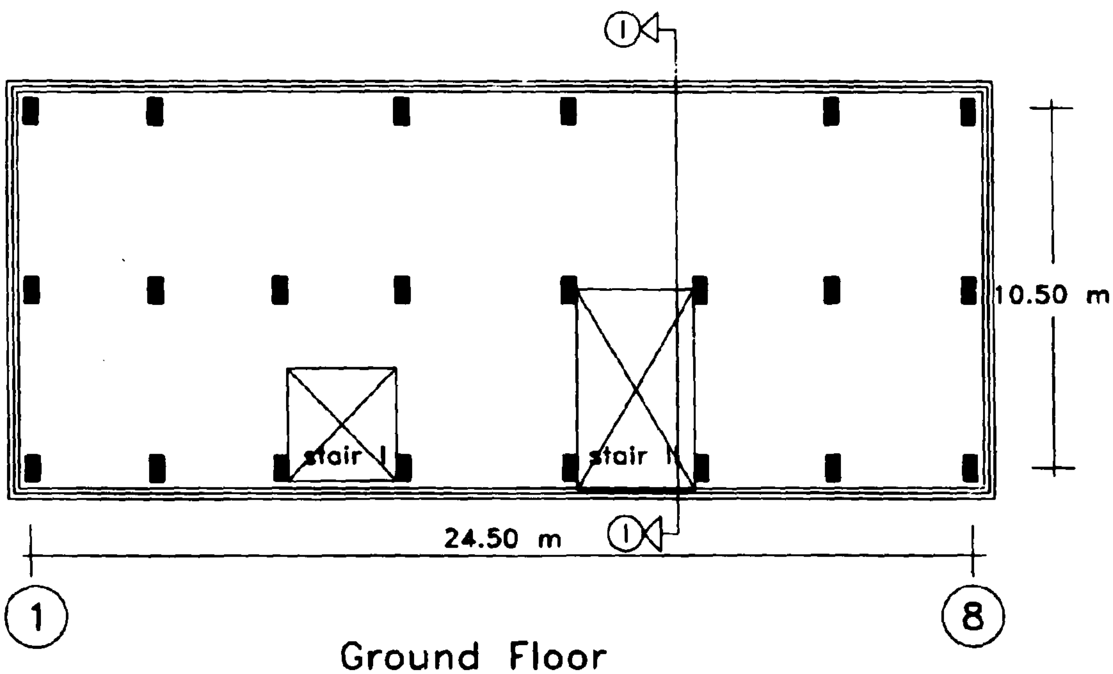


Figure 2b: **Section of Ground Floor and Location of Columns in Shantou Demonstration Building**

Figure 3: Photograph of Shantou Demonstration Building

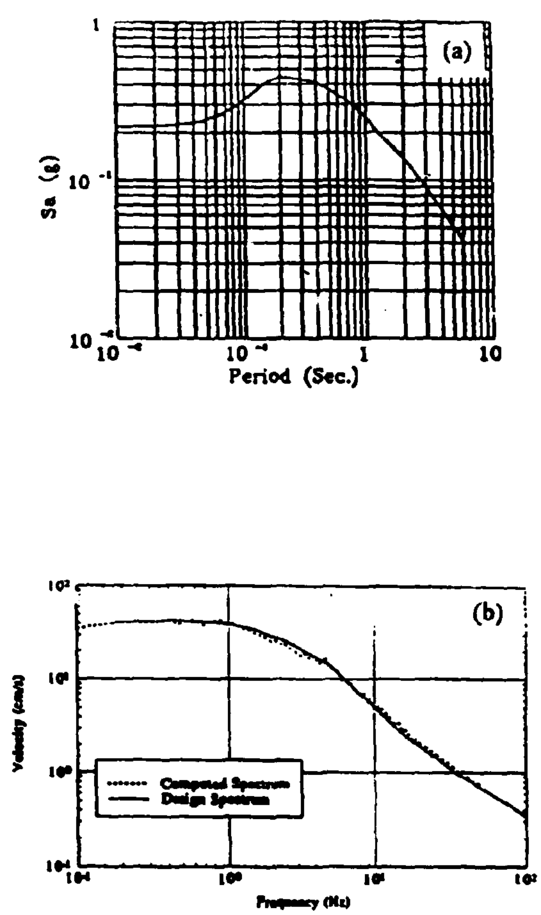
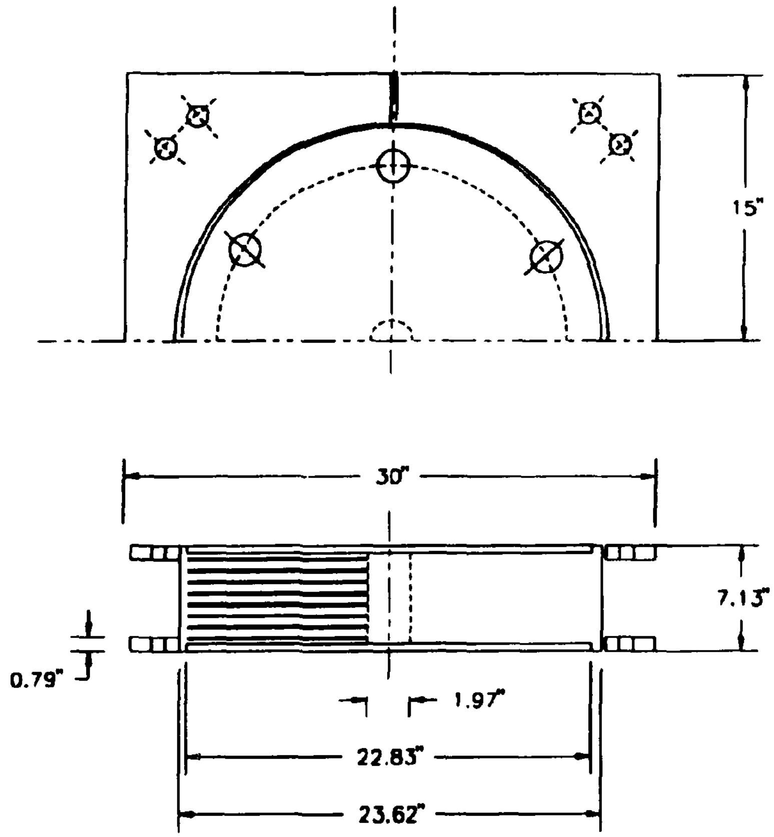


Figure 4: **Site Specific Earthquake: (a) Design Spectrum; (b) Velocity Spectrum of Compatible Synthetic Time History**



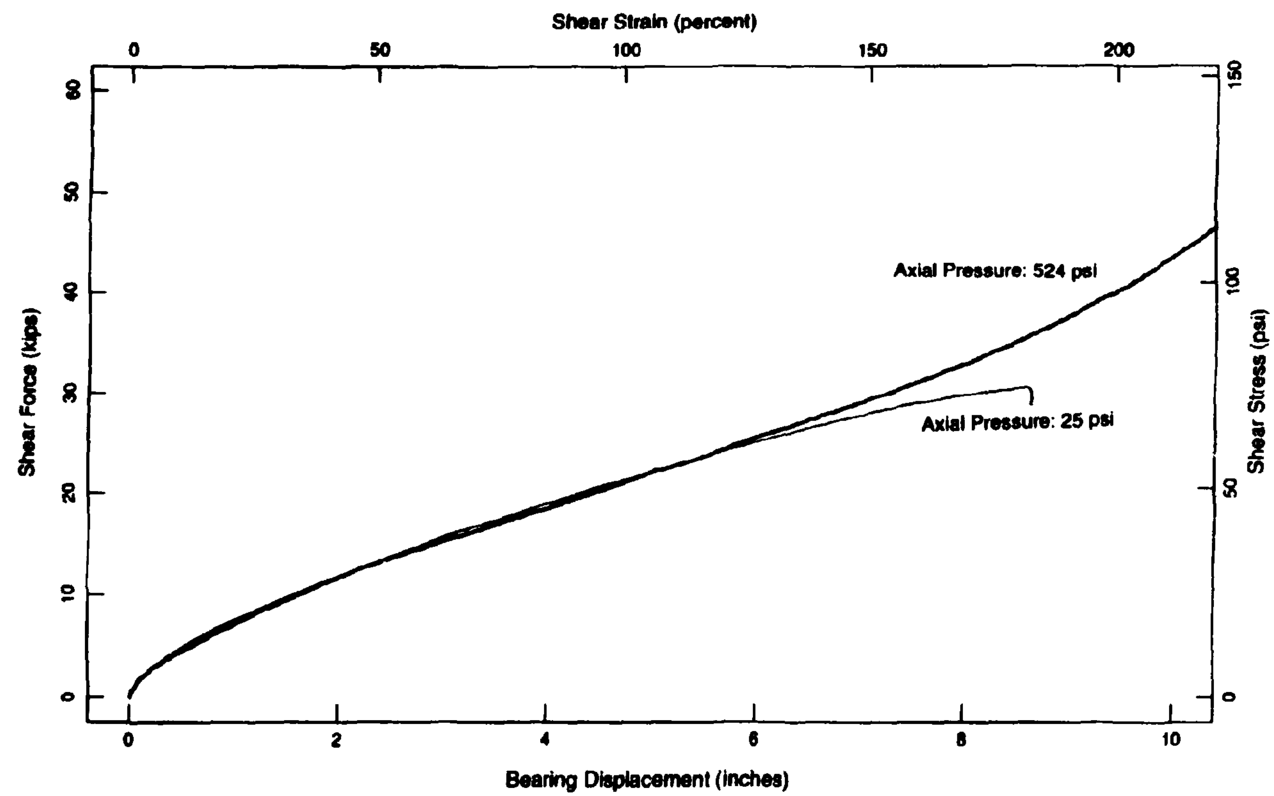
HUME/SHANTOU High Damping Bearing

RUBBER : overall diameter = 23.62" (600 mm)
Thickness = 4.724" (120 mm)
No. of layers = 8 layers

STEEL : Shims diameter = 22.83" (580 mm)
Shim thickness = 3 mm
Cover plate thk. = 20 mm

EARTHQUAKE ENGINEERING RESEARCH CENTER UNIVERSITY OF CALIFORNIA AT BERKELEY	
HUME/SHANTOU BEARING TEST	
BEARING DIMENSION	
TEST DATE: JAN 1994	REVISED: FEB.23, 1994

Figure 5: Connection Between Bearings and Structure and Dimensions of Bearings



Files: 940125.22

Date Plotted: May 06, 1994 Hume/Shantou Bearing Tests Files: 940127.01

Figure 6: **Monotonic Loading on Soft Compound Bearings at Design Pressure and Low Pressure**

Appendix A

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Appendix B

List of Papers and Authors

List of Papers and Authors

Section A: Keynote Papers

Section A: Ke; ,note Papers

Section B: Analysis and Design

Section B: Analysis and Design

Section B: Analysis and Design

Section C: Application and Testing

Section C: Application and Testing

Section D: Bridge and Facility

Section D: Bridge and Facility

Section E: Ideas and Other Approaches

Appendix C

National Science Foundation-Sponsored Participants' Papers

The Architect's Role in Base Isolation

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Abstract

Investigations into implementing base isolation technology in the United States have shown that, in general, architects have played a minor role in the decision to isolate new structures and have been content to receive the design requirements as a given after the decision has been made. In the instances where historic structures have been retrofitted using base isolation, the role of the architect has been stronger, with preservation of architectural features a major determinant in selecting isolation as the retrofit strategy. There have also been a few instances where architectural firms have used their knowledge and experience in base isolation as an effective marketing tool.

In some cases, however, the architect has been seen as playing an inhibiting role (as have many engineering practitioners). One possible reason for this is the uncertainty that surrounds any new innovative technology; the architect must make a careful professional evaluation of any design system on the behalf of the owner. There is a fine line between a negative decision to use innovative technology base on ignorance, or the desire to preserve the status quo, and a negative decision based on informed judgement.

Base isolation is one design choice among many to be evaluated when choosing a seismic-resistant system for a structure. The process of selecting a seismic design system, that ideally should be a shared activity between the architect and seismic engineer from the inception of the design process, is no less important when evaluating a base isolation strategy. In order to facilitate this process, the architect should have a good conceptual understanding of the limitations and strengths of base isolation as part of his professional knowledge.

APPLICATIONS OF BASE ISOLATION SYSTEMS TO THE SEISMIC RETROFIT OF HISTORICAL BUILDINGS IN THE UNITED STATES

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SUMMARY

Seismic isolation (or base isolation) has been used in the United States for the earthquake protection of new buildings and bridges for almost ten years. However, despite the advantages of isolation, the number of applications to new buildings are still relatively few. On the other hand, applications to the seismic retrofit of existing buildings are increasing and notwithstanding the construction difficulties involved, cost-effective solutions have been developed and refined.

Several major public buildings of historic and functional importance have been, or are being, retrofitted using base isolation in the United States at this time. These include the Salt Lake City and County Building in Utah, the Mackay School of Mines at the University of Nevada in Reno, the Oakland City Hall, and the Ninth Circuit United States Court of Appeals in San Francisco. Future applications include the City Halls for both San Francisco and Los Angeles. In addition to these public buildings, private buildings have also been retrofitted using isolation such as the 8-story, non-ductile, concrete frame building that houses a computer facility for Rockwell International in Los Angeles. In each case, seismic performance has been improved while minimizing disruption to the occupants and reducing the overall reconstruction cost.

This paper describes the application of seismic isolation to historical buildings and presents four examples in which base isolation has been used to preserve architectural integrity while at the same time adding a substantial measure of seismic protection.

INTRODUCTION

Seismic isolation is a design strategy based on the premise that it is both possible and feasible to uncouple a structure from the ground and thereby protect it from the damaging effects of earthquake motions. To achieve this result, while at the same time satisfying all of the in-service functional requirements, additional flexibility is introduced usually at the base of the structure. Additional damping is also provided so as to control the deflections which occur across the isolation interface.

The concept is not new and many proposals have been made since the turn of the Century for "...devices which absorb or minimize shock to buildings arising from earthquake, vibrations caused by heavy traffic or other disturbances of the earth's surface" [1].

These four buildings are summarized in Table 1 and further discussed in subsequent sections of this paper. It is noted that future applications include the City Halls for both San Francisco and Los Angeles. It is also noted that the 80-year old Parliament House building for the New Zealand Government was recently retrofitted using isolation.

Table 1. Retrofitted Buildings Using Seismic Isolation

Building	Type of Construction	Year Constructed	Year Retrofitted	Isolation System	Additional Strengthening Required?
City and County Building Salt Lake, Utah	unreinforced brick and sandstone	1894	1986/7	lead-rubber bearings	minor
Mackay School of Mines, Reno, Nevada	unreinforced brick, wood floors and roof trusses	1908	1990	high damping rubber bearings and sliders	minor
US Court of Appeals, San Francisco, California	non-ductile steel frame and unreinforced masonry cladding	1905	1993/4	friction-pendulum bearings	yes
City Hall Oakland, California	non-ductile steel frame and unreinforced masonry cladding	1914	1993/4	lead-rubber bearings	yes

City and County Building, Salt Lake, Utah

The City and County Building in Salt Lake City, Utah was completed in 1894. It is located in a moderate seismic zone and has been damaged in past earthquakes. During a complete rehabilitation of the building in 1987, seismic isolation was used to improve its performance in future earthquakes. This description of the building and the isolation system is by Elsesser, Walters and Allen [5].

The Salt Lake City and County building is a monumental, highly ornamented unreinforced brick and sandstone structure measuring 130 x 270 feet in plan, with five main floors and a 12-story clock tower (Figure 1). The plan is approximately doubly symmetrical (Figure 2).

The seismic vulnerability of the structure, due to its lack of reinforcement, is aggravated by the closeness of the site to the nearby active Wasatch Fault Zone. The building has a record of damage from various earthquakes, the largest occurring in 1934 with a Richter magnitude of about 6.2. Seismic damage to the building included cracks in the bearing walls and loss of sculptures, roof stones and mechanical equipment from the clock tower.

The structure is supported by bearing walls of unreinforced brick and sandstone masonry which rest on sandstone plinths and 8'-6" wide continuous concrete footings. The interior brick bearing walls have a maximum thickness of 24 inches at the base. The exterior walls, which have an exterior wythe

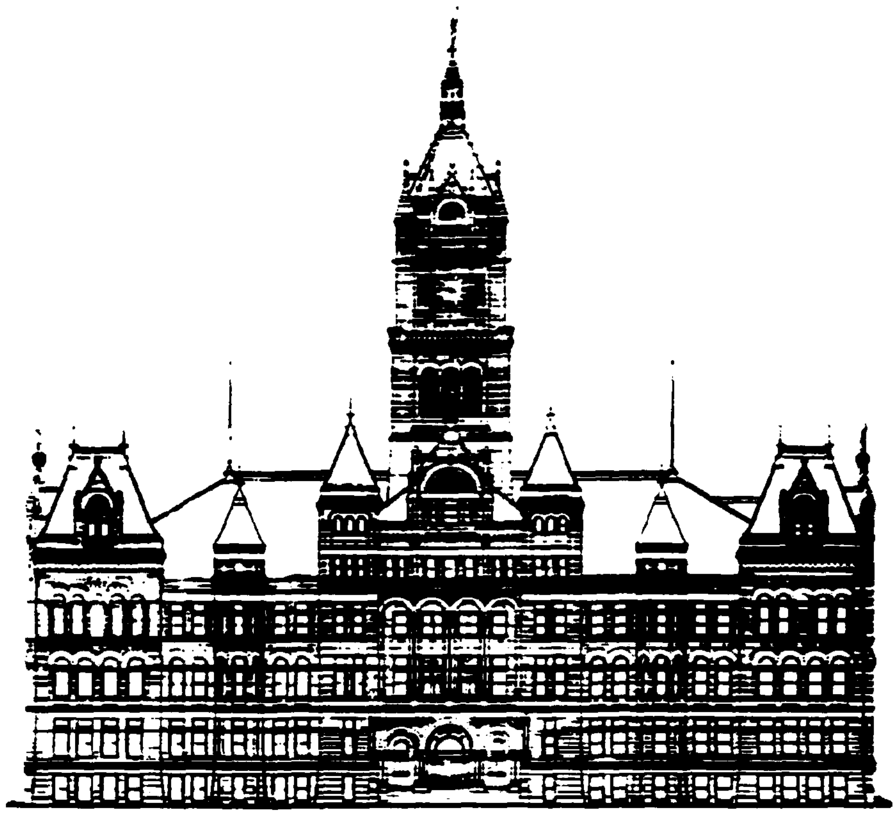

Figure 1. Elevation of City and County Building, Salt Lake City (from Reference 5)

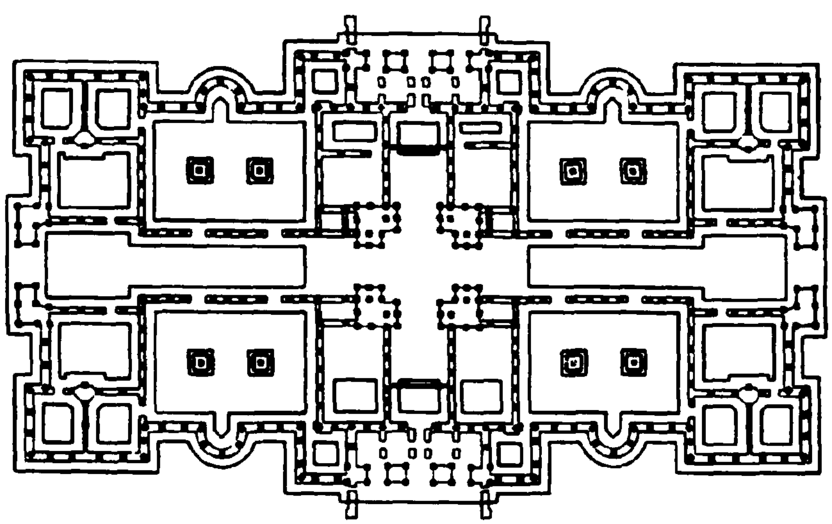

Figure 2. Basement plan showing isolator locations, City and County Building, Salt Lake (from Reference 5)

of sandstone masonry, reach a base thickness of 36 inches. The multiple wythes of brick in each bearing wall are bonded together solely by the original sand-lime mortar, which is quite deteriorated in many locations.

The central unreinforced masonry tower, which is approximately 40 feet square in plan at its base, rests on four solid piers of sandstone masonry which are L-shaped in plan and have a maximum dimension of 13 feet.

The 1st and 4th floors are framed with timber joints and planks, with a wooden floor surface in some locations and a concrete topping in others. The 2nd and 3rd floors are framed with steel beams supporting shallow brick "jack arches", which are covered with stone ballast and a concrete topping. At all levels, horizontal anchorage between the walls and floors is minimal.

In late 1984, the architects, the Ehrenkrantz Group of San francisco and Burtch W. Beall, Jr., FAIA, of Salt Lake City, considered three different rehabilitation schemes, one of which was base isolation. The other two concepts involved "conventional" reinforcement systems which required the addition of concrete shearwalls and the corresponding removal and replacement of costly architectural wall finishes, such as oak wainscotting and plaster. In addition, conventional methods would have required a substantial amount of reinforcement to tie the walls to the floors and to resist out-of-plane wall loading, all of which would also be disruptive to the finishes. In order to minimize the need for wall reinforcement and replacement of finishes, it was decided to concentrate on developing an economically competitive base isolation scheme. By isolating the structure, horizontal accelerations were reduced substantially, thus minimizing the need for wall strengthening and, thereby, removal and replacement of architectural finishes. Another benefit of base isolation was the reduction in out-of-plane anchorage forces and bending moments in the unreinforced masonry walls.

The installation sequence required that each masonry wall be gripped between a pair of reinforced concrete "side beams" which were then notched into each wall to allow direct bearing, and tied together through the wall by regularly spaced concrete cross beams and ducted prestressing rods. Once these beams were cast and clamped to the wall, portions of brick and plinth below the cross beams were then removed, creating a space in which the isolators and bearing plates were installed to bear on the existing concrete footings (see Figure 3). A similar scheme was developed for the central tower, whereby each of the four sandstone support piers was jacketed with a reinforced concrete collar which was then clamped to the pier leg in each direction by prestressing rods. Pieces of stone plinth below the pier were then removed in stages, starting at the corners, to create space for the new isolators.

In total, there are 447 isolation bearings in the building. Of these, 208 are lead-filled elastomeric bearings using standard natural rubber. They are 17 inches square by about 15 inches tall with a 2.8 inch diameter lead core. The remaining 239 isolators are standard elastomeric bearings of the same overall size but without a lead core.

Average displacements and base shears for two design earthquakes were calculated to be 4.1 inches and 0.085W for the 0.2g event and 10.3 inches and 0.19W for the 0.4g event. Tower base shears were 530 and 840 kips respectively. Since the base shear capacity of the existing masonry was estimated to be 0.09W and 650 kips respectively, no masonry strengthening was specified. However some floor-to-wall ties were still required and strengthening of the clock tower, above the roof line of the main building, was also performed.

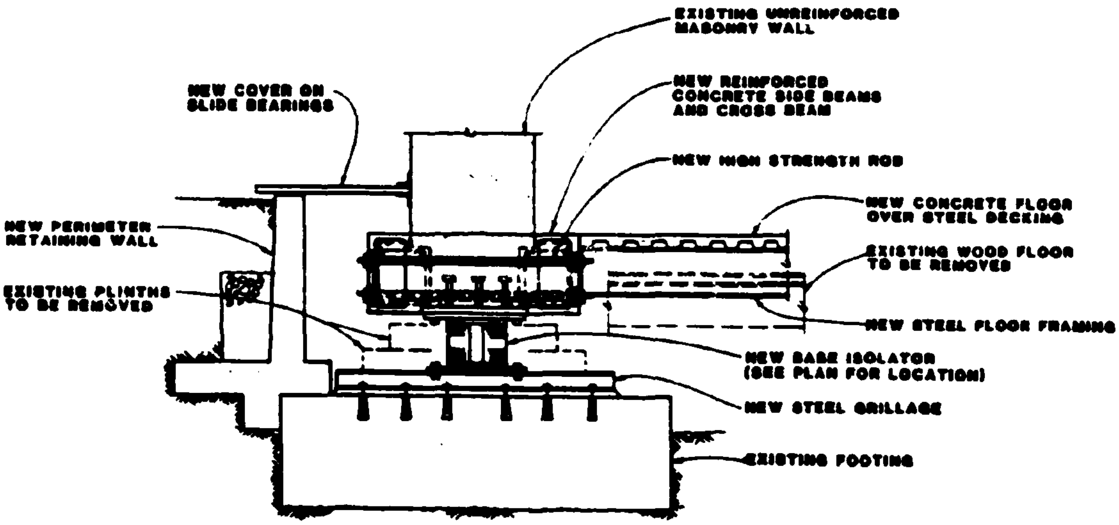

Figure 3. Typical elastomeric isolator installation details City and County Building, Salt Lake (from Reference 5)

To reduce the effects of elastic axial shortening of the isolators, which was calculated to be about 0.1 inches, the isolators were pre-loaded by flat hydraulic jacks placed beneath the new bearing plates before shimming and grouting the plates. With the isolators installed, the remaining plinth stones were then removed to allow the isolators to translate freely in the event of an earthquake.

<u>The Mackay School of Mines, University of Nevada, Reno</u>

The Mackay School of Mines was one of the original buildings on the campus of the University of Nevada in Reno. Constructed in 1908, the building is designated as a National Historic Monument (Figure 4). It has been remodelled several times during its lifetime and most recently in 1990 when a seismic retrofit was also undertaken using base isolation. This description of the building and the isolation system is by Way and Howard [6].

Constructed of local bricks in the early 1900's, the building is entirely unreinforced masonry (URM) with wood joist floors throughout. This method of construction is a life safety hazard and in recent years the structure has not been used as a classroom facility so as to restrict the occupancy loads. Many studies have been done to address this hazard and remodelling has taken place over the years that has significantly altered the original design of the building.

In 1975 a master plan for the Mackay School of Mines was developed that proposed the expansion of the School with the construction of an entirely new facility as well as the rehabilitation of the original Mines building. In this plan it was proposed to strengthen all of the URM walls of the original building by adding reinforcing steel and gunite concrete. At the same time,

Figure 4. Elevation of the Mackay School of Mines
University of Nevada, Reno
(from Refernce 6)

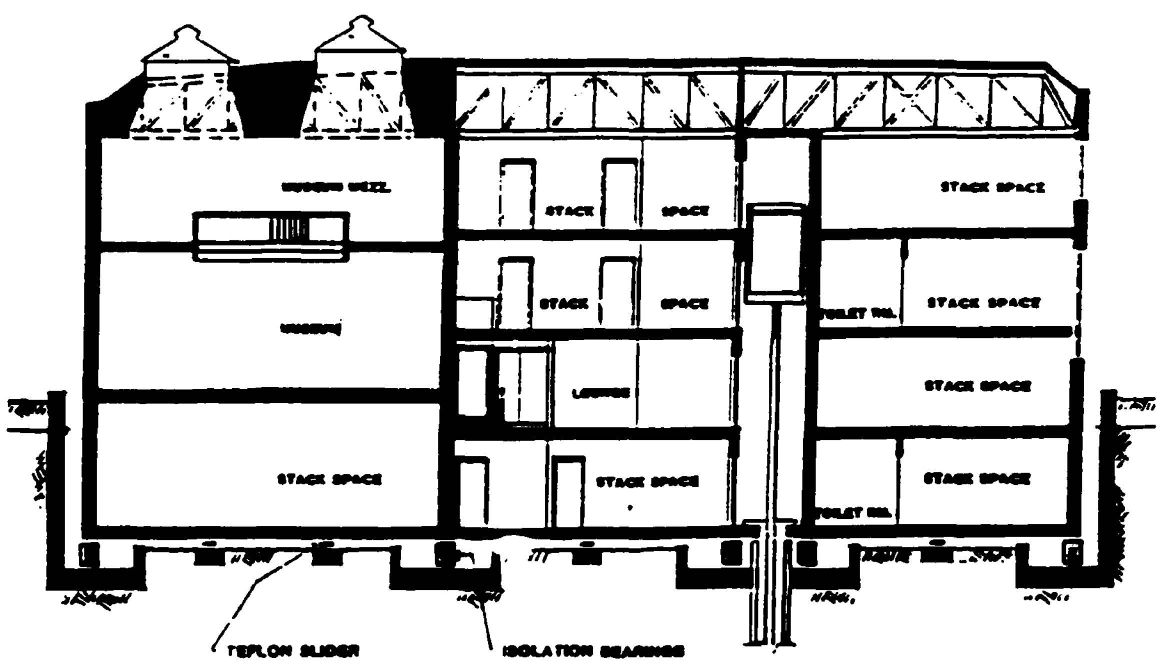

Figure 5. Typical section, Mackay School of Mines
University of Nevada, Reno
(from Reference 6)

additional space was to be provided (by developing a full basement) so that the Engineering/Mines library might be relocated within the building.

In reviewing the 1975 study and its proposed strengthening operation, it was quickly determined that the gunite solution would be impractical given the fact the building is on the list of Historic Places and structural modifications that alter the appearance of the building were prohibited. Alternative methods to seismically strengthen the building were investigated. Seismic isolation was adopted because it could be built relatively inexpensively, satisfy the seismic requirements of the structure and not interfere with the historic nature of the building. Since all of the primary URM bearing walls would be underpinned any way, it proved to be a very short step to incorporate base isolation into the design of the building.

The isolation system consists of 67 high-damping rubber bearings and 42 PTFE/stainless steel sliding bearings. The bearings are located between a suspended concrete flat slab (which serves as the basement floor) and the foundation (Figure 5). This slab also acts as a structural tie above the isolation system.

To provide support for the flat slab so that the spans become manageable, teflon sliders with rubber seals for the protection of the teflon surfaces were incorporated. The sliders, which were targeted at a coefficient of friction of 0.10, also add additional damping to the system. Three different sizes of high damping bearings were designed to accommodate varying vertical loads, ranging from 45 to over 300 Kips.

The sequence of construction involved first the underpinning of the concrete walls and footings. Pockets were left open in the walls at the top of the footings for the installation of the bearings. A steel shim plate was placed between the wall and the footing to maintain a separation after cutting. Flat hydraulic jacks were subsequently installed on top of the bearings. After all the bearings and flat jacks were in place, the hydraulic system was pressurized. In the process, the vertical load was transferred from the steel shim plates to the bearings, after which the steel shim plates were removed. The flat jacks were grouted and left in place.

Site specific time histories of ground motion were developed for the site which had peak ground accelerations ranging from 0.53g to 0.58g. Calculated displacements for the building averaged 5.9 inches with a maximum base shear of 0.15g. The total equivalent viscous damping from the combined hysteretic and friction bearings was estimated at 26%. Peak accelerations in the upper floors and roof did not exceed 0.30g which was the threshold at which out-of-plane failure of the masonry walls was expected to occur. As a consequence no additional strengthening was performed.

Ninth Circuit U.S. Court of Appeals, San Francisco

The Ninth Circuit U.S. Court of Appeals at Seventh and Mission Streets in San Francisco was damaged by the 1906 San Francisco earthquake and again in 1989 by the Loma Prieta earthquake. It is now being retrofitted with a combination of seismic isolation, additional shear walls, and diaphragm strengthening. When completed in 1995, it will be the largest base-isolated structure in the United States. This description of the building and the isolation system is by Amin, Mokha and Fatehi [7].

Constructed in 1905, the original building was U-shaped until 1933 when a fourth wing was added, giving the building a rectangular shape with a central atrium. Approximate plan dimensions are 330 feet by 265 feet; the total floor area is about 350,000 square feet. The building is a five-story, 80-foot tall structure with steel framing, concrete slabs, unreinforced granite masonry exterior walls, and hollow clay tile interior partitions. The foundation consists of steel grillage footings under the 1905 building and piers with reinforced concrete pile caps under the 1933 addition. Interior finishes are extremely ornate. They include carved marble figures, inlaid marble walls and floors, and highly articulated plaster ceilings. The Beaux Arts building is on the National Register of Historic Places. Figure 6 is an external view of the building.

The isolation system chosen for the building was the friction pendulum system (FPS) which consists of an articulated slider on a concave spherical stainless steel surface. The slider is faced with a PTFE composite bearing material and the curvature of the spherical surface provides a lateral restoring force due to gravity and self weight. Once sliding commences the period of vibration is independent of the building weight and directly proportional to the radius of curvature. These isolators are compact in size which was a distinct advantage given the restricted head room in this particular building (between the pile caps and the basement floor). They were preferred to elastomeric isolators, which would have been physically larger in size and required more extensive structural modification to the foundation substructures to allow their installation.

As a consequence, 256 FPS bearings are being installed in the building at the present time. Each has a radius of 74 inches which corresponds to a design isolation period of 2.75 seconds. The design coefficient of friction is 0.07 (maximum) for sliding velocities greater than 2.0 in/sec and 0.045 (minimum) at very low sliding velocities. A typical FPS isolator installation is shown in Figure 7 in which the concave sliding surface is shown facing downwards. It is fixed to the underside of a new reinforced concrete footing that is cast integrally with a column jacket. This jacket is required to strengthen the existing steel column particularly for bending about its weak axis. The articulated slider is fixed to a masonry plate that is first levelled and then grouted to the supporting pedestal.

Calculated values for mean displacement and base shear at 1.2 times the design level earthquake (0.48g peak ground acceleration) are 13.6 inches and 0.20W respectively. Peak accelerations in the upper floors are estimated at 0.47g. The maximum interstory drift ratio is 0.12%.

The existing stone, brick masonry, and concrete slab diaphragms do not have adequate capacity to withstand the above forces, and are being strengthened. Vertical elements, which are being added for lateral force resistance, include: four perimeter concrete frames, four interior concrete frames around the courtyard, and a number of short concrete shearwalls perpendicular to the frames. These concrete elements will be tied to existing steel columns and beams with shear studs and rebar ties. Concrete slab diaphragms will be connected to the frame walls by drilling and epoxy grouting into the existing slab. Since the concrete slab strengths in the 1905 building are very low (in the range of 1000 psi), diaphragm strengthening is required, to transfer forces to vertical elements. Topping slabs on the roof and some floor areas will be removed and replaced by new reinforced concrete slabs. Where diaphragm strengthening is required but topping slab removal is impossible due to marble or mosaic floors, the strengthening will be done with steel plates attached to the bottom flanges of existing steel beams below the floor level. Other miscellaneous strengthening includes bracing of existing parapets and strengthening of existing penthouses.

<u>City Hall, Oakland, California</u>

The Oakland City Hall, completed in 1914, was the first high rise government office building to be constructed in the United States and is

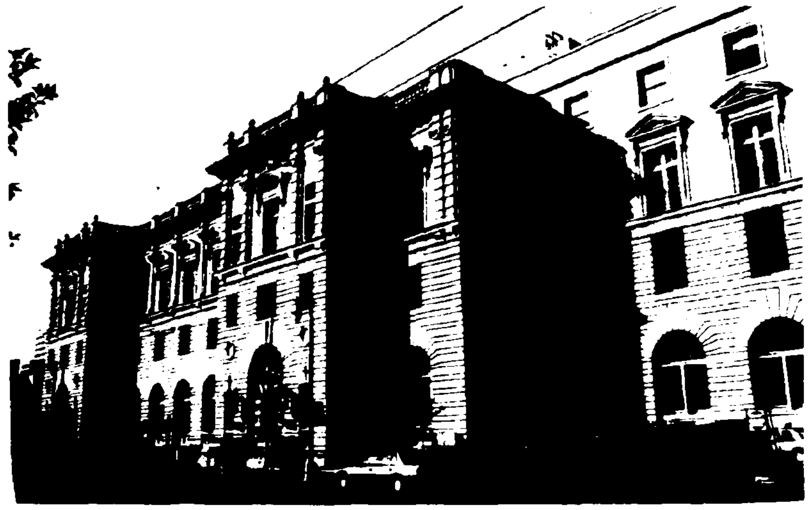

Figure 6. Exterior view of US Court of Appeals
San Francisco, California
(from NCEER)

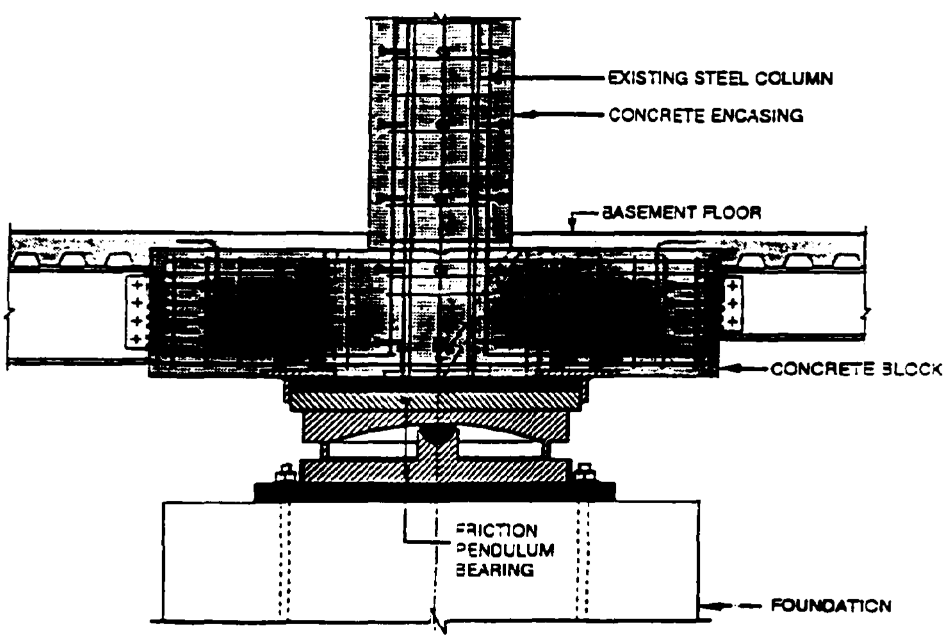

Figure 7. Typical friction-pendulum isolator installation
US Court of Appeals, San Francisco
(from NCEER)

today listed in the National Register of Historic Places. It was heavily damaged during the Loma Prieta earthquake, at which time the building lost 20% of its lateral strength in the north-south direction, and 31% in the east-west direction, primarily due to extensive cracking of the numerous interior hollow clay tile partitions in the office tower. The clocktower at the top of the building "rocked" during the earthquake, resulting in large shear cracks in some infill masonry walls and severe damage to several support transfer girders supporting the clocktower. The building is presently being retrofitted with a seismic isolation system in combination with structural strengthening in the east-west direction. When completed in 1994, it will be the tallest seismically isolated building in the United States. This description of the building and the isolation system is by Honeck, Walters, Sattary and Rodler [8].

The building is 18 stories high, 324 feet above the street, and contains a one-level full basement (Figure 8). The lowest and widest portion of the building, known as the podium, is 3 stories and contains a central rotunda, council chambers, and administration offices of the Mayor and City Manager. Above the podium is a 10-story office tower. Above the office tower is a 2-story clocktower base supporting a 91 foot high clocktower. The building steps back at each successive portion.

The structure of the building is a riveted steel frame with infill masonry walls of brick, granite and terracotta. The clocktower is clad entirely in terracotta over brick masonry. The building is supported on a continuous concrete mat foundation.

A comprehensive post-earthquake study done by a team of architects and engineers, and reviewed by the Federal Emergency Management Agency and State Building Officials, concluded that seismic isolation was the most cost-effective and behavior-effective method to protect the landmark building from seismic hazards.

To implement this solution, many issues required resolution including:

- assessment of the properties and interaction of the riveted steel frames and the infill masonry, and evaluation of the safe drift capacity of this system.
- assessment of the dynamic modal properties.
- control of isolator uplift during the maximum credible earthquake.
- provision of a rational, continuous path for resistance of lateral loads where such a path previously did not exist, while minimizing disruption of historic elements.
- provision for the jacking and re-support of columns.
- development of methods to repair and protect historically sensitive, brittle elements of the building.

In this short paper only the provision of the lateral load path and the isolation system is described below. The remaining issues are discussed in Reference 8.

As shown in Figure 8 the building has four distinct sections, each with its own lateral load resisting system: the clocktower, office tower, podium and basement. Lateral strengthening of the office tower and the provision of outrigger trusses in the basement are described below. The clocktower and podium are summarized in Reference 8.

In order to assess the contribution of existing masonry materials in the 10-story office tower to the resistance of future lateral loads, extensive in-situ testing was performed on the brick infill to determine its strength and stiffness properties. It was determined that 100% of the lateral forces

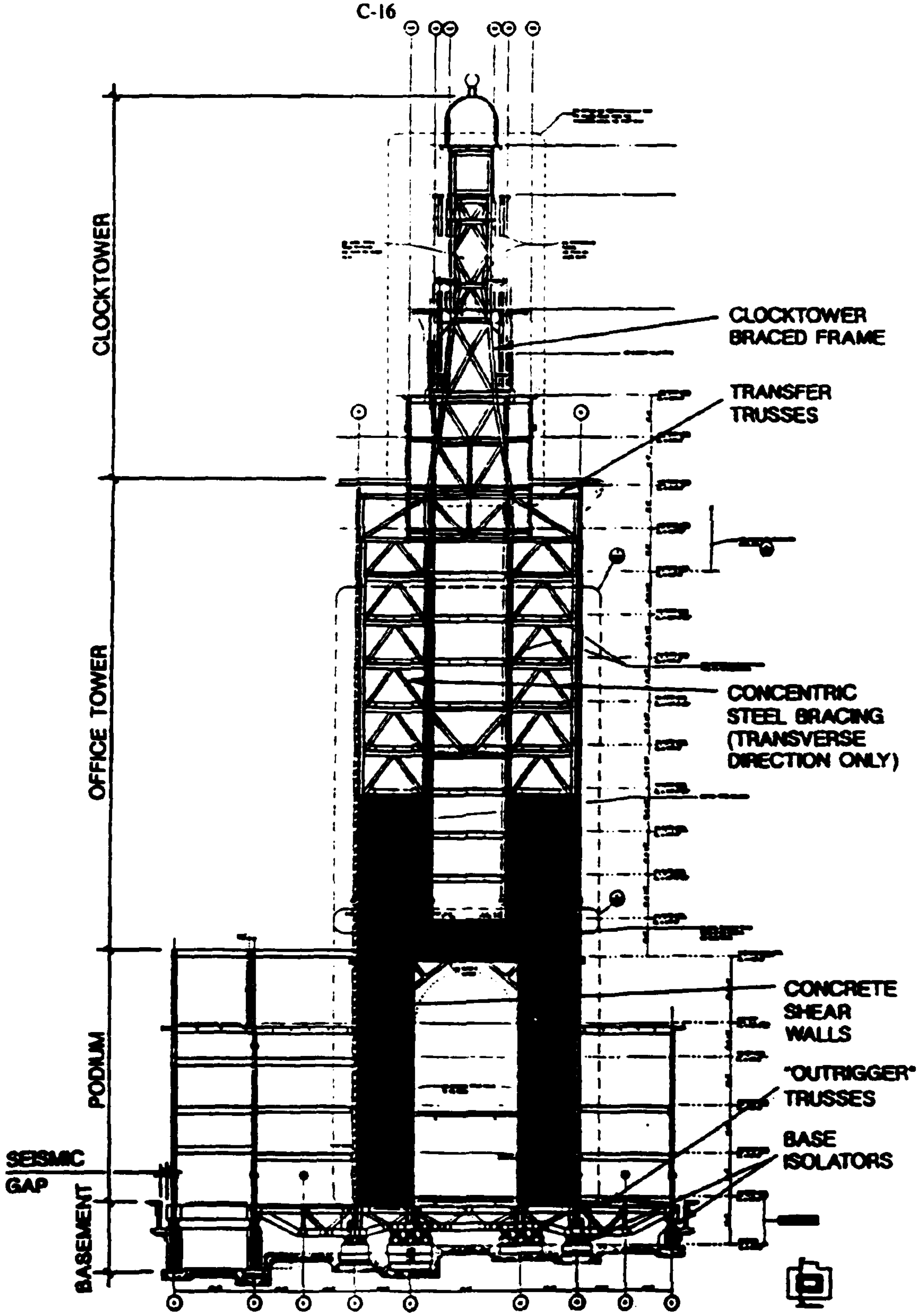

Figure 8. Typical section, Oakland City Hall, California
(from Reference 8)

could be resisted by the north-south (longitudinal) masonry infill walls provided base isolation was used. In the transverse east-west direction, since the walls are shorter in length, it was determined that supplemental bracing was required to control potential damage to these walls. Two lines (4 bays) of concentric steel braced frames, supported by eight new columns that support the clocktower trusses, were designed to resist approximately 25% of the lateral load in the transverse direction; the remaining 75% would be resisted by the existing masonry infill walls. These braced frames extend down to the 7th floor where they transition to concrete shear walls. The new braced frames were designed to be compatible with the stiffness of the existing transverse infill masonry walls. New steel collector beams were added at all floors to deliver lateral loads to the new braced frames and shear walls.

The concrete shear walls terminate on new 8.5 feet deep continuous steel "outrigger" trusses in the basement. Typically, double lines of trusses straddle the existing steel columns. These trusses are encased in concrete to provide additional stiffness and to tie the double lines of trusses together. The purpose of the trusses is to distribute the building overturning moments over a broad footprint so that the base isolators located beneath the office tower perimeter will not be overloaded nor subjected to any appreciable uplift. The existing basement concrete walls have concrete side beams added on both sides so that the walls, after being cut, will span between base isolators to support the massive exterior podium walls.

A system of horizontal steel braces forms a "diaphragm" below the first floor to deliver lateral loads to a system of 111 lead-rubber isolation bearings. The isolators are supported on a grid of existing and new steel/concrete pedestals that are supported on the existing concrete mat foundation. Multiple isolators (up to 4 per group) are used to support individual columns with dead loads in excess of 3300 kips each.

Calculated displacements during the design earthquake (475 year return period) are 13 inches near the center of the building and 17 inches near the corners. At this displacement the effective period is 2.8 secs and the maximum base shear is 0.14W.

The use of base isolation as a seismic upgrade strategy has reduced the expected seismic force levels in the building and resulted in fewer shear walls than a traditional method of upgrade would require. With fewer shear walls, the impact of the upgrade on the historically sensitive interior finishes of this landmark building is significantly reduced. Base isolation proved to be an economically feasible solution when compared to conventional fixed base upgrade schemes. Through a combination of base isolation, extensive testing of the existing exterior masonry infill walls and a comprehensive finite element study of typical wall panels, the majority of seismic lateral forces can be shown to be resisted by existing unreinforced masonry infill walls in the office tower portion of the building.

By designing stiff, concrete-encased, steel trusses in the basement, seismic overturning forces from this relatively tall building have been distributed over many base isolators so that they will not be overloaded nor subjected to appreciable uplift.

CONCLUSIONS

This short paper has presented four case studies in seismic retrofit and in this way has illustrated the application of base isolation systems to the retrofit of historical buildings. The particular advantage of the isolation technique for this class of building, is the minimal disruption to the interior and exterior finishes and the protection of the architectural integrity of these structures. It is however noted that in each of the cases discussed above, additional strengthening was also required. This was necessary because the original buildings were designed before modern seismic codes were adopted and constructed of non-ductile materials with poor connection details. As a consequence their capacity for seismic load was so weak that isolation could not reduce the demands sufficiently, i.e., to below the existing strength. Nevertheless the amount of additional strengthening is less than if isolation had not been used and in some cases it was relatively minor. Further, as contractors develop and refine construction techniques necessary to install isolation systems and strengthen existing buildings, the cost of doing so will decrease. The number of historical buildings that are retrofitted with seismic isolation can be expected to increase in the years ahead.

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Research and Development of a Sliding Isolation System

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ABSTRACT

A sliding isolation system has been developed and studied. This system is composed of sliding bearings with relatively high friction coefficient (20% in high velocity range) and rubber devices with both restoring and stopper functions. The seismic response characteristics of a bridge model with and without the sliding isolation systems were studied using the shaking table in the Public Works Research Institute, Ministry of Construction, Japan. A numerical model that can be efficiently used for the analysis and numerical simulation of the seismic response of isolated bridges is proposed. The experimental and numerical simulation study demonstrates the effectiveness of the sliding isolation system for protecting bridges from earthquakes, even when a bridge has flexible piers.

INTRODUCTION

In order to study dynamic characteristics of sliding-isolated bridges, a sliding isolation system composed of sliding bearings and rubber restoring force devices was developed and tested on a shaking table with a bridge model in PWRI, as a joint research project between National Center for Earthquake Engineering Research (NCEER), USA, and PWRI. This bridge model was designed by the Japanese standard specification for bridges and the bridge girder is supported by two flexible piers. This model was previously used for the shaking table test of the isolation systems with lead rubber bearings (LRBs) and high damping rubber bearings (HDRs)[1]. Several earthquake records and design earthquakes with different intensities and frequency contents were used as input motions in the test. Unfortunately it was not possible to use the input motions with large intensities due to the displacement limitation of the shaking table. Therefore the behavior of the bridges under such large intensity earthquakes was analyzed by numerical simulation. Such numerical simulation makes use of the dynamic characteristics of the sliding system identified by an identification test performed prior to the shaking table test and confirmed by the shaking table test carried out at the level of earthquake intensity that the table was able to tolerate.

To be able to numerically simulate with satisfactory accuracy the response of actual bridges equipped with sliding isolation systems is also important for the design of bridges with such isolation systems. In this respect, M. Constantinou proposed an analytical procedure using Y.K. Wen's model[2]. The model provides a convenient analytical tool to solve relatively simple problems. However, for the design of more complicated bridges such as a continuous bridge supported by multiple piers with sliding systems, a simpler model is much more preferable for computational ease. For this purpose, a simple numerical model based on the direct integration method is proposed and utilized throughout this paper. The model is idealized in terms of an explicit mathematical expression[3 & 4], instead of the usual algorithm that describes the stick-slip model of sliding. The accuracy of the numerical procedure is confirmed by the comparison of results between the shaking table test and the numerical simulation performed on the bridge model. The advantages of the sliding isolation system is demonstrated here not only by the shaking table test but also by means of numerical simulation.

LOADING TEST AND SHAKING TABLE TEST

Experimental Setup

The bridge model used in the shaking table test is shown in Fig.1. The bridge span, the pier height, and the deck weight are 6.0m, 2.5m, and 390 kN respectively. The fundamental natural period of the bridge is 0.48 seconds when the girder is supported by piers through a fixed bearing on one end and a roller bearing on the other.

The sliding type isolation system developed by NCEER[3] is equipped on the bridge. Two sliding bearings and a rubber restoring device were installed on each pier with the rubber device located in the middle of two sliding bearings. In total, therefore, four sliding bearings and two rubber restoring devices were used for the model.

As shown in Fig. 2, the sliding bearing consists of a stainless steel plate attached to the deck and a circular Teflon plate (diameter = 10 cm) fixed on the pier through a bearing plate. The bearing plate has a semi-spherical surface which can rotate freely from the pier deformation to keep the Teflon plate in horizontal and in perfect contact with the steel plate. A load cell is installed between the bearing and the pier to measure the vertical load on the bearing. The pressure on the sliding surface of the Teflon plate is evaluated as 12.4 MPa.

Figure 3 shows a typical relationship between the friction coefficient and the sliding displacement observed during a loading test carried out prior to the shaking test. Figure 4 illustrates the relationship between the friction coefficient and the sliding velocity during the shaking table test using the Kaihoku earthquake record. The friction coefficient increases as the sliding velocity increases. The solid line in the figure indicates the approximation formula proposed by M. Constantinou et al[3]. The average friction coefficient at low velocity range and high velocity range of the bearing were found to be 8% and 20% respectively.

Figure 5 depicts the rubber restoring force device, which consists of a rubber block and an anchor bar. Figure 6 indicates the force-displacement relationship of the rubber restoring force device obtained from a loading test. The device works as a horizontal spring within a small displacement range, and serves as a displacement restrainer when the displacement approaches a certain limit. However, the device did not reach the limiting displacement during the present shaking table test. The natural period evaluated from the weight of the deck (390 kN) and the stiffness of the device (1.32 kN/cm) is 2.44 seconds.

Two earthquake records (Kaihoku and Hachirougata) and two artificial design motions (Japanese Level 1 and level 2 earthquake motions on ground condition II - stiff soil) were used in the test. As shown in their response spectra in Fig. 7, these motions have different intensities and frequency contents. Due to the limited displacement capacity of the shaking table, however, it was not possible to use the earthquake motions with large intensities for the shaking table test. Therefore, the Hachirougata and Level 2 motions were used after scaling them down linearly by a factor of approximately 1/2 to 1/3 respectively. The shaking was applied only in the longitudinal direction.

Test Results

Table 1 lists the maximum values of the normalized shear force, deck acceleration, bearing displacement and permanent displacement of the model bridge under different earthquake motions. Permanent displacements were observed only under the Kaihoku and Hachirougata motions. Maximum response except for the permanent displacement occurred under the Kaihoku 0.544g input motion. Figure 8 plots a typical set of time histories of various responses and the force-displacement relationship of the isolation system under the Kaihoku 0.544g motion. The maximum deck acceleration is 0.244g which is much smaller than the table acceleration (0.544g) due to the isolation effect. The pier acceleration reaches a maximum value of 1.158g because of the pier reaction to the initiation of sliding, but this does not affect the pier shear force. In fact the corresponding shear force normalized by the deck weight is only 0.254 which is almost equal to the normalized inertia force of the deck. The maximum sliding displacement is 3.43 cm, but the permanent displacement in this case is almost zero. The maximum permanent displacement of 0.379 cm occurred under the Kaihoku 0.184g motion.

Figure 9 shows the maximum values of the pier acceleration, deck acceleration, normalized shear force of the pier, and bearing displacement as functions of the maximum table acceleration. The pier acceleration and bearing displacement become larger as the table acceleration increases. However, the deck acceleration and normalized shear force of the pier remain constants at their respective maximum values beyond the table acceleration of 0.2g, regardless of the increase of table acceleration. This is the unique and significant advantage of the sliding base isolation system as applied to bridges. The maximum deck acceleration is 0.22g corresponding to the friction force plus the restoring force.

NUMERICAL SIMULATION

Analytical Model

The analytical model depicted in Fig. 10 is used to simulate the shaking table test. The discontinuous function $sgn(\dot{u}_d)$ in the governing equations of motion (1) and (2) is replaced by the analytical expression (3)[4] for approximation,

$$m_d(\ddot{z}+\ddot{u}_p+\ddot{u}_d)+c_d\dot{u}_d+k_du_d=-sgn(\dot{u}_d)\mu m_dg \tag{1}$$

$$m_p(\ddot{z}+\ddot{u}_p)+c_p\dot{u}_p+k_pu_p-c_d\dot{u}_d-k_du_d=sgn(\dot{u}_d)\mu m_dg \tag{2}$$

where $\ddot{z}$ is the ground acceleration (table acceleration), u_p is the displacement of pier relative to ground, u_d is the displacement of deck relative to pier (bearing displacement), and μ is the friction coefficient.

$$sgn(\dot{u}_d)=\frac{1-\exp(-\delta\dot{u}_d)}{1+\exp(-\delta\dot{u}_d)} \tag{3}$$

where δ is a parameter to define the shape of the function $sgn(\dot{u}_d)$ in approximation (4.0 sec/cm is used in this analysis).

The friction coefficient μ is evaluated in Eq. (4) as a function of the sliding velocity $\dot{u}_d$.

$$\mu = \mu_{max} - (\mu_{max} - \mu_{min}) \cdot \exp(-a \cdot |\dot{u}_d|) \tag{4}$$

where a is a parameter defining the relationship between the friction coefficient and sliding velocity as shown in Fig. 4 (0.2 sec/cm in this analysis), μ_{max} is the friction coefficient in low velocity range (8% in this analysis, and μ_{min} is the friction coefficient in high velocity range (20% in this analysis).

Simulation Results

Newmark's β method is used in the dynamic response simulation. Figure 11 compares the time histories obtained from the simulation and the test. These time histories represent sliding displacement and deck acceleration, together with the force-displacement relationship of the isolation system under the Kaihoku and Hachirougata ground motions. In both cases, the simulation and the test produced almost the same peak response values and similar time histories. Table 2 compares these peak response values. It is important to observe that both peak deck acceleration and peak bearing displacement are very similar, since this means that the proposed analytical model represented by Eqs. (1) - (4) can be reliably used in evaluating the maximum values of the key response quantities, thus it is useful in the design procedure of bridges with sliding isolation systems.

CONCLUSIONS

The following observations were made through the experimental and analytical study of a sliding-isolated bridge.

1) The deck acceleration and pier force of the bridge isolated by the sliding system is limited to constant values regardless of intensities of input ground acceleration, even when the deck is supported by flexible piers. Therefore the advantage of sliding isolation system is demonstrated.
2) There was practically no residual displacement in the sliding isolation system after each earthquake.
3) An analytical method based on the direct integration procedure using the continuous mathematical formula representing sliding behavior is proposed and utilized. The accuracy of this procedure is confirmed through the comparison between experimental results and analytical simulation.

ACKNOWLEDGMENT

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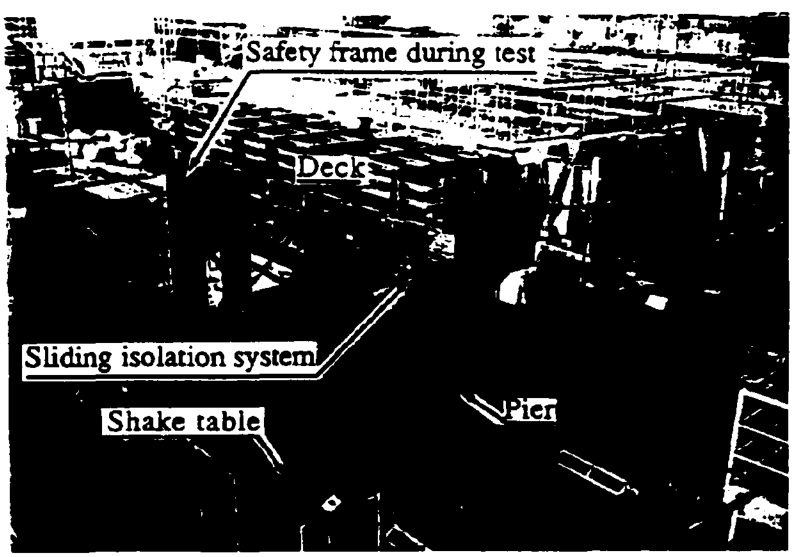


Figure 1 Bridge model

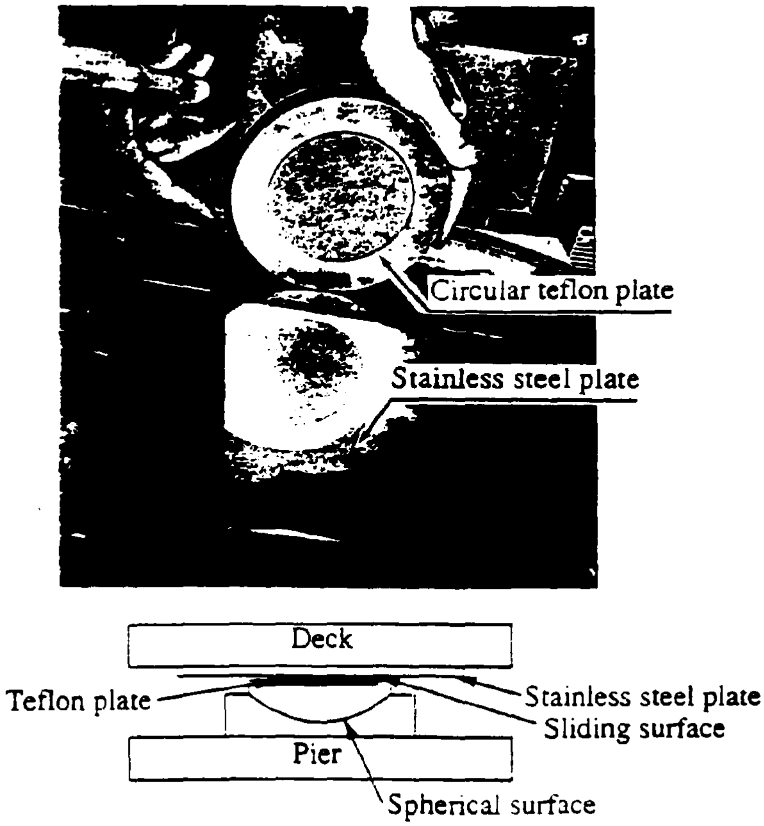


Figure 2 Detail of sliding bearing

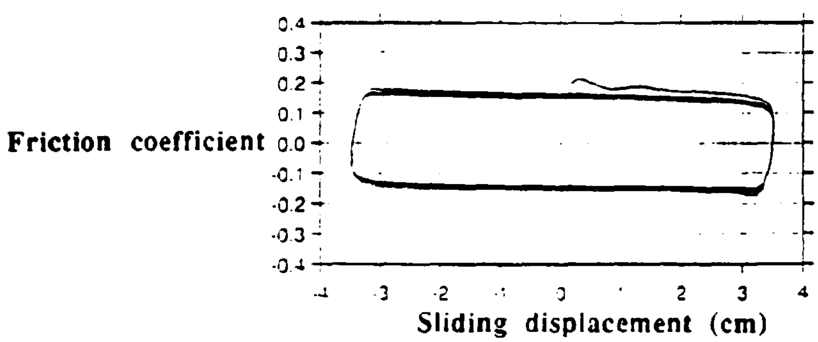


Figure 3 Example of identification test
(Maximum sliding velocity : 20cm/sec)

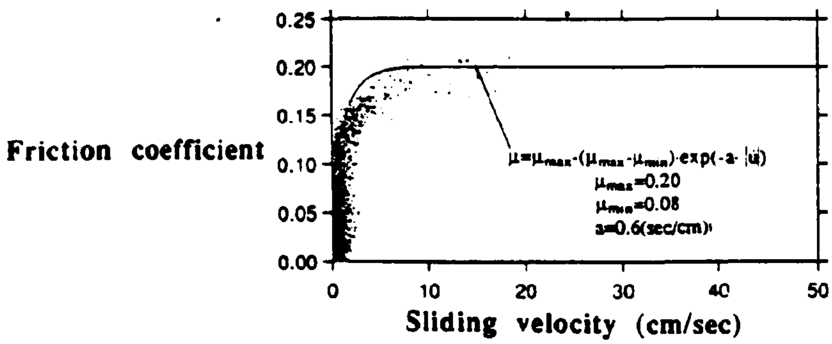
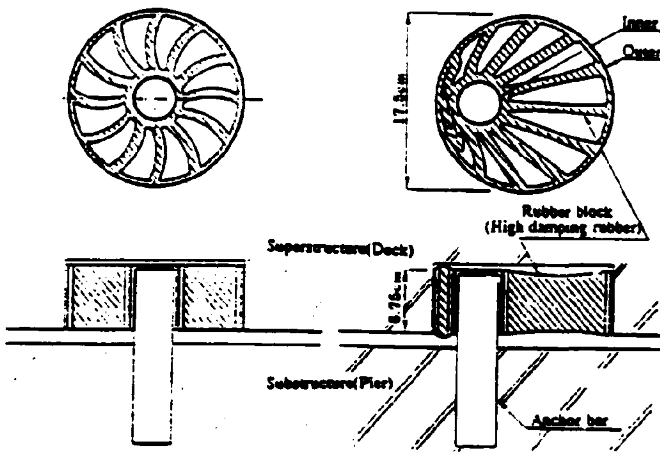


Figure 4 Realtionship between sliding velocity and friction coefficient



Numbers in this figure indicate size of rubber restoring force device used in the test

(a) Configuration of rubber device (b) Deformed rubber device

Figure 5 Rubber restoring force device

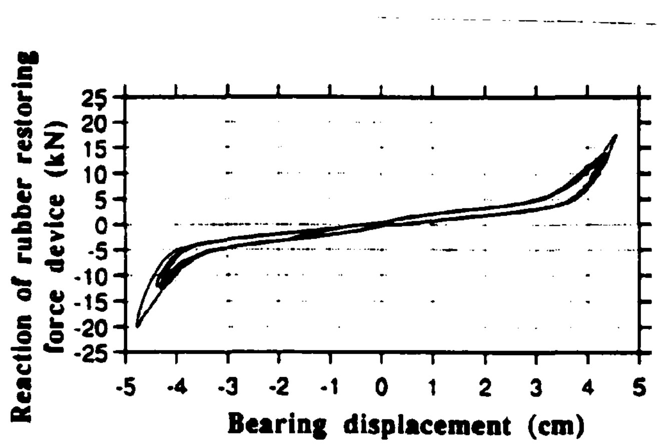


Figure 6 Force displacement relationship of rubber restoring force device

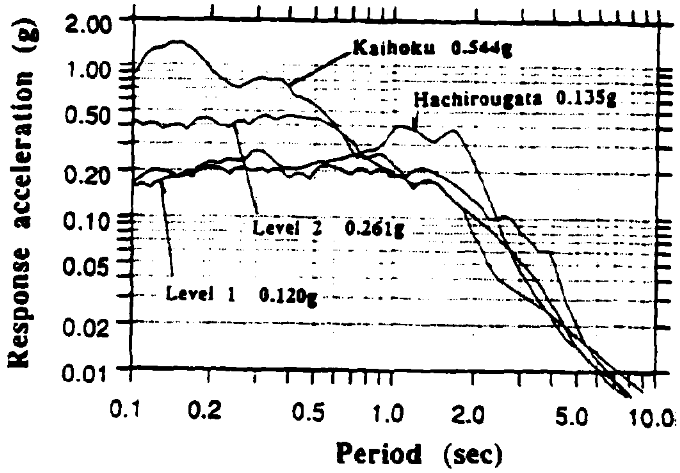


Figure 7 Resoponse spectra of input motions

Table 1 Maximum responses of model bridge during shake table test

Input Motion	Table Acceleration (g)	Normalized Shear Force	Deck Acceleration (g)	Bearing Displacement (cm)	Permanent Displacement (cm)
Kaihoku	0.087	0.116	0.126	0.120	0.077
	0.184	0.208	0.203	0.546	0.379
	0.183	0.178	0.190	0.600	0.242
	0.255	0.230	0.217	1.041	0.150
	0.250	0.207	0.212	0.970	0.044
	0.432	0.254	0.220	2.636	0.214
	0.428	0.245	0.216	2.741	0.060
	0.426	0.254	0.217	2.780	0.055
	0.489	0.247	0.218	3.027	0.091
	0.543	0.248	0.220	3.341	0.065
	0.544	0.254	0.224	3.425	0.003
Hachirougata	0.043	0.056	0.065	0.040	0.008
	0.084	0.120	0.130	0.095	0.006
	0.115	0.156	0.161	0.241	0.036
	0.135	0.181	0.181	0.487	0.105
Level 1 Ground Condition II	0.078	0.152	0.158	0.151	——
	0.121	0.185	0.187	0.345	——
	0.120	0.152	0.166	0.338	——
Level 2 Ground Condition II	0.180	0.215	0.206	1.035	——
	0.270	0.231	0.207	1.326	——
	0.265	0.223	0.216	1.286	——
	0.261	0.222	0.214	1.502	——

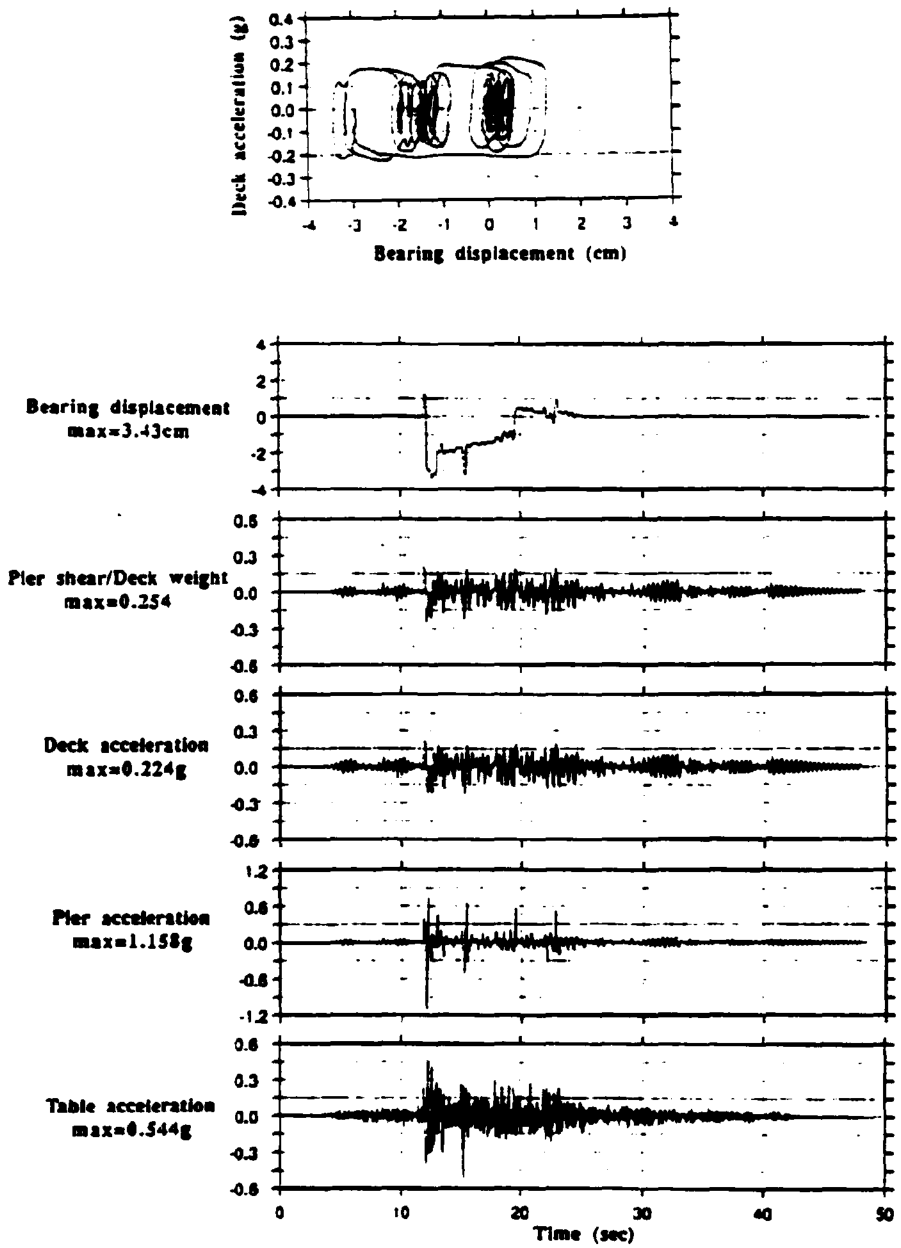


Figure 8 Response of base-isolated bridge with sliding system (Input motion : Kaihoku 0.544g)

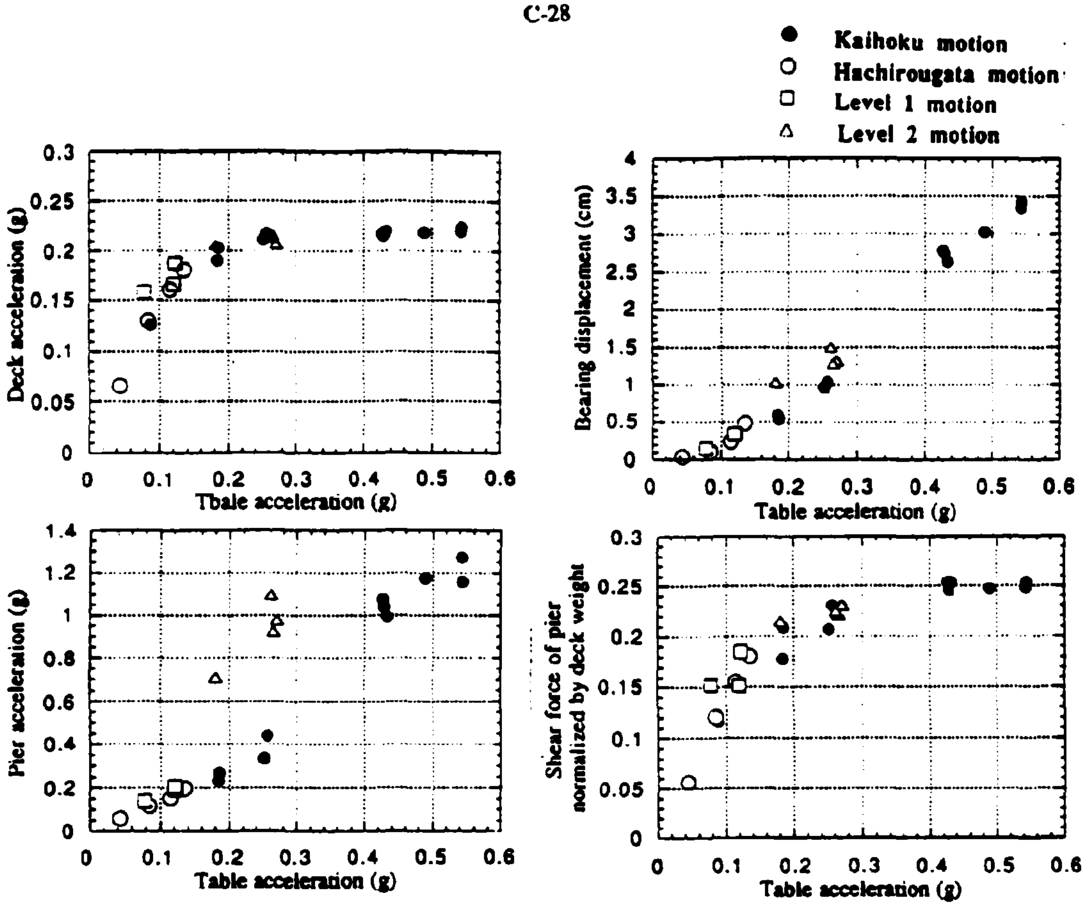
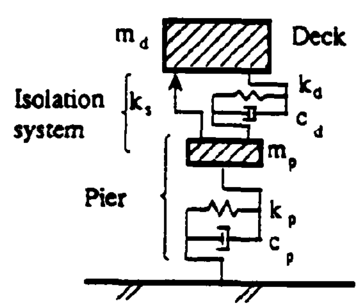


Figure 9 Maximum response of model bridge

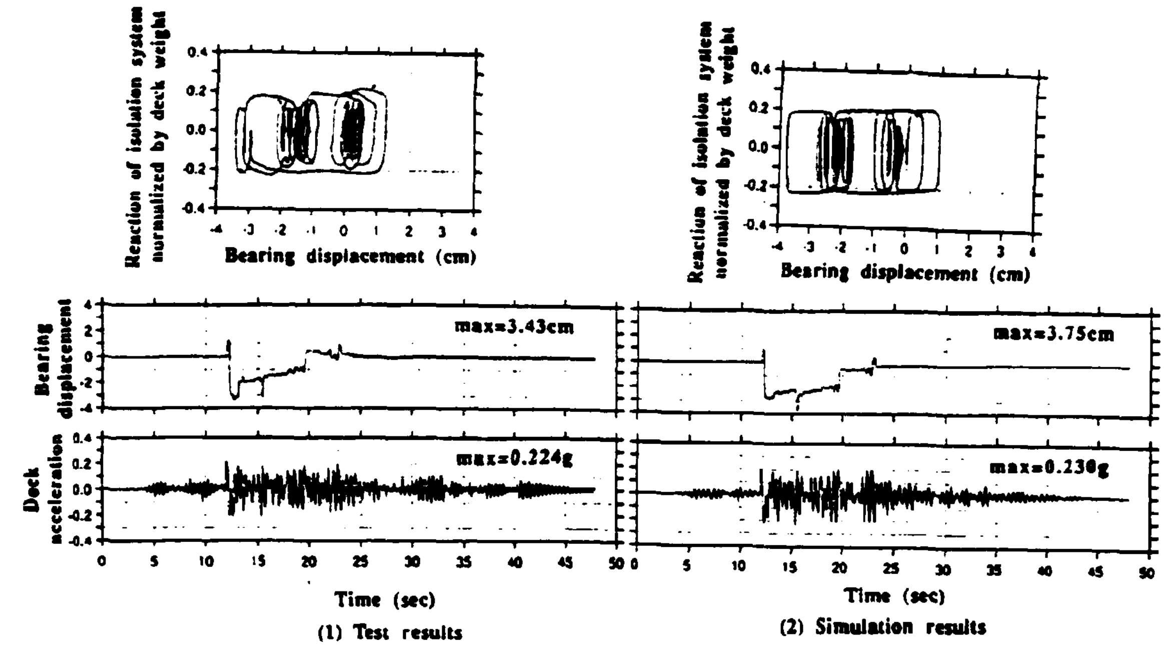


m_p : Mass of pier
m_d : Mass of deck
k_p : Linear spring of pier
k_d : Linear spring of isolation system
c_p : Dashpot of pier
c_d : Dashpot of isolation system
k_s : Non-linear spring of the sliding bearing

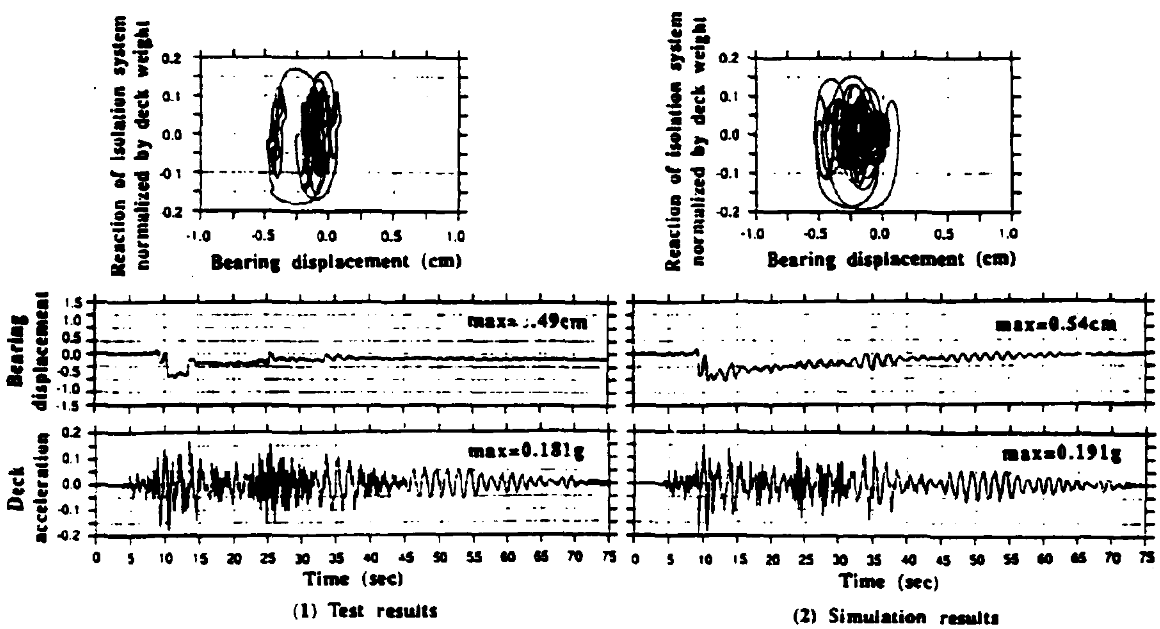
Figure 10 Simulation model

Table 2 Comparison of simulation results with test results

Input Motion	Table Acceleration (g)	Deck Acceleration (g)			Bearing Displacement (cm)		
		Test Results	Simulation Results	Simulation / Test	Test Results	Simulation Results	Simulation / Test
Kaihoku	0.087	0.126	0.116	0.92	0.120	0.057	0.48
	0.184	0.203	0.202	0.99	0.546	0.389	0.71
	0.183	0.190	0.202	1.06	0.600	0.392	0.65
	0.255	0.217	0.210	0.97	1.041	1.086	1.04
	0.250	0.212	0.210	0.99	0.970	1.083	1.12
	0.432	0.220	0.221	1.00	2.636	2.782	1.06
	0.428	0.216	0.221	1.02	2.741	2.801	1.02
	0.426	0.217	0.221	1.02	2.780	2.777	1.00
	0.489	0.218	0.226	1.03	3.027	3.311	1.09
	0.543	0.220	0.230	1.05	3.341	3.733	1.12
	0.544	0.224	0.230	1.02	3.425	3.754	1.10
Hachirougata	0.043	0.065	0.064	0.98	0.040	0.051	1.28
	0.084	0.130	0.133	1.02	0.095	0.130	1.37
	0.115	0.161	0.165	1.03	0.241	0.249	1.03
	0.135	0.181	0.191	1.06	0.487	0.544	1.12
Level 1 Ground Condition II	0.078	0.158	0.156	0.99	0.151	0.145	0.96
	0.121	0.187	0.202	1.08	0.345	0.450	1.30
	0.120	0.166	0.201	1.21	0.338	0.458	1.36
Level 2 Ground Condition II	0.180	0.206	0.208	1.01	1.035	0.923	0.89
	0.270	0.207	0.211	1.02	1.326	1.546	1.17
	0.265	0.216	0.211	0.98	1.286	1.555	1.21
	0.261	0.214	0.211	0.99	1.502	1.517	1.01
			Average	1.02		Average	1.05



(a) Input motion : Kaihoku 0.544g



(b) Input motion : Hachirougata 0.135g

Figure 11 Results of simulation

APPLICATION OF SEISMIC ISOLATION FOR NEW BUILDINGS
- EXPERIENCE IN THE COUNTY OF LOS ANGELES -

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SUMMARY

Base isolation has been proven to be an effective means of mitigating earthquake induced hazard to buildings. Many new and existing buildings in the United States have used the system. Records from the most recent January 17, 1994, Northridge earthquake again showed the beneficial effect of base isolation on buildings in the Los Angeles area. Although the shake was not strong enough in these buildings to test the ultimate performance of the bearing, records show a significant reduction of ground accelerations in these buildings.

Four facilities owned by the County of Los Angeles used seismic isolation technology. The authors of this paper have been involved in the planning and review of these projects. Lessons learned are summarized to assist future projects that may consider using the seismic isolation technology. Issues that affected the application of the isolation technology in the United States are discussed.

INTRODUCTION

In the County of Los Angeles, the four buildings which have used seismic isolation technology are:

County Fire Command and Control Facility
County MLK/Drew Medical Center Trauma Center
County Emergency Operation Center
County/USC Medical Center Diagnostic and Treatment Building

The location of these new buildings and the epicenter of the January 17, 1994, Northridge earthquake are shown on Figure 1. Basic information about these facilities is shown in Table 1.

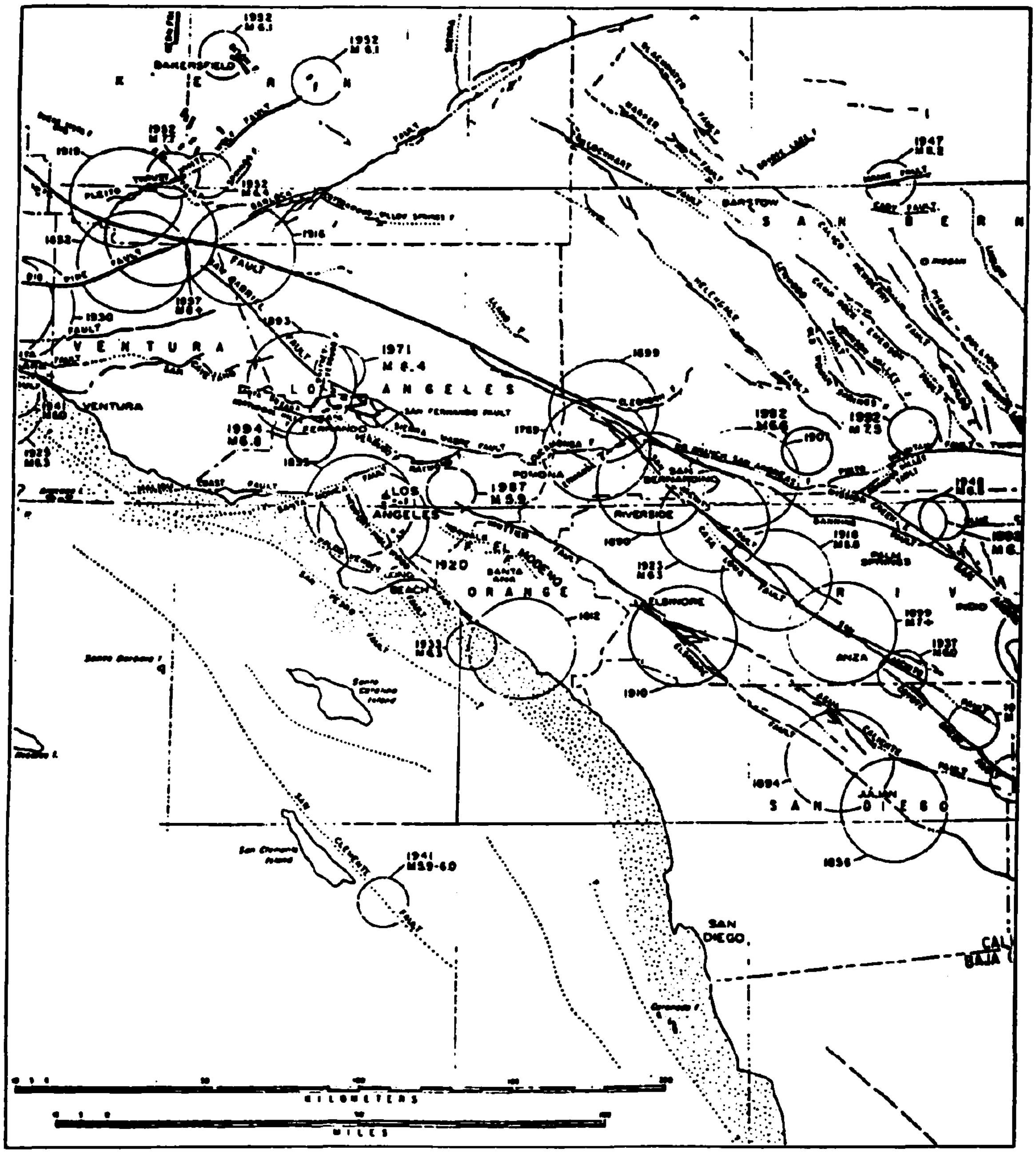

Figure 1: Map of Los Angeles Area

Table 1 : Base Isolated Facilities Owned by County of Los Angeles

	Los Angeles County Fire Command and Control Facility(FCCF)	**Los Angeles County Emergency Operation Center(EOC)**	**Los Angeles County MLK/Drew Med. Center Trauma Building**	**Los Angeles County/USC Med. Center Diagnostic and Treatment Building**
Total floor area	30,000 sf.	30,000 sf.	150,000 sf.	500,000 sf.
No. of Levels	2	2	6 above grade, one below grade	split levels 6 and 7
Structure type	Steel braced frame	Steel braced frame	Steel braced frame above grade, concrete shear wall below grade	Steel ordinary moment frame
Structural consultant	Flour Daniel	Daniel Mann Johnson Mendenhall (DMJM)	John A. Martin and Associates (JAMA)	KPFF
Isolation consultant	Flour Daniel	DMJM	Base Isolation Consultant (BIC)	KPFF
Geotechnical consultant	Woodward/Clyde Consultants	Law/Crandall	Law/Crandall	Dames and Moore
Peer review panel	James Kelly and others	Bill Holmes, James Beck, Charles Kircher	Tom Anderson, Roland Sharpe, Charles Kircher, George Linkletter	Alex Tarics, James Kelly, Bob Bachman, and Charles Kircher* *(County consultant)
Bearing type and supplier	High damping rubber with ultimate restrain chain by Fyfe Associates	High damping rubber by Bridgestone	High damping rubber by Dynamic Isolation System	High Damping Rubber by BTR/Silver Stone

Table 1 (cont.): Base Isolated Facilities Owned by County of Los Angeles

	Los Angeles County Fire Command and Control Facility(FCCF)	**Los Angeles County Emergency Operation Center(EOC)**	**Los Angeles County MLK/Drew Med. Center Trauma Building**	**Los Angeles County/USC Med. Center Diagnostic and Treatment Building**
Bearing fabricater	Dynamic rubber, Athens, Texas	Bridgestone, Yokohama, Japan	Furon, Athens Texas	BTR/Andrew, England
No. of bearings	32	28	70 bearings + 12 sliders	155 (115x39"ϕ + 40x44"ϕ)
Gross Size of bearings	18" sq. x 14"H w/4.5"ϕ hole	616mm ϕ x 341.8mm H	40"ϕ x 22"H	39"ϕ x 19"H + 44"ϕ x 19"H
MCE design displacement	9.6"	15.7"	20"	20"
Building separation	No constraint (chain engage at 15")	16"	24"	20"
Design base shear (MCE)	.15g	.40g	.30g	.24g
Current status	Completed in 1989	To be completed in September 1994	To be completed in 1995	Complete design in June 30, 1994

CONSIDERATIONS IN THE PLANNING PHASE

The consideration of using isolation technology started at the planning stage. A feasibility study is generally the first step in the decision making process. This study should present all relevant facts about the seismic isolation technology and its potential impact on the project. Included in the feasibility study should be an implementation plan. This plan should detail all the elements and sequences to be followed for the work. With this information, the owner could then decide how base isolation could benefit a specific project.

Feasibility Study

The feasibilty study needs to address the following general issues: cost of design, test and construction, design and construction schedule, confidence level of the owner and architect. In addition, the area seismicity, building configuration, available space on the site, etc., should also be addressed. The following list identifies elements of a complete feasibilty study:

1. General considerations

- Building location and area seismicity
- Regulations or Codes applicable to project
- Function of the facility
- Preliminary building configuration and potential change in the building configuration
- Estimate impact to project budget
- Estimate impact to construction schedule
- Availability of stand-alone utilities sources and space for storage
- Availability of suppliers and testing facilities

2. Detailed considerations

Advantages and disadvantages of using the isolation system should be compared. Any comparison should not only compare fixed base structure vs. isolated structure. It needs to be recognized that the current Building Code in California requires different levels of design ground force for isolated buildings. Therefore, at least two fixed base schemes should be considered for any study. One is a fixed base building designed to building code. The other is the same fixed base building designed to meet the isolation code provisions, using generally higher ground motion. This difference should be recognized between fixed base and isolated structure. The following is a list of items to be addressed:

Disadvantages of using base isolation:

- Cost of additional site work including excavation of the pit, retaining wall around building, etc. and access for observing the isolators
- Cost of implementing the isolation system including design, procurement, test and construction
- Cost of the first floor structure

- Cost of providing secure source of utilities such as water, power, and communications equipment, and flexible connections for continued operation of the building.
- May have to modify the building configuration to eliminate excessive torsion
- Longer design time and additional coordination efforts
- Longer plan check time by regulatory agencies

Advantages of using base isolation:

- Life-cycle cost savings considering that the building will experience an appropriate number of different level earthquakes during its expected life time
- Higher level protection of building, contents, occupants and the assurance of continued operation of the building over its life after both major and minor earthquakes
- Cost of anchorage and support of non-structural will be lower.
- Lower superstructure framing cost

The life-cycle cost analysis is the most important factor in the study and requires thorough understanding of building operation.

3. Conclusions including life-cycle cost comparisons and recommendations

Finally, a conclusion should be made focusing on the life-cycle cost comparisons.

Implementation Plan

An implementation plan should also be part of the feasibility study. After the decision to use base isolation was made by the owner, the implementation plan provides the elements and procedures to complete the project. This plan should include the following:

- Determination of the design ground motion
- Static or dynamic analysis: Current building codes used in California allow static analysis for regularly shaped buildings
- Selection of a special consultant with base isolation expertise and experience
- Selection of a Peer Review Panel:
  - Direct experience
  - Knowledge of various systems
  - Knowledge of owners objective for the facility
  - Communication skill
- Selection of isolation system:
  Comparison of different systems:
  - Stiffness
  - Sensitivity to uncertainties associated with the prediction of design ground motion
  - Sensitivity to wind induced vibration
  - Sensitivity to ambient ground borne vertical vibration
  - Past performance record of the proposed elastomeric compound or compounds

  - Past performance record of the fabricator
  - Impact of base isolator stiffness on the design of superstructure
  - Proprietary or generic system
- Procurement process:
  Competitive bidding or single source procurement
- Fabrication quality assurance
  Prototype test and Production test
- Maintenance
  Test of selected bearings after major events
- Instrumentation
  To gain information on base isolator performance in earthquakes

ADDITIONAL CONSIDERATIONS

Performance expectations

The structural engineers should inform owners and architects about the potential consequences or hazards associated with many of the decisions they make, including their dependence on the Uniform Building Code(UBC). Owners and architects should be informed that the UBC provisions for fixed base building is to protect the life safety of the occupants. The provisions for the base isolation design offer not only better life safety protection but protection of the loose contents of the buildings as well. Consequently base isolation will significantly increase the possibility of continuing operation of the buildings. Lessons learned from the 1994 Northridge earthquake revealed that most people underestimate the damage caused by earthquakes. Although many buildings did not suffer major structural damage, costs of clearing, business disruption, repairing, etc. far exceeded expectation.

It was reported *[Los Angeles Times, January 20, 1994]* that "2,600 hospital beds were lost in the recent Northridge earthquake, not because buildings collapsed, this happened also, but because of the "melt down" of the mechanical/electrical systems: water mains burst, lines carrying oxygen into hospital broke, elevators went out along with power supplies, auxiliary generators failed,..." There was also similar experience around San Francisco area after the 1989 Loma Prieta earthquake.

Function is the most important reason for building a building. The architect should be educated about the philosophy of resisting earthquake forces before he starts to design a building. Only if the architect, the structural engineer, and the owner, working together as a team, reaching a consensus to reduce earthquake damage and preserve the ability of the building to function, can a facility reach its optimum performance.

Building configuration

Building configuration is an important factor affecting the earthquake performance of a building. In the United States, structural engineers generally act as consultants to the architect, who is the primary consultant to the owner. Very often in the facility planning stage, the focus is on the functions and the rather subjective aesthetic aspect of the building. The input of the structural consultant at this stage is limited. Building configuration is generally determined before earthquake resistance is considered. The building configuration is very important to base isolated buildings because it significantly affects the performance of the building during earthquakes.

Non-structural elements

These are parts of the building that receive their support from the structure and are attached to the structure. The exterior skin of the building, windows, parapets, elevators, certain stairs, the space dividing interior partition walls, suspended ceilings, mechanical and electrical equipment, ventilating pipes, ducts and wiring, etc.

In conventional design/construction practice, structural engineers focus more on the performance of the structural systems, and less on non-structural elements. In the US, structural engineers input to the arrangement and support of non-structural elements is limited. In addition, many anchorage and support systems for non-structural elements are only outlined on the design drawings and are specified as design/supply/construction packages to be furnished by the contractor. Because most construction projects require competitive bidding, the design architect/engineer cannot possibly provide details of installation for all bidders who provide different type equipment or systems. Therefore, it should be required that the construction inspection of all non-structural elements be performed by professionals knowledgeable about seismic resistant design.

Guesstimate of design earthquake force

1. Uncertainty in ground motion assessment:

Ground motion design criterion is one of the most important factors and controversial issue in base isolation design.
Current state-of-art ground motion determination is using the probabilistic method rather than the deterministic method. Whether one method provides better data than the other is beyond the purpose of this paper. Considering the relatively short history of this branch of science and the few facts we know about earthquakes, it is important to understand the degree of uncertainty associated with ground motion prediction. It should be realized that although the methodology used in either method is scientific, the input could be highly speculative. Therefore, the structural engineering profession should use its judgment in utilizing these data for design applications.

2. Analysis procedures:

The current Building Code requires time history analysis for the design of base isolated buildings. The analytical process is very time consuming to say the least. Considering the high uncertainty of the given ground motion, we question if such procedure, that provides more precision than accuracy, is justified. Any simplification of the analysis procedure, taking into account the uncertainty of the ground motion assessment, will encourage more engineers and owners to consider base isolation technology.

It was recently revealed that there are many previously unknown hidden thrust faults underneath the Los Angles area. These hidden faults are capable of generating higher earthquake forces than previously predicted. Previous seismic research focused on slip-strike faults that could be identified with surface ruptures. It was suggested that a uniform response spectrum be prepared for areas such as the San Fernando Valley and the Los Angeles basin. Such a spectrum could be a useful tool for evaluating survivability and potential damage level of existing buildings at different locations. It could also be used as the basis for the planning of an upgrading program for existing buildings.

CONCLUSIONS

Seismic isolation technology offers much higher degree of earthquake protection than conventional fixed base buildings. As many building owners now demanding higher level of protection, the technology will gain more strength in the future. More tests will establish additional confidence and may simplify the design and selection process.

Testing of Natural Rubber Isolation Bearings for UNIDO Demonstration Building at Shantou City, P.R. China

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SUMMARY

An eight-story housing block has been built in Shantou City, Guangdong Province, P.R. China using a natural rubber multi-layer isolation system. The purpose of this project is to demonstrate the application of this technology to public housing in developing countries. A number of such demonstration buildings are completed or under construction around the world using this innovative seismic-resistant design approach. The design and testing of the isolators for this project were carried out with partial support from the United Nations Industrial Development Organization. This paper describes in detail the types of elastomeric bearings used to isolate the demonstration building in Shantou City and the test series conducted at the Earthquake Engineering Research Center of the University of California at Berkeley to determine the characteristics of these elastomeric bearings.

INTRODUCTION

The application of base isolation technology to create earthquake-resistant structures is a radical departure from the traditional approaches used by structural engineers. In conventional fixed-base design, strengthening a structural system to provide superior seismic performance leads to a stiffer structure which attracts more force to the structure and its contents; a fixed-base building tends to amplify the ground motion. To minimize this amplification, the structural system must either be extremely rigid or incorporate high levels of damping. At best, rigidity leads to the contents of the building experiencing the ground accelerations which still may be too high for sensitive internal equipment and contents. Incorporating high levels of damping in a structural system means either damage to the system in the event of a major earthquake or designing an expensive structural form to mitigate this damage.

When a building is built on an isolation system, it should have a fundamental frequency that is lower than both its fixed-base frequency and the dominant frequencies of the ground motion. The first mode of the isolated structure then involves deformation only in the isolation system, the structure above being almost rigid. The higher modes which produce deformation in the structure

One demonstration project under this program is a base-isolated apartment dwelling in Shantou City, Guangdong Province, P.R. China. Completed in 1994, this building is the first rubber base-isolated building in China. The purpose of this report is to evaluate the mechanical properties of the high-damping natural rubber bearings used in this project.

DEMONSTRATION PROJECT IN SHANTOU, CITY, P.R. CHINA

The UNIDO demonstration project in P.R. China involves the construction of two eight-story housing blocks in Guangdong Province, an earthquake-prone area of southern China. Two identical and adjacent buildings were built in the coastal city of Shantou, formerly called Swatou. One building is of conventional fixed-base construction and the other is base-isolated using high-damping natural rubber isolators. The design, testing and manufacture of the isolators was funded by MRPRA from a grant provided by UNIDO. The demonstration project is a joint effort by MRPRA, EERC and Nanyang University, Singapore.

The demonstration building is the first building in China to use elastomeric bearings and is intended to be an example of this new method of earthquake-resistant design. The success of this project is crucial for the widespread use of base isolation technology in developing countries. Because there is a large demand for housing in highly seismic regions of China, this project aims to provide safe and affordable housing.

The design of the fixed-base building complied with the current Chinese earthquake-resistant design codes. According to this code, the Standard Design Earthquake (SDE) is an earthquake with a 10% probability of being exceeded in 50 years, and the performance of the building should be evaluated for what is equivalent to a Maximum Capable Earthquake (MCE) defined as the standard earthquake scaled by a factor of 2, representing 2% probability of being exceeded in 50 years.

The isolation system for the building was designed to have a damping ratio of 10% with a period of 2.0 sec, which differs significantly from the 0.5 sec fundamental period of the fixed-base frame structure. Under the SDE, the superstructure has to remain elastic and the resulting displacement is 120 mm (4.72 in). Under the MCE, the building should perform without failure and the resulting maximum displacement is 240 mm (9.45 in). See Ref. [2] for details.

The eight-story reinforced concrete building structure is supported by 22 columns with a plan dimension of 29 m × 16 m (88.4 ft × 48.8 ft). According to the column load distribution, one set of bearings supports a mean column load of 954 kN (214 kips) and another set of bearings carries a mean column load of 1490 kN (335 kips). The two types of bearings are designed to have the same dimensions; this is achieved by using two different rubber compounds with different shear moduli. One bearing is located under each column, except for the heaviest column which is supported by two bearings comprised of the softer compound. The connection between the bearing and the structure is done using recess plates. A 20 mm- (0.79 in.) thick plate with a hole the diameter of the bearing is bolted to the foundation plate and the bearing sits within this hole. There is an identical recess plate with the same configuration at the top of the bearing.

SHANTOU/HUME TEST BEARINGS

The bearings used in the demonstration building in Shantou are comprised of two different types of rubber compound; type I is a soft compound and type II is a hard compound. The two different types of bearings have the same dimensions but different properties in order to be able to accommodate the variation of the column loads. These high-damping natural rubber compounds were developed by MRPRA for this project and are filled with carbon black, thus they have a significantly lower shear modulus, yet retain the lost factor and the elongation to break. The high-damping characteristics of the rubber resulted in low-cost, lighter and more stable bearings, even under low vertical pressure.

The type I bearings are made of a soft compound with 0.50 MPa (72.52 psi) shear modulus at 100% strain, while the type II bearings are made of a hard compound with 0.79 MPa (114.58 psi) shear modulus at 100% strain. The bearings are circular with a shape factor, $S = \phi/4t$, where ϕ is the diameter of the bearing and t is the thickness of individual rubber layer. Here S=10 is kept moderate so that the vertical frequencies of the bearings are low.

The dimensions of the test bearings, which were manufactured by Hume Industries of Kuala Lumpur, Malaysia under the supervision of Dr. C.T. Loo to MRPRA specifications, are shown in Fig. 1. The bearings consist of 8 layers of 15 mm- (0.59 in.) thick rubber with 7 steel shims, 3 mm- (0.12 in.) thick. There are two end plates, 20 mm- (0.79 in.) thick, which are covered with a layer of 4.5 mm- (0.18 in.) thick rubber for a total height of 190 mm (7.48 in). The shim diameter is 580 mm (22.83 in.) and there is a 10 mm (0.39 in.) of vertical cover for a total diameter of 600 mm (23.62 in.). There is a concentric guide pin, 50 mm (1.97 in.) in diameter, which was used as an aid in manufacturing the bearings and which was removed and the hole sealed later with the same rubber as in the rest of the bearing.

The connection between the bearings and the structure is shown in Fig. 1. Two recess plates were used to restrain the slip between the bearings and the structure. The recessed steel plates are 762 sq. mm (30 sq. in.), and 20 mm- (0.79 in.) thick, and were attached to the structure by eight bolts, 25.4 mm (1 in.) in diameter. The gap between the recess plates and the bearing was 3 mm (0.12 in.). During the experiment the recess plates were split into two, however, in the actual structure the recess plates were not split.

TEST FACILITIES

The testing program was carried out on a bearing test machine at the Earthquake Simulator Laboratory at EERC and is described as follows.

Bearing Test Machine

The bearing test machine is shown in Fig. 2. The test machine is capable of subjecting four bearings to simultaneous vertical and horizontal dynamic loading. The test machine is mounted on an elevated concrete base block and consists of three main parts: a base platen that supports two vertical actuators, a middle part that houses the horizontal actuators, and the upper loading beam that distributes the load from the vertical actuators to the bearings.

Two test bearings (soft and hard) were placed between the horizontal actuator housings and the base platen and the other two bearings (soft and hard) were placed between the horizontal actuator housings and the upper loading beam. Each bearing type was placed on one side of the testing machine, one above the other. During testing, the bearings were loaded to the required vertical load and then the testing machine was locked so that when one type of bearings was tested horizontally, the other set of bearings only carried axial load. Under this configuration, each horizontal actuator developed a maximum dynamic load of 667 kN (150 kips) in extension and 534 kN (120 kips) in retraction. The maximum displacement was ±477 mm (18 in.) (i.e., 914 mm (36 in.)/ stroke). The servo-valve on the horizontal actuator has a capacity of 5 gpm for loads less than 178 kN (40 kips) and 3 gpm for higher load levels.

The vertical load was applied through the upper loading beam which distributed the forces from the vertical actuators to the test bearings. Each vertical actuator, including the weight of the upper loading beam, has a maximum capacity of 3180 kN (715 kips) in compression and 4195 kN (943 kips) in tension. The servo-valve on the vertical actuator has a capacity of 25 gpm. The spaces to accommodate the test bearings can be lifted up to 508 mm (20 in.). The size of the pedestal plate is 965 mm (38 in.) square.

Instrumentation

A total of fourteen channels of data were recorded for the Hume/Shantou bearing tests using the double-bearing test configuration. Table 1 shows the list of the channels used in the experiment. The loads applied by the hydraulic actuators were measured by pre-calibrated load cells, and linear potentiometers were used to measure the corresponding displacements.

The compression load on the bearings was calculated by averaging the measured forces from two vertical actuators. It was assumed that both bearings (top and bottom) were under the same vertical load. The vertical actuators were under force control in order to keep a constant vertical load test independent of the displacement of the horizontal actuators. The differential displacement between the two vertical actuators was maintained at zero.

It was assumed that the top and bottom bearings would perform identically, thus the horizontal force was the average of the measured forces, and the differential displacement between the two horizontal actuators was maintained at zero. In general, to apply constant vertical loads to the test bearings, the horizontal actuators are under displacement control. Four direct current differential transformers (DCDTs) were used to measure the vertical displacement of the top bearing at the level of the upper loading beam to ensure that the loading beam was kept horizontal. One linear potentiometer, used as a back-up information channel, was located at the back of the horizontal actuator. One extra channel was used to register time.

Data Acquisition and Control System

The Automatic Testing System (ATS) software package was used for data acquisition and control of the actuators. The software runs on a personal computer under Microsoft window environment. The software can control simultaneously up to four actuators and record sixteen channels of data with the additional capability of channel calibration and real-time display of the data. A special module in the ATS was developed for conducting this bearing test to control two independent vertical actuators and horizontal actuators. The sampling rate of the data was 200 data/sec.

TEST PROGRAM

Horizontal Tests

Each pair of bearings was subjected to an identical test program but with different levels of vertical pressure. Two sequences of horizontal displacement cycles were imposed on each pair of bearings. Each sequence included three cycles of displacement at each level of strain as follows:

Sequence 1: ±5%, ±25%, ±50%, ±75%, ±100%, ±150%

Sequence 2: ±200%, ±250%

Sequence 1 was repeated under the nominal vertical pressure. The second round of Sequence 1 was done immediately after the end of the first round of Sequence 1. Sequence 2 was carried out when the bearings were checked to insure that the two test programs of Sequence 1 had been completed without damage to the bearings. The constant velocity tests were carried out at the rate of 5 mm (0.2 in.)/sec.

Roll-Out Tests

After the dynamic test sequence was completed, the soft compound bearing was loaded monotonically at the same rate of 5 mm (0.2 in.)/sec to roll-out. Roll-out was assumed to be reached when the contact area between the bearing and the structure was less than 50% of the total area of the bearing. At this stage, the test was stopped even though the bearing did not lost its capability to carry more load. The complete test program is shown in Tables 2 and 3.

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 using the formula in Ref. [3].

The strain at the design earthquake level was specified to be 100%. The effective stiffness for the soft compound type I bearing under the design pressure was 87% of the nominal value for the first cycle, and reached 83% at the third cycle. However, at 100% strain level, the effective stiffness for the hard compound type II bearing was 116% of the nominal value and reached 110% at the third cycle. The combination of the variation of the effective stiffness of both compounds maintained the nominal horizontal natural frequency at 0.55 Hz.).

The horizontal force-displacement hysteresis loops for type I bearing with two vertical pressures are plotted for 5.64 MPa (818 psi) and 0.69 MPa (100 psi) in Figs. 3 and 4, respectively. The horizontal force-displacement hysteresis loop for the type II bearing with 5.64 MPa (818 psi) vertical pressure is shown in Fig. 5. As the strain increased, the hysteresis loops changed from being quite elliptical to being elongated with parallel sides and strong hardening. However, the strong hardening only occurred when the pressure was large enough to keep the bearing in place to prohibit any occurrence of up-lift.

The damping factor at the nominal vertical pressure and at the nominal design strain level of 100% was 7.5% for the type I bearing and 10% for the type II bearing. This is comparable with the nominal design damping factor of 10%. These damping factors decreased slightly in the higher strain levels up to the strain hardening strain level. The damping factors quoted are based on modeling the bearings as elastic and linear viscous elements. This model predicts that the energy dissipation is quadratic in displacement (see Ref. [3]) and that the effective stiffness is independent of the displacement. However, as shown in Figs. 6 and 7, the energy dissipation is not quadratic but varies roughly as displacement to the power 1.65, while the effective stiffness increases at the higher strains; both factors act to reduce the damping factor. The most important aspect of the bearing behavior is that the energy dissipation continues to increase for the higher levels of strain for type I bearing and for type II bearing. The hardening of the elastomers would eliminate the possibility of resonance response.

Since the effective stiffness of the type I bearing is lower than the nominal value, the effective shear modulus derived from the effective shear stiffness was 93% of the nominal value. However, the effective shear modulus for the type II bearing was 124% of the nominal value. Thus the effective shear modulus decreased for higher strain levels until strain hardening occurred when the strain level went beyond 200%.

Influence of Axial Pressure on the Dynamic Properties

The nominal pressure on the bearings used for the Shantou demonstration building was 3.61 MPa (524 psi) for the type I bearings and 5.64 MPa (818 psi) for the type II bearings. The average pressure generally used for the elastomeric bearings are in the range of 5 to 7 MPa (700 to 1000 psi). The bearings have a moderate shape factor of 10 and a low height-to-width ratio of 0.2 so that the stability and the dynamic properties would not be sensitive to vertical load.

The dynamic tests for the type I bearings were carried out at four levels of vertical pressure, 0.69, 3.61, 7.23 and 10.98 MPa (100, 524, 1048 and 1572 psi). The tests for the type II bearings were carried out at three levels of vertical pressure, 0.69, 5.64 and 10.34 MPa (100, 818 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 shown in Fig. 6 for the type I bearings, and in Fig. 7 for the type II bearings. The stiffnesses of the type I bearings for a strain less than 150% with 0.69 MPa (100 psi) vertical pressure and pre-maximum displacement test with nominal vertical pressure are higher, but the effect is small and it can be ignored; above 150% strain the effect of the pressure is very small. The stiffnesses of the type II bearings are almost constant for the various pressures.

The pressure has a very definite effect on the damping. The enclosed area of the hysteresis loops for fixed-strain increases with increasing pressure leading to higher damping factors. The damping factors for each pressure level and each peak strain level are computed using the formula developed in Ref. [3] and are shown in Fig. 6 for the type I bearing and in Fig. 7 for the type II bearing. The damping factors for the type I bearing vary very little if the pressure is at nominal value or less. However, if the pressure is double or triple the nominal value, the damping factor increases by 50% to 100% for all strain ranges. The type II bearings, in general, had higher damping factors; the damping factor also increased under higher pressure and varied very little over the strain range of 100% to 200%. The fact that the damping factor varied in the range from 7% to 12% for the soft compound and from 10% to 12% for the hard compound for the strain up to 200% would encourage the production and the use of high-damping natural rubber to isolate structures.

Roll-Out Test Results

The connection between the bearing and the structure was done by using recess plates. Roll-out is defined when the contact area between bearing and the structure becomes less than half of the area of the bearing, even though the horizontal shear stiffness remains positive.

Roll-out tests for the type I compound were carried out at a nominal pressure of 3.61 MPa (524 psi). The test was stopped at a strain level of 368% (17.4 in.). At this stage, the horizontal force was 523 kN (128.7 kips) and the tangent stiffness was positive, namely, the bearing was still capable of carrying additional loading. At an axial pressure of 0.17 MPa (25 psi), the test was stopped at a strain level of 182% (8.6 in.); the horizontal force was 136 kN (30.5 kips) and the tangent stiffness was also positive. The shear force-displacement path was independent of the axial pressure, as shown in Fig. 8, where shear force-displacement curves from both roll-out tests were plotted using the same scale. It is expected that the bearing will exhibit similar behavior in a structure.

Roll-out tests for the type II hard compound could not be carried out because at small axial pressure, i.e., 0.69 MPa (100 psi), the bearing started to experience up-lift and at a nominal pressure of 818 psi (5.64 MPa), the machine force capacity of 667 kN (150 kips) shear force was only enough to bring the strain level to 220%.

CONCLUSIONS

The test results indicate that the isolators have a very large inherent margin of safety and have been conservatively designed and reliably manufactured by Hume-Malaysia. The results attest to the high quality of both the design and manufacturing processes.

The best estimate of the maximum displacement demand on the isolators under Maximum Capable Earthquake loading is 240 mm (9.45 in.) or 200% strain. The tests have shown that the soft compound isolators can reach displacements of 440 mm (17.4 in.) corresponding to 368% shear strain. The hard compound isolators were tested to a displacement of 264 mm (10.4 in.) and 220% shear strain. These are the displacements at the maximum capacity of the test machine and there is good reason to assume that the maximum strain can be larger than this. The design of the isolator is such that the width-to-height ratio is very large and buckling for such isolators, even at large horizontal shear, is not important. Roll-out is also unlikely to be significant for these large flat isolators since roll-out would require a large increase in the gap between the top and bottom connection plates, which while possible in the test machine, is highly unlikely in the actual building.

The test results show that the horizontal stiffness of the isolators is unaffected by the level of vertical load, but the damping can be substantially increased by increasing the vertical load.

At strains that exceed the design level strains, the elastomer exhibits a strain hardening effect This will have the effect of reducing the displacements if an earthquake of unanticipated level occurs. At the highest level of cyclic shear strain, the maximum shear stress for the soft compound is 1.41 MPa (205 psi) and for the hard compound is 2.55 MPa (370 psi). The design pressure of the soft compound bearings is 3.61 MPa (524 psi) and for the hard compound is 5.64 MPa (818 psi). The ratio of these maximum shear stresses to the design pressures are 0.39 for the soft compound and 0.45 for the hard compound. In a beyond-design-basis earthquake, these would represent the base shear to which the superstructure would be subjected. The type of superstructure used for this building is such that significant yielding would be expected in the lateral force resisting system at these levels of base shear, or, a softening of the system, increasing the energy dissipation in the frame and increasing the period of the superstructure. Consequently, the superstructure will absorb a larger fraction of the overall displacement than it would if it remained elastic and stiff. This will reduce the displacement demand on the bearings, assuring that in the case of an earthquake of unanticipated magnitude, the bearings will not be the weak link in the overall structural system.

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[3] Kelly, J.M. and Quiroz, E., "Mechanical Characteristics of Neoprene Isolation Bearings," *Report No. UCB/EERC-92/11*, Earthquake Engineering Research Center, University of California at Berkeley, Calif. (1992).

Table 1: Channels used in Hume-Shantou Bearing Tests

Channel	Name	Units	Remarks
1	TIME	Seconds	
2	H.DISP1	Inches	wire potentiometer
3	H.DISP2	Inches	wire potentiometer
4	H.LOAD1	Kips	load cell
5	H.LOAD2	Kips	load cell
6	V1.DISP	Inches	wire potentiometer
7	V2.DISP	Inches	wire potentiometer
8	V1.LOAD	Kips	load cell
9	V2.LOAD	Kips	load cell
10	dcdtvSE	Inches	DCDT
11	dcdtvSW	Inches	DCDT
12	dcdtvNE	Inches	DCDT
13	dcdtvNW	Inches	DCDT
14	Backupwp	Inches	wire potentiometer

Table 2: Soft (Type I) Pair Bearing Test Sequence

Filename	Test Type	Test Name	Axial Pressure (psi)	Test Velocity (in/sec.)	Duration (min.)	Remarks
940124.01	pre-MD	saw550	524	0.20	5	5,25,50% strain
940124.02	pre-MD	saw 75	524	0.20	5	75% shear strain
940124.03	pre-MD	saw100	524	0.20	6	100% shear strain
940124.04	pre-MD	saw150	524	0.20	8.5	150% shear strain
940124.05	post-MD	saw550	524	0.20	5	5,25,50% strain
940124.06	post-MD	saw 75	524	0.20	5	75% shear strain
940124.07	post-MD	saw100	524	0.20	6	100% shear strain
940124.08	post-MD	saw150	524	0.20	8.5	150% shear strain
940125.01	Horiz.	saw550	100	0.20	5	5,25,50% strain
940125.02	Horiz.	saw 75	100	0.20	5	75% shear strain
940125.03	Horiz.	saw100	100	0.20	6	100% shear strain
940125.04	Horiz.	saw150	100	0.20	8.5	150% shear strain
940125.05	Horiz.	saw550	1048	0.20	5	5,25,50% strain
940125.06	Horiz.	saw 75	1048	0.20	5	75% shear strain
940125.07	Horiz.	saw100	1048	0.20	6	100% shear strain
940125.08	Horiz.	saw150	1048	0.20	8.5	150% shear strain
940125.09	Horiz.	saw550	1572	0.20	5	5,25,50% strain
940125.10	Horiz.	saw 75	1572	0.20	5	75% shear strain
940125.11	Horiz.	saw100	1572	0.20	6	100% shear strain
940125.12	Horiz.	saw150	1572	0.20	8.5	150% shear strain
940125.13	Horiz.	saw200	818	0.20	11	200% shear strain
940125.14	Horiz.	saw250	818	0.20	13.5	250% shear strain
940125.15	Horiz.	saw200	100	0.20	11	200% shear strain
940125.16	Horiz.	saw250	100	0.20	13.5	250% shear strain
940125.17	Horiz.	saw200	1048	0.20	11	200% shear strain
940125.18	Horiz.	saw250	1048	0.20	13.5	250% shear strain
940125.19	Horiz.	saw200	1572	0.20	11	200% shear strain
940125.20	Horiz.	saw250	1572	0.20	13.5	250% shear strain
940125.21	Horiz.	saw250	80	0.20	13.5	250% shear strain
940125.22	"Roll-out"	monfail	524	0.20	1.7	368% shear strain
940127.01	"Roll-out"	monfail	25	0.20	1.7	182% shear strain
940127.02	Horiz.	saw150	524	0.20	13.5	150% shear strain

Table 3: Hard (Type II) Pair Bearing Test Sequence

Filename	Test Type	Test Name	Axial Pressure (psi)	Test Velocity (in/sec.)	Duration (min.)	Remarks
940114.01	pre-MD	saw550	818	0.20	5	5,25,50% strain
940114.02	pre-MD	saw 75	818	0.20	5	75% shear strain
940114.03	pre-MD	saw100	818	0.20	6	100% shear strain
940114.04	pre-MD	saw150	818	0.20	8.5	150% shear strain
940114.05	post-MD	saw550	818	0.20	5	5,25,50% strain
940114.06	post-MD	saw 75	818	0.20	5	75% shear strain
940114.07	post-MD	saw100	818	0.20	6	100% shear strain
940114.08	post-MD	saw150	818	0.20	8.5	150% shear strain
940114.09	Horiz.	saw550	100	0.20	5	5,25,50% strain
940114.10	Horiz.	saw 75	100	0.20	5	75% shear strain
940114.11	Horiz.	saw100	100	0.20	6	100% shear strain
940114.12	Horiz.	saw150	100	0.20	8.5	150% shear strain
940118.01	Horiz.	saw550	1500	0.20	5	5,25,50% strain
940118.02	Horiz.	saw 75	1500	0.20	5	75% shear strain
940118.03	Horiz.	saw100	1500	0.20	6	100% shear strain
940118.04	Horiz.	saw150	1500	0.20	8.5	150% shear strain
940118.05	Horiz.	saw200	818	0.20	11	200% shear strain
940120.01	Horiz.	saw250	818	0.20	13.5	250% shear strain
940120.02	Horiz.	saw200	100	0.20	11	200% shear strain
940120.03	Horiz.	saw200	1500	0.20	11	200% shear strain
940120.04	Horiz.	saw250	1500	0.20	13.5	250% shear strain

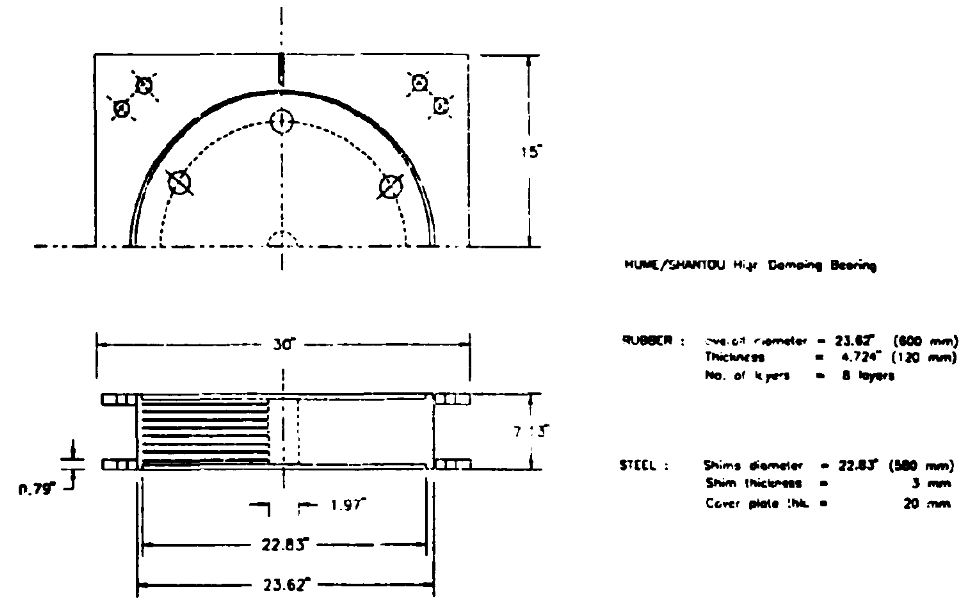
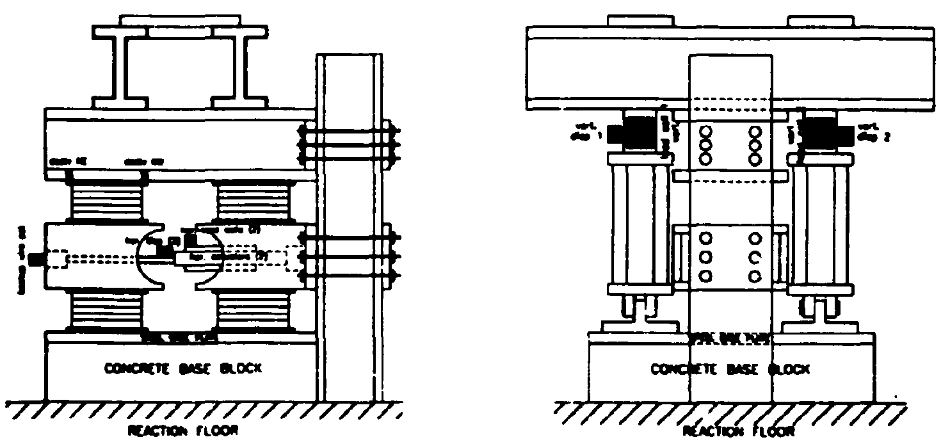


Figure 1:Bearing Design Details



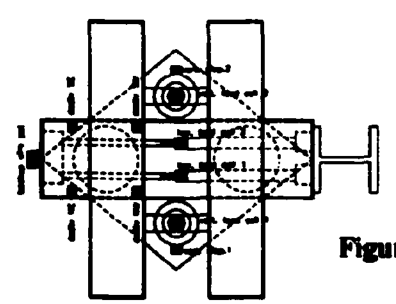


Figure 2:Bearing Test Machine

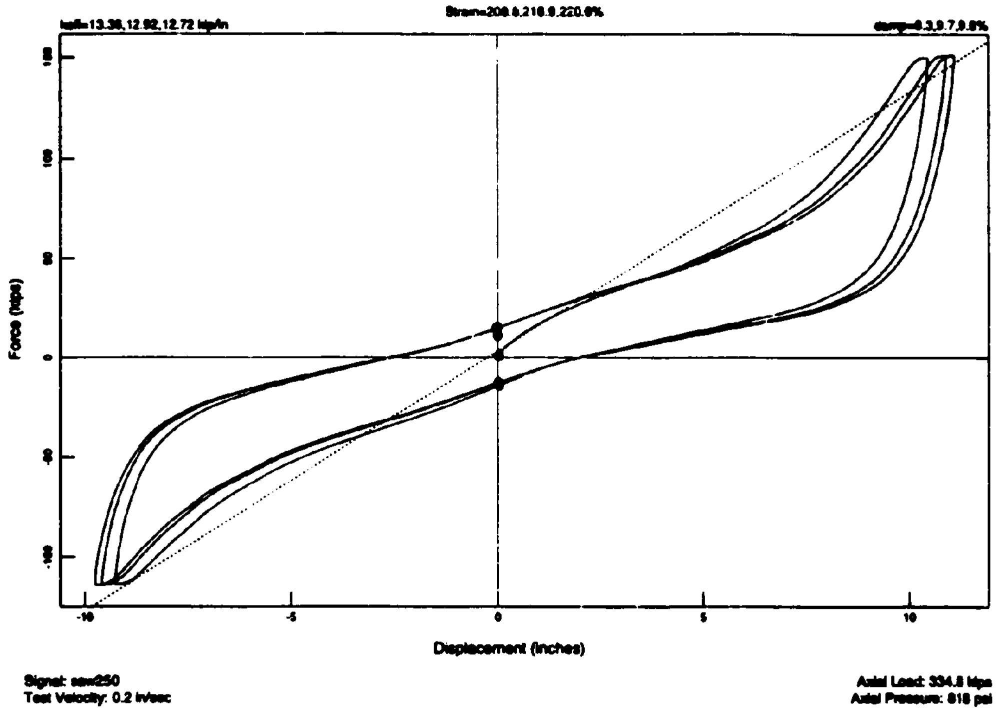


Figure 5: **Hysteresis Loops at 220% Shear Strain (Peak Load Limited) for Hard Compound Bearing at 818 psi**

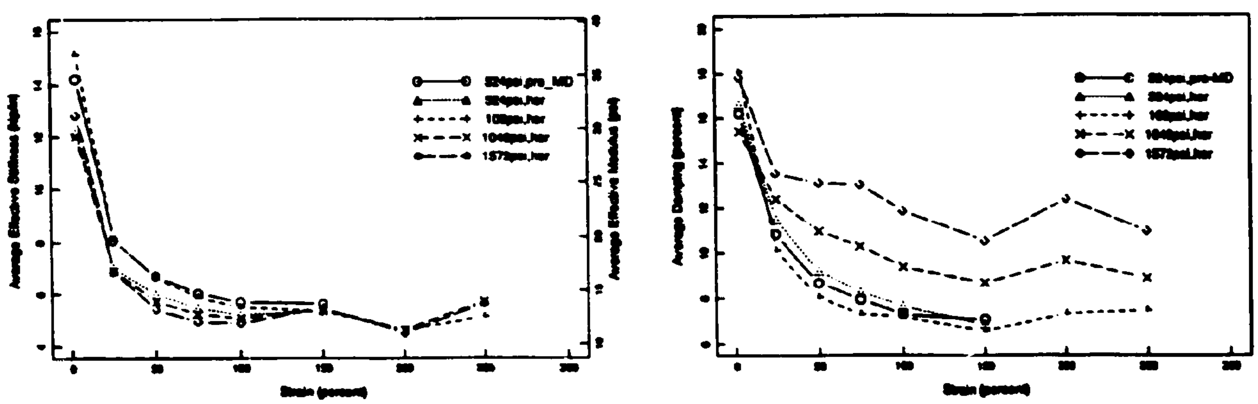


Figure 6: **Effective Stiffness and Damping for Soft Compound Bearing for Different Strains and Pressures**

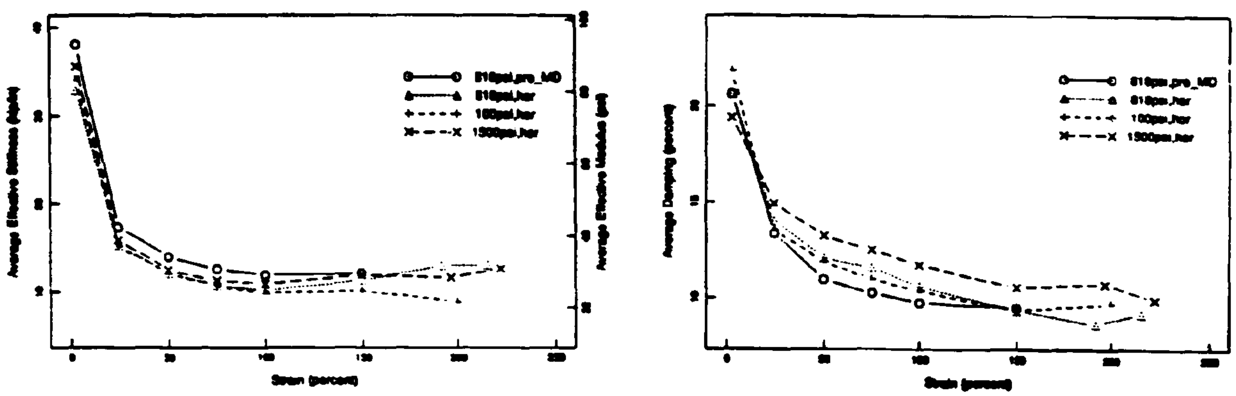


Figure 7: **Effective Stiffness and Damping for Soft Compound Bearing for Different Strains and Pressures**

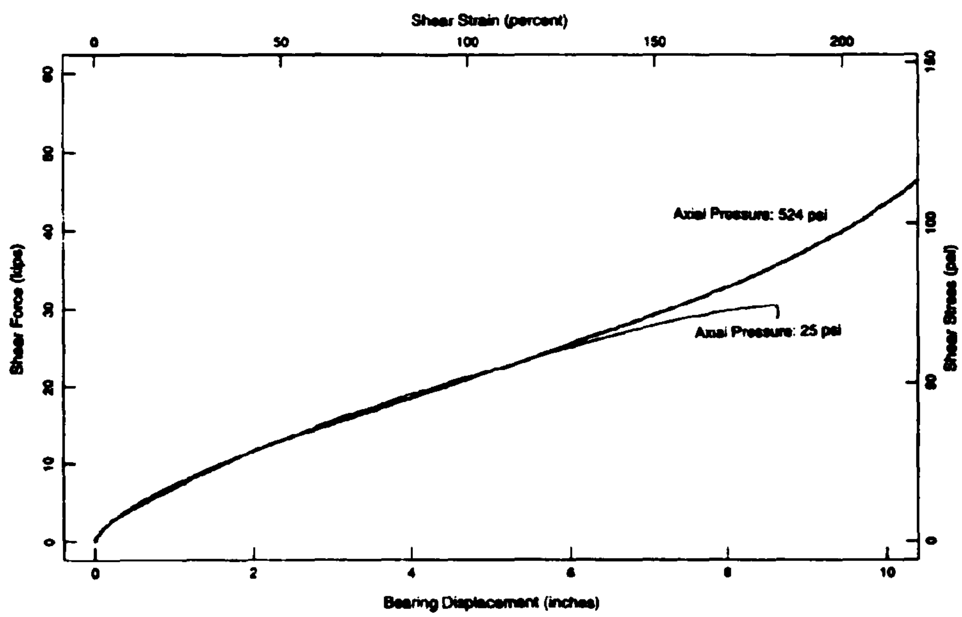


Figure 8: **Monotonic Loading on Soft Compound Bearing at Design Pressure and Low Pressure**

INDUSTRIAL APPLICATIONS OF SEISMIC ISOLATION

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SUMMARY

Seismic isolation of nuclear and non-nuclear industrial facilities in the U.S. is summarized in the paper. The U.S. Advanced Liquid Metal Reactor (ALMR) uses seismic isolation to facilitate plant standardization and to enhance seismic safety. Recent developments in the ALMR seismic isolation qualification program are summarized. Seismic isolation of individual components is very beneficial in situations where existing components and their supports have to be requalified for higher seismic loads. By using seismic isolation, it may be possible to avoid expensive retrofitting of the supporting facility and the foundation. Such an example is the isolation of solid rocket motor segments at Vandenberg Air Force Base in California. In the design of large tanks, hydrodynamic loads can be substantially reduced by isolating the tanks resulting in simplifications in the tank design and its supports.

INTRODUCTION

It has long been recognized that power plant vessels, computers, sensitive equipment, and tanks typically found in industrial facilities are more vulnerable to earthquake damage than buildings. During the Northridge Earthquake there was significant damage attributed to failure of contents such as tanks and pipes. Seismic isolation is a practical approach for providing seismic protection for such systems and components. This is demonstrated in this paper by reviewing several examples of seismic isolation where the primary purpose of using isolation was the protection of components.

SEISMIC ISOLATION OF ADVANCED NUCLEAR REACTORS

Several countries have initiated programs to develop seismic isolation systems for nuclear applications [1] and [2]. In the U.S. the Department of Energy (DOE) sponsored Advanced Liquid Metal

Reactor (ALMR) has adopted seismic isolation to simplify the design, to enhance safety margins, and to support the development of a standardized design for the majority of the available U.S. reactor sites. The nuclear island is being designed for a safe shutdown earthquake (SSE) with a maximum horizontal and vertical acceleration (PGA) of 0.5g. Detailed seismic analyses of the ALMR have been performed and the results reported elsewhere [3]. The ALMR isolated structural configuration consists of a stiff rectangular steel-concrete box structure which supports the reactor vessel, the containment dome and the reactor vessel auxiliary cooling system stacks. The total isolated weight is about 23,000 tons and is supported on 66 high damping rubber bearings, see Fig. 1. The horizontal isolation frequency is 0.7 Hz, and the vertical frequency is greater than 20 Hz.

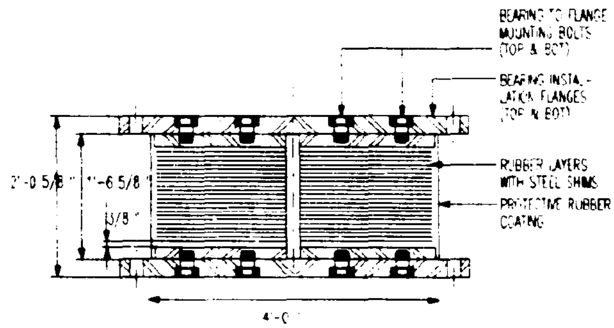


Fig. 1. ALMR Seismic Isolation Bearing

A qualification program for the ALMR seismic isolation system was established to demonstrate that all seismic design and safety requirements are met. The key categories are bearing tests, both scaled and full-size, bearing environmental tests, and seismic isolation system tests [4]. To date a large number of bearing tests have been performed and the results reported elsewhere [1].

A system test program was initiated in 1994. The earthquake simulator at the Earthquake Engineering Research center (EERC)of the University of California at Berkeley will be used. The model will be representative of the ALMR system layout and weight distribution, mounted on several 1/4 and 1/8 scale bearings (Fig. 2). The tests will verify the system response under various loading conditions, verify analysis tools, demonstrate that the system has sufficient margin beyond SSE, and evaluate system response post rupture of one or more bearings.

To validate the response for different level earthquakes two bearing sizes will be utilized. Performance level tests will be conducted to verify the ALMR response characteristics for a range of dynamic loading conditions. The effects of coupling between horizontal and torsional components, and vertical and rocking components will be investigated at design level strains. Oscillators with frequencies corresponding to the scaled frequencies of significant components of the ALMR will be located on the isolated platform and also for comparison, directly on the shake table. Four 1/4-scale isolators will be used for these tests. Margin tests will follow the performance test using the same model this time supported on eight 1/8-scale bearings. Smaller bearings will be used to achieve maximum displacements in the bearings consistent with the capability of the test facility. Bearing displacements near failure will be achieved. It is also considered to perform the tests with one or more pre-failed bearings if bearing failure cannot be induced on the table.

Prior to the shake table tests a large number of 1/4 and 1/8-scale bearings will be tested. These bearings are currently being procured from two different vendors. These tests will determine scaling effects, the mechanical characteristics of the bearings, their large-strain response, and their failure modes. The true 1/8-scale bearings will have individual rubber layer thicknesses of about 1 mm. While these layers are very thin, such sizes are not unprecedented. Ishida [5] has reported on tests of 1/15-scale bearings with layer thicknesses as thin as 0.6 mm. To determine the effects of true scaling of the rubber layers on the mechanical properties of the bearings and demonstrate that the properties are not influenced by the properties of the bonding agent when the layer thicknesses are very small an additional set of bearings will be procured and tested. These will be of the same size as the 1/8-scale bearings, but will differ in the number of rubber layers and the individual layer thicknesses. However the total rubber thickness will be maintained.

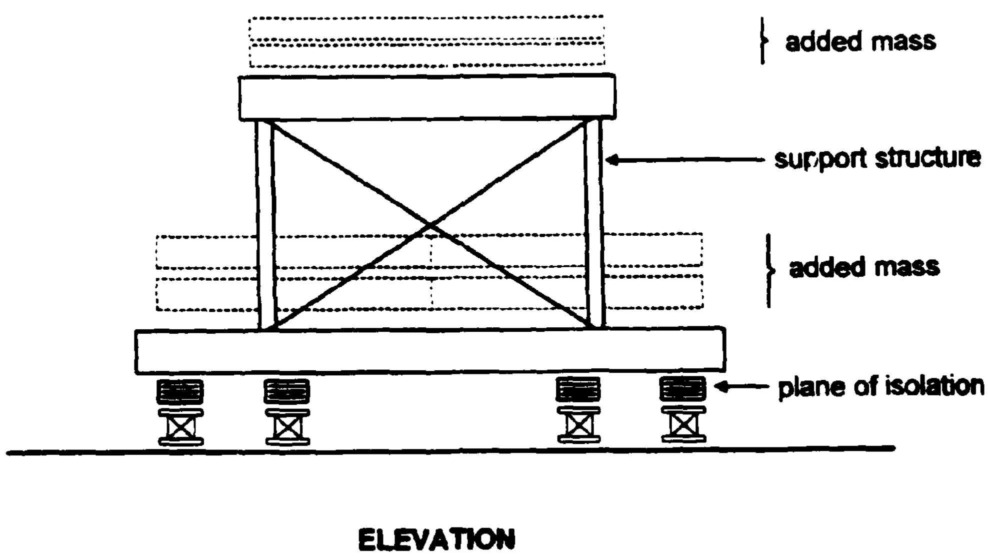


Fig. 2. ALMR Shake Table model

SEISMIC ISOLATION OF COMPONENTS

The benefits of individual component isolation were recognized early on leading to the isolation of 230 kV circuit breakers in Southern California [6]. This preceded application of seismic isolation in buildings. This was followed by shake table tests at EERC, which clearly demonstrated the benefits of seismic isolation of large power plant components [7] as well as light secondary systems [8]. More recent applications of component isolation include the 1500 tons Mark II Detector at the Stanford Linear Accelerator Center which is part of the Linear Collider isolated using four lead-rubber bearings [9]. Also Fragile art objects at The J. Paul Getty Museum in Malibu, California have been isolated using a sliding isolation device [10].

Raised floor systems are widely used around the world in computer rooms and clean room facilities. Most electrical or mechanical equipment are rigidly secured to the floor or are supported on wheels that are locked from movement. In Japan, it is now common practice to isolate raised floor systems. Several different systems have been developed and applied by major construction companies [11]. To date, isolated raised floor systems have not been implemented in the USA. A number of shake table tests have been performed on floor isolation systems developed by IBM at Stanford University [12] and at LERC [13]. In the latter tests, the isolation system consisted of elastomeric bearings with Teflon elements sliding on polished stainless steel plates. Restoring force was provided by a steel-laminated elastomeric bearing. The tests demonstrated that the isolation system is effective in limiting forces transferred to equipment supported on the floor.

Pacific Gas and Electric Company (PG&E) has investigated seismic isolation of non-safety-related equipment at the Diablo Canyon Plant in California. A situation arose in which it was necessary to provide a back-up emergency system in the event of a major malfunction of the main turbine generator exciter in the turbine building. The exciter is not a safety-related piece of equipment, but is needed for the electric power generation. One solution that was considered was the use of two existing mobile exciters which would be placed in the Turbine building to replace the malfunctioning exciter until it could be repaired and put back in service. The mobile exciters consist of transformers and switch gear mounted on a truck trailer, each weighing about 32 tons. PG&E needed to demonstrate that the mobile exciters could not fail during the design-basis earthquake in such a manner as to compromise other nearby safety-related structures. Calculations showed that the exciters would exert large seismic reaction forces; as a result: (1) strengthening of the truck trailers and equipment anchorages would be required, (2) an excessively large mounting skid would be required, and (3) the number of foundation bolts would be excessive. These modifications were undesirable because it would lengthen the installation time of the mobile exciters, increasing the duration of a forced outage. An alternate approach using seismic isolation was developed in which four high-damping rubber bearings were used to support each trailer. The isolation frequency was 1.0 Hz. Eight bearings were procured. A ninth additional bearing was extensively tested at EERC.

No specific design codes or regulations exist for seismic isolation of components. In general, most of the guidelines developed for seismic isolation of buildings are applicable. For most components, the isolated weight is relatively small, and only four isolators are used. This means that there is less redundancy in the design of the isolators requiring high quality isolators with minimum variation in properties. If the load per bearing is less than 5 tons, it becomes impractical to use conventional elastomeric bearings, and other types of isolators have to be used. Another implementation difference is that components are not always supported at ground level. For these cases, it is necessary to modify the design earthquake to incorporate any building amplifications. Furthermore, the vertical component of the earthquake and the building floor flexibility should be accounted for in the design. Finally, the effect of moderate earthquakes should be considered in the design and selection of the isolation system used for equipment, even though it is not required by existing building codes.

<u>Seismic Isolation of Titan IV Solid Rocket Motor Upgrade Segments</u>

Another novel application of seismic isolation was the isolation of solid rocket motor upgrades (SRMU). SRMUs will help increase the payload of Titan IV launch vehicles (LV) to be used to send payloads into space from Vandenberg Air Force Base (VAFB), California. Each Titan LV consists of a liquid-fueled core vehicle, and two strap-on Each SRMU is divided into three segments: Aft, Center, and Forward, see Table 1. Before launch, the SRMU segments will be stored and checked in an existing facility near the launch site. This facility required modifications for SRMU handling. Because VAFB is located in a seismically active region, there was concern that personnel could be endangered due to movement or failure of SRMU segments during an earthquake. Several

conventional design concepts to restrain the SRMU segments were evaluated, including rigidly attaching the bases of the segments to their foundations, and using existing steel access platform stands to provide lateral support for the SRMUs. It was concluded that the Forward segments, which are lighter than the other segments could be supported rigidly at the base without requiring major additional modifications to the supports. However, for the Aft and Center segments the implementation of rigid foundations would have required the excavation of the existing foundation slab and installation of new foundations underneath the segments' support stands. Use of existing steel access platforms to restrain the SRMU segments would have required considerable facility structural modifications. This would have imposed severe operational constraints due to the need to install and remove bracing during placement and removal of a segment from a stand. The above modifications would also have required an extended construction period and thus would not have been compatible with program objectives. Seismic isolation was therefore selected for the Aft and Center Segments. This minimized the requirement for major modifications to the existing foundations and structures.

Table 1 Properties of SRMU Segments

Segment Type	Maximum Height (m)	Weight (tons)	Fixed-Base Frequency (Hz)
Aft (with Nozzle)	15.5	167	4.8
Center	10.4	144	6.1
Forward	5.2	67	25

The site is relatively stiff, and consists of engineered fill underlain by shale with an average shear wave velocity of 730 m/s. A site specific spectrum was developed for this site which had a peak ground acceleration of 0.6 g. This spectrum is compared with the SEAOC/UBC spectrum for a Zone 4, S1 site in Fig. 3. No near-field effects had to be considered. The SRMUs are supported at grade on foundations separated from the building foundation. The ground spectrum was used directly as input. The design was governed by the SEAOC spectrum and the static displacement formula.

The SRMU segments to be housed in the processing facility may be located either in designated storage areas or moved to the build-up areas where they are checked before assembly. Isolation bearings are used in both areas to isolate the Aft and Center segments individually, since the total number of segments at any given time housed in the facility is not fixed. Each segment is supported on an isolated steel stand resting on four bearings. Additionally, the frame used to move the Aft and Center segments inside the facility (rotating fixture), will be parked on isolated concrete tables if processing requires that the segments remain on the rotating fixture for more than 24 hours.

Four high damping rubber bearings were used to support each stand. To simplify the procurement, testing and installation of the bearings, it was decided that only one size be used for all isolated SRMU configurations. The important design requirements were that the selected bearing type be effective in isolating moderate as well as large earthquakes and that it would have a self-centering capability. The majority of seismic isolators used in buildings in the USA have been designed with only the "big" earthquake as an important design consideration. Usually, the effect of a moderate close-by earthquake is not considered since it does not control the design. The resulting isolation systems, whether they are high damping rubber bearings, or rubber bearings with hysteretic damping, tend to be stiff at low strains. Thus, during small or moderate earthquakes, buildings supported on these systems would not necessarily behave as isolated structures and the base response would be amplified by the structure. This has been observed and documented in a study of the measured response of isolated structures in the USA [14].

Recent developments of new rubber compounds have made it possible to design isolators which are effective during moderate earthquakes by selecting isolators with lower stiffness at small shear strains. For example, high damping elastomeric compounds with lower shear moduli have been developed. These compounds are more appropriate for isolating components and buildings founded on soft sites where it is preferable to lengthen the isolation period from 2 to 3 [15]. In Japan, a lead rubber bearing with a stepped plug has been developed and used for the same reason [16]. Other efforts in the U.S. to develop high damping rubber bearings which are effective both for moderate earthquakes as well as large earthquakes are underway [17]. Shake table tests at EERC in which the same model was tested on elastomeric bearings with different moduli confirmed that the softer bearings were more effective in isolating lower level earthquakes [18].

For this project, a new softer high damping rubber compound was developed by the bearing manufacturer. The shear modulus and damping characteristics of this compound are compared with a more conventional compound in Figures 4 and 5. The use of this softer compound will insure better response during small earthquakes, at the same time results in a bearing configuration with a larger diameter to height ratio, enhancing stability during large earthquakes.

The selected bearing design had a diameter of 38 cm, and a total height of 25.4 cm. The rubber stack consisted of 23 layers with a thickness of 6.4 mm. A total of 3 prototypes and 64 production bearings were made. The isolation frequencies were 0.52 Hz for the Aft segment and 0.56 Hz for the Center Segment at the design displacement. The bearings were bolted to the foundation and the SRMU support frame. The total maximum displacements for the Aft and Center segments were 22.4 cm and 20 cm respectively. Fig. 6 shows a schematic of the isolated Center and Aft segments in the storage area.

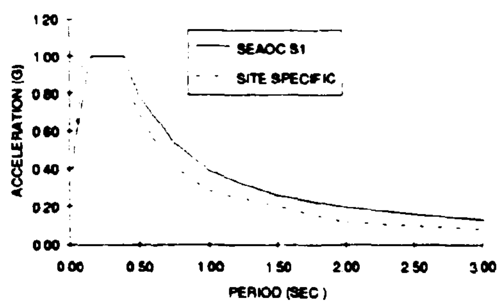


Fig. 3. Comparison of Seismic Design Spectra (5% Damping)

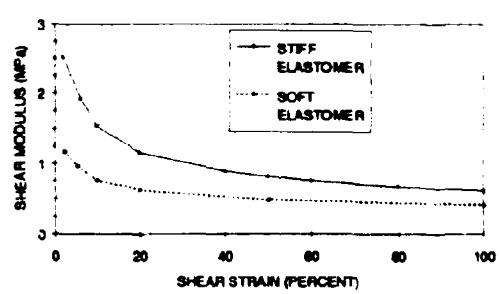


Fig. 4. Variation of Rubber Shear Modulus with Shear Strain

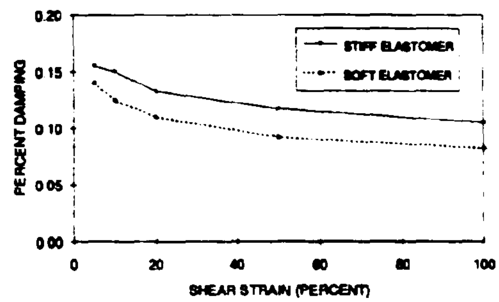
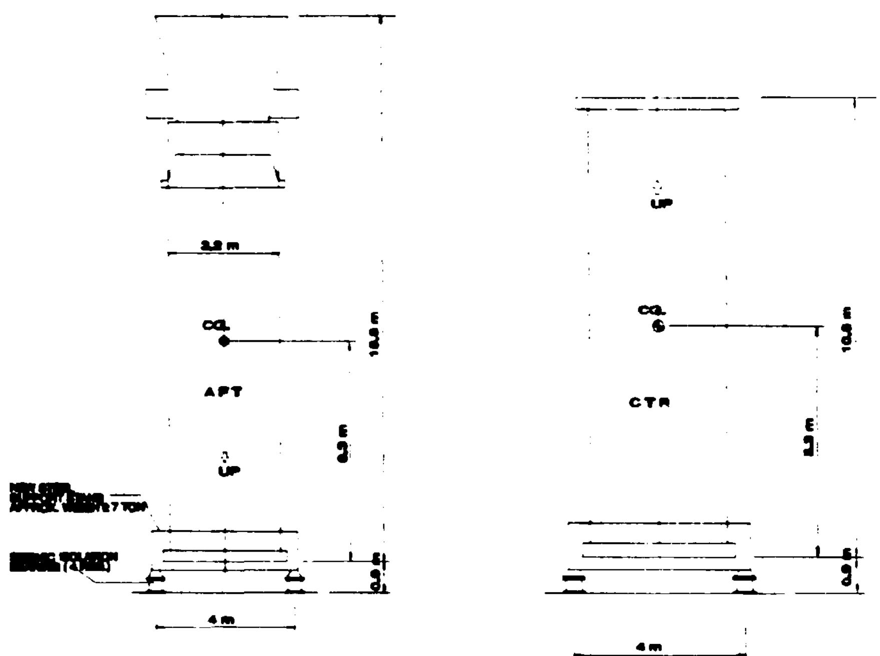


Fig. 5. Variation of Rubber Damping with Shear Strain

SEISMIC ISOLATION OF TANKS

Tanks have not performed well during recent earthquakes. Both concrete and steel tanks were seriously damaged during the Northridge Earthquake. One way to avoid damage to tanks is the strength approach, whereby the tank and its supports are designed such that yielding is minimized. Another approach is to use seismic isolation to considerably reduce hydrodynamic loads. Shake table tests were carried out at EERC which compared the response of isolated tanks versus fixed tanks [19]. It was shown that seismic isolation drastically reduced impulsive dynamic pressure exerted by the fluid on the tank walls. It was also shown that even though sloshing frequencies were close to the isolation frequencies, the reduction in the impulsive component was much more significant than the increase in convective component.

Seismic isolation has been and will be used to isolate various tank configurations. An emergency water tank at the DOW Chemical Company in Pittsburg, California and an ammonia storage tank in Calvert City, Kentucky were isolated using FPS isolators [20]. Calculations have shown that the seismic forces are reduced by 60 percent during a severe earthquake. More recently, Cygna Consulting Engineers have developed designs to seismically upgrade large elevated water tanks in Seattle, Washington using high damping rubber isolators [21]. The tanks have a 3.78 million liter capacity. The isolation bearings will be placed at the foot of the existing column underneath an annular concrete base slab tying the base of the superstructure. Twenty-one isolators will be used. Dynamic analysis results showed that the interstory drift and base shear in the tanks were reduced by 2 and 3 times. A cost analysis was also performed showing that seismic isolation was 62 percent less costly than conventional upgrading.



a. Aft Segment b. Center Segment

Fig. 6. Seismic Isolated SRMU Segments

Another type of tank with increasing applications around the world is used for storing Liquefied Natural Gas (LNG). These tanks are very large and have capacities around 150,000 m^3. These tanks pause a great risk I they fail during an earthquake. A review of LNG projects which may be isolated was given in [22]. The results of a study to determine the benefits of isolating large LNG tanks are summarized below. The tanks consisted of a free-standing steel tank which contained the LNG, insulation, and a concrete tank encasing and protecting the inner tank. The inner tank wall was supported by a concrete ring beam that slid to relieve thermal stresses. The tank outside diameter is approximately 58 m and is about 43 m high. A schematic of the tank is shown in Fig. 7. The total weight of the tank when full was about 67,000 tons. Both inner and outer tanks were supported on a common concrete mat. Usually, such tanks are supported by a group of closely spaced short columns to allow air to circulate beneath the tanks. Seismic isolators were placed between the column tops and the tank-base to reduce seismic loads on the tank, and to relieve horizontal loads on the columns resulting from thermal contraction of the base slab.

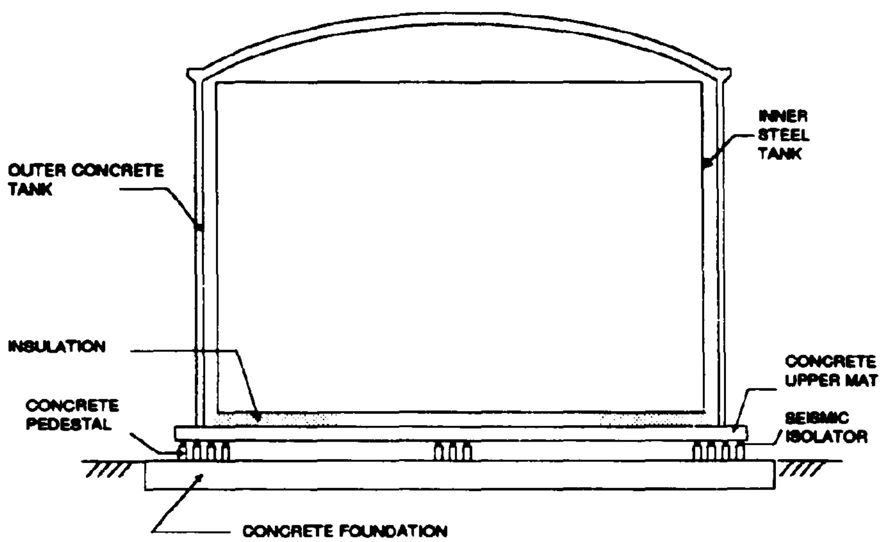


Fig. 7. Schematic of Isolated LNG Tank

The tanks were modeled using a lumped mass stick model. Hydrodynamic effects were represented using Housner's analog [23]. The isolators were represented as equivalent springs. A damping of 10 percent was assumed for the isolators. Soil-structure interaction effects were represented as equivalent springs and dampers attached to the lower foundation. Soil damping (radiation damping) was limited to 20 percent. Composite modal damping was computed by the strain energy approach. The design earthquake was represented as a Newmark-Hall response spectrum scaled to a peak horizontal ground acceleration of 0.2 g. The horizontal input was scaled by 2/3 in the vertical direction. The isolation frequency was selected as 0.45 Hz. The maximum horizontal displacement was computed to be 11.7 cm. The overturning moments (M), shear forces (V), and axial forces (P) at the base of the two tanks are summarized in Table 2.

The benefits of isolating the tank can be clearly seen in the table above. For example, the overturning moment and base shear at the base of the steel tank are reduced by more than six folds. An important observation is that the lateral response is insensitive to SSI effects. In general, when evaluating the response of isolated structures on most sites, SSI effects can be neglected or simply represented by springs and dampers. This is one of the benefits of using seismic isolation; making it unnecessary to perform elaborate SSI analyses normally required for most critical structures, . Since the tank is not isolated in the vertical direction, SSI effects can be important and should be considered. As can be seen in the above table, the axial loads are higher when SSI effects are included due to a dominant vertical foundation mode.

Table 2 Summary of Tank Seismic Response

Case	Concrete Tank			Steel Tank		
	M (ton-m)	V (ton)	P (ton)	M (ton-m)	V (ton)	P (ton)
No Isolation, No SSI	220,050	20,900	10,100	196,925	47,100	650
No Isolation, With SSI	188,650	17,900	11,000	201,150	48,100	7,000
With Isolation, No SSI	34,350	4,600	13,000	30,700	7,600	2,500
With Isolation, With SSI	35,100	4,600	11,400	30,640	7,600	7,700

An important aspect of the inner tank design is to avoid the use of anchor straps to minimize the welded attachments to the cryogenic steel. The mechanism of tank uplift is complex and not completely understood. To describe it fully, the effects of large displacements yielding of the base plate, membrane forces in the base, phase relationship between the horizontal and vertical components and the effect of these parameters on the period of the system need to be considered. To simplify the uplift evaluation, the recommendations of the American Petroleum Institute [24] were followed. The resistance to shell overturning provided by the weight of the tank, w_L, can be estimated using the following equation:

$$w_L = 7.9 t_b \sqrt{F_{by} G H}$$

where, t_b is the bottom plate thickness in inches, F_{by} is the yield strength in psi, G is the specific gravity of fluid, and H is the fluid depth in ft. Using the formula gives a resistance capacity of 11.7 kip/ft (17.1 ton/m). The maximum overturning tension per unit length assuming an inner tank diameter of 54 m, is 5.7 kip/ft (8.3 ton/m). The resistance capacity exceeds the demand by a factor of 2. The demand without isolation is 17.5 kip/ft (25.6 ton/m) exceeding the capacity by 1.5 times. Thus, by isolating the tank, the use of anchor straps can be avoided.

A parametric study was performed to determine the optimum size and number of isolation bearings used and the concrete support column spacing. The minimum spacing was controlled by maintenance considerations. The total volume of the bearings to be used was constant and was a function of the isolated weight, isolation frequency, and design displacement. If the total cost of the isolators per unit volume was assumed to be constant, then bearing size was not an important parameter when seeking an optimum design. The additional cost to test and install a larger number of small bearings was not accounted for in this study. The remaining material costs were the concrete base slab and the columns. The study indicated that the concrete volume could be reduced by using more closely spaced small bearings. Although the column concrete volume increased, this was more than offset by reductions in the concrete slab thickness. This study also assumed that the use of a stepped slab, which is thicker at the tank wall/slab connection, was practical. If a uniform thickness slab was preferred, then the columns should be spread as far as possible.

CONCLUSIONS

In general, the rules developed for the design of isolators for buildings are also applicable to components. However, in applications where only four isolators are used, it is important to specify more comprehensive quality assurance tests than normally required in building applications. One test useful in uncovering manufacturing defects in elastomeric isolators would be to perform shear tests with minimum axial load. Tighter acceptance tolerances should be specified to minimize property variations between bearings. Although the acceptance of this technology for isolation of components and tanks has been slower than in buildings, future applications should increase as owners of industrial facilities realize that conventional seismic design techniques may not be adequate in protecting sensitive equipment and tanks. The development of new isolation techniques including softer elastomers and low friction rollers would make it easier to isolate lighter components and possibly further lengthen the isolation period to further reduce seismic forces and or to use seismic isolation on soft sites with fundamental periods between one and two seconds.

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Appendix D

List of Base-Isolated Buildings in P.R. China

List of Base-Isolated Buildings in P.R. China

No.	Type of Building	City/ Province	Size of plan (m)	Height (m)	Stories	Total Floor Area (m^2)	Superstructure Type	Isolation System	Date Completed
1	Dwelling	Huaping, Yunnan	4×4	5	1	16	Adobe wall	Sand layer	1975
2	Dwelling	Xichang, Sichuan	4×5	6	1	20	Tamped wall	Sand layer	1975
3	Weight bridge Dwelling	Anyang, Henan	3×4	6	1	12	Brick wall	Sand layer	1980
4	Dwelling & E-Q obs. center	Beijing	7×25	15	4	700	Brick wall, RC floor	Sand layer	1981
5	Equipment building	Hashi, Xinjiang	8×24	12	3	576	Brick wall, RC floor	Sand layer	1983
6	Dwelling	Xichang, Sichuan	12×28	21	6	2016	Brick wall, RC floor	Graphite & lime mortar	1985
7	Dwelling	Darli, Yunnan	12×24	20	6	1728	Brick wall, RC floor	Slide steel piece	1992
8	Dwelling	Xian, Xanshi	14×28	26	8	3136	Brick wall, RC floor	Slide steel piece	1993
9	Dwelling & shops	Xian, Xanshi	16×22	20	6	2112	Brick wall, RC floor	Slide steel piece	1993

No.	Type of Building	City/ Province	Size of plan (m)	Height (m)	Stories	Total Floor Area (m^2)	Superstructure Type	Isolation System	Date Completed
10	Dwelling & shops	Shantou, Guangdong	13x26	27	8	2704	RC frame	Rubber bearing	1993
11	Dwelling & restau-rant	Anyang, Henan	10x36	19	6	2160	RC frame (1st-2nd fl.) Brick wall (3rd-6th fl.)	Rubber bearing	Under con-struction
12	Industrial facility	Xichang, Sichuan	16x24	20	5	1920	RC frame	Rubber bearing	Under con-struction
13	Dwelling & stops	Xichang, Sichuan	19x33	24	6	3762	Brick wall & RC floor	Rubber bearing	Under con-struction
14	Dwelling	Xichang, Sichuan	16x30	20	6	2880	Brick wall	Rubber bearing	Under con-struction
15	Dwelling	Xichang, Sichuan	16x30	20	6	2880	Brick wall	Rubber bearing	Under con-struction
16	Dwelling	Xichang, Sichuan	16x30	20	6	2880	Brick wall	Rubber bearing	Under con-struction
17	Dwelling	Xichang, Sichuan	16x30	20	6	2880	Brick wall	Rubber bearing	Under con-struction
18	Student dormitory	Guangzhou Guangdong	14x42	23	7	4116	RC frame	Rubber bearing	Under con-struction
19	Dwelling & office building	Darli, Yunnan	14x43	26	8	4816	RC frame	Rubber bearing	Under con-struction

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