Technology Transfer Package on Seismic Base Isolation

Volume III

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Base isolation response to extreme ground motions

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ABSTRACT: The elastomeric seismic base isolation system employed in the Los Angeles County Fire Command and Control Facility incorporates an "ultimate restraint" system, intended to limit building displacements under extreme levels of ground shaking. Isolator-superstructure response under postulated extreme ground motions, calculated by nonlinear time history analysis, are given. Results indicate that the response is displacement driven and, therefore, the ultimate restraint system does not significantly limit building motions. As expected, overall base shear is increased as the restraint system is engaged. Displacement overcapacity of the isolation system is recommended as the preferred means of providing safety margin above working values.

1 INTRODUCTION

The Los Angeles County Fire Department's Fire Command and Control Facility (FCCF) is an indispensable link in the County's earthquake lifelines. It receives, dispatches and directs county-wide fire and medical emergency relief services in response to 911 calls. This mission is of critical importance during and following major earthquakes in the Los Angeles area. Seismic base isolation was employed by Fluor Daniel in order to provide the highest level of seismic protection for the facility and to assure its continued function.

As an additional design precaution, the isolation system was fitted with a unique ultimate restraint system (URS), meant to limit building motions in the highly unlikely event the ground motions exceed the design displacement capacity of the isolators themselves. In effect, the ultimate restraint system, coupled in series with the reserve displacement capacity of the bearings, exhibits a strain hardening behavior similar to that exhibited by mild steel when deformed beyond its yield plateau. The restraint system was sized to resist a load equal to 125 percent of the design base shear which is transmitted through the isolation system. It should be noted that the decision to use an ultimate restraint system was not to satisfy any code seismic requirement, since at the time detailed engineering for the FCCF was carried out code provisions for seismic isolation had yet to be promulgated.

Analysis, design and construction details of the FCCF seismic isolation system, including the ultimate restraint feature, have been described by Anderson (1990). Dynamic response of this facility as calculated by nonlinear time history analysis for site-specific design ground motions was reported by Bachman, Gomez and Chang (1990). This paper extends those earlier works by reporting on the nonlinear response of the building

subjected to ground motions large enough to fully engage the ultimate restraint system.

The primary margin of safety for the isolator system is embodied in the generous 100 percent displacement reserve above that required to satisfy the demand of the 500 year design earthquake. Accordingly, the ultimate restraint feature is highly unlikely to ever be loaded by a real earthquake. Further, the isolation design meets and generally exceeds all extant seismic code requirements that subsequently have been adopted for building base isolation systems (see for example Uniform Building Code (1991)).

The reporting of these special studies is in the general interest of furthering the understanding of design improvements and supporting wider adoption of seismic isolation technology into the mainstream of structural design practice.

2 GROUND INPUT MOTION

Seismic design criteria for the FCCF are described in detail by Bachman, Gomez and Chang (1991). The criteria includes a site-specific ground motion time history designated as SEGM-2, representing a 1000 year event with a peak ground acceleration of 0.50g. This motion has broad band frequency content and is rich in long period components (see Figure 1).

Preliminary analyses showed that the SEGM-2 record required a scaling factor of about two in order to initially engage the ultimate restraint. Additionally, in order to fully exercise the restraint system, an earthquake record was sought with a large velocity pulse and related near-fault characteristics. The James Road record (see Figure 2) from the 1979 Imperial Valley earthquake was selected for this purpose and used in the analyses. It has extremely large long period motion and spectral displacements at least twice as large as the SEGM-2 record.

Figure 1. SEGM-2 Response Spectra.

Figure 2. James Road Response Spectra.

3 DYNAMIC MODELS

Two primary planar models were developed for the nonlinear time history dynamic analyses. The first model (see Figure 3) was used for transverse frame

analyses. It represented a tributary portion of the two-story steel braced frame superstructure using linear elastic elements and a tributary portion of the elastomeric isolator bearings with the ultimate restraint system using idealized tri-linear elements. The force-deflection curve for a single isolator bearing, fitted with the ultimate restraint, is shown in Figure 4.

A second model (see Figure 5) was used for performing torsional (about a vertical axis) analyses. It consisted of a horizontal, elastic planar grid supported on the 32 elastomeric isolator bearings. Only the perimeter isolator units are fitted with the internal ultimate restraint feature. The force-deflection curve

Figure 3. 2-D Transverse Frame Model

Figure 4. Idealized Base Isolator Force Deflection Properties.

Figure 5. 2-D Base Frame Model.

shown in Figure 4 represents the behavior for all isolators, except the interior elements do not include the increased stiffness leg shown in this figure for displacements over 12.5 inches; they continue to deflect at the 2.2 kips/in rate for all large displacements. The hysteresis loop represents an equivalent fifteen percent viscous damping for the model at a displacement of about ten inches.

For both of the analysis models, the stiffness behavior of the ultimate restraint component of the models was varied in several ways to assess response sensitivity. The standard system was assumed to respond as previously described (and as generally represented by Figure 4) up to a lateral displacement of 15 inches after which the load-displacement curve was assumed to be perfectly plastic (zero stiffness). This condition is identified in the subsequent tables and figure as non-linear w/URS. A first variation on this standard system was to consider the absence of the ultimate restraint component and assume the isolation system had a simple bilinear load-displacement curve, denoted as non-linear wo/URS in subsequent tables.

A second variation (denoted as mod 1) was to assume the ultimate restraint system exhibited unchanged stiffness as it displaced beyond 15 inches, rather than go from elastic to perfectly plastic at this point. For the third variation, the ultimate restraint system stiffness was increased by a factor of about three, and denoted as mod 2 in the tables and figures.

4 DYNAMIC ANALYSES

The computer code DRAIN-2D (Kanaan (1974)) was used to conduct all of the nonlinear time history analyses. The program was modified to include an equivalent "chain element" to represent the ultimate restraint system. Transverse and torsional analysis models were subjected to a variety of SEGM-2 time histories, scaled from 0.05 to 4 times the normal record, as well as the unscaled James Road record. Scaling of the SEGM-2 record was undertaken in order to study response variations as the intensity of input motion was increased to gradually, then fully engage the ultimate restraint system.

5 ANALYSIS RESULTS

5.1 Displacements

Peak response relative displacements using the transverse model are summarized in Table 1. Note, the ultimate restraint system begins to engage at a lateral displacement of 12.5 in.

An examination of displacements in Table 1 indicates that response is clearly displacement driven. That is, the ultimate restraint system does not provide a significant reduction in maximum displacement. The URS is engaged at an input level of about two and one-half times the SEGM-2 input motion. Hence, differences in response arising from the URS would be seen in comparing responses for 4.0 * SEGM-2 and the James Road inputs. For the 4.0 * SEGM-2 input the URS essentially has no influence on displacement. In fact, the response is seen to increase slightly with the

Table 1
Calculated Horizontal Peak Relative
Displacement at Center of Floor

Analysis Model	Input Motion	Displacement (in)		
		1st Floor	2nd Floor	Roof
Non-Linear wo/URS	0.10 * SEGM-2	0.83	0.88	0.90
	0.15 * SEGM-2	0.95	0.99	1.00
	0.25 * SEGM-2	1.23	1.27	1.30
	1.0 * SEGM-2	4.08	4.15	4.21
	2.0 * SEGM-2	10.89	11.06	11.15
	4.0 * SEGM-2	29.63	30.07	30.29
	JAMES ROAD	20.33	20.63	20.66
Non-Linear w/URS	0.10 * SEGM-2	0.83	0.88	0.89
	1.0 * SEGM-2	4.08	4.15	4.21
	2.0 * SEGM-2	10.89	11.06	11.15
	4.0 * SEGM-2	31.57	32.07	32.34
	JAMES ROAD	17.62	17.99	18.03
	JAMES ROAD (mod 1)	17.25	17.76	17.80
	JAMES ROAD (mod 2)	17.41	18.28	18.34

URS, e.g., first floor displacement increases from 29.63 in., without the URS, to 31.57 in., with the URS. The nonlinear increase in response due to a doubling in input motion (2.0 to 4.0 * SEGM-2) is a result of engaging the URS (leads to reduced displacement) and a reduction in equivalent viscous damping (leads to increased displacement) as displacements are increased. It appears that for this particular input, the latter effect outweighs the former.

For the James Road input, the displacement response is seen to reduce with the URS in place, but the level of reduction, 15 percent, is quite small. The URS does not appear to significantly limit displacements for the two input motions considered.

Furthermore, modifications to the URS stiffness (designated as mod 1 and mod 2 in Table 1) has essentially no influence on total system response. This result further supports the observation that the response is strongly displacement driven.

The foregoing observations on nonlinear response are also supported by the response factors shown in Table 2. Equivalent period and corresponding equivalent viscous damping as calculated at the point of maximum relative first floor displacement are given for each input time history shown in Table 1. In addition, Table 2 shows the spectral displacement S_d corresponding to the calculated equivalent period and damping values. It can be seen that because of the nonlinearity of stiffness (or period) and damping, the corresponding spectral relative displacement does not increase in a linear fashion with a corresponding increase in input ground

Table 2
Equivalent Period, Viscous Damping and Corresponding
Spectral Displacement at Maximum 1st Floor Displacement

Analysis Model	Input Motion	T (sec)	ζ (% critical)	$S_d(T, \zeta)$ (in.)
Non-Linear wo/URS	0.10*SEGM-2	1.31	5.4	0.50
	0.15*SEGM-2	1.37	10.1	0.52
	0.25*SEGM-2	1.50	16.4	0.71
	1.0 * SEGM-2	2.01	17.9	3.8
	2.0 * SEGM-2	2.27	9.7	11.6
	4.0 * SEGM-2	2.39	4.2	36.3
	JAMES ROAD	2.35	5.8	22.3
Non-Linear w/URS	0.10*SEGM-2	1.31	5.4	0.50
	1.0 * SEGM-2	2.01	17.9	3.8
	2.0 * SEGM-2	2.27	9.7	11.6
	4.0 * SEGM-2	2.40	3.9	38.2
	JAMES ROAD	2.34	6.6	18.7
	JAMES ROAD (mod 1)	2.33	6.7	18.7
	JAMES ROAD (mod 2)	2.34	6.7	18.7

motion, or acceleration. Also, the resulting spectral displacements in Table 2 compare reasonably well with the first floor displacements from which one would conclude that a single degree of freedom approximation is reasonable.

5.2 Accelerations

Use of the URS involves the trade off between displacement and acceleration as always exists in the field of structural dynamics. Any displacement-reduction benefit will result in increased response accelerations. Calculated response accelerations are summarized in Tables 3 and 4.

The effect of the URS on the acceleration response is best illustrated by comparing Tables 3 and 4 for the 4.0 * SEGM-2 records. Addition of the URS is seen to result in an 80 percent increase in response acceleration as measured at the first floor end frame for the 4.0 * SEGM-2 input, viz., 0.58g to 1.06g. The increase is smaller for the James Road ground motion, increasing somewhat less than 50 percent, except for mod 2 in which the URS stiffness is increased. For mod 2, the result is a factor of three increase in acceleration. Clearly, this is a large potential penalty to pay for an insignificant reduction in response displacement. It could be concluded from this result that additional safety margin might better be achieved with additional isolator displacement capacity reserve (and corresponding additional rattle space surrounding the building) rather than considering displacement control in this fashion.

It can also be noted from Table 3 that significant nonlinearity occurs between 0.1 and 1.0*SEGM-2 input motion. Reduction of ground motion experienced by the building, i.e., isolation, begins at input motion just above 0.1*SEGM-2.

5.3 Torsional response

An evaluation of torsional response, using the planar grid model for generally the same pattern of ground motion inputs, revealed similar response behavior noted above. That is, response is displacement driven and is little influenced by the URS.

It was also observed during the course of the investigation that a conservative preliminary estimate (within 20 percent) of the maximum displacement of the isolated building system, with or without an URS, could be simply taken as the spectral displacement of a single DOF system at an equivalent viscous damping ratio corresponding to the nominal design displacement. This evaluation would allow one to size the rattle space gap at an early stage in design development for later confirmation by nonlinear time history studies.

6 CONCLUSIONS

The results of the analyses performed indicate that the FCCF will not only meet its original design criteria, but also will safely respond to input motions approaching four times the site-specific ground motions.

Ultimate restraint systems may be quite effective in resisting uplift arising from overturning in more slender structures. However, incorporation of an ultimate restraint system to control building displacements as ground motion input levels increase appears to have limited usefulness. Providing additional isolator displacement capacity and free rattle space seems to be the more realistic approach to achieving increased safety margin. It is believed that this latter conclusion has important implications for the design of all seismic isolation systems.

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Table 3
Calculated Peak Horizontal Response Accelerations
Non Linear Response w/URS

Location	Acceleration (g) SEGM-2 Time History				Accel. (g) James Road Time History
	0.10	1.0	2.0	4.0	
Scaling Factor	0.10	1.0	2.0	4.0	1.0
Roof End Frame	0.054	0.22	0.42	0.87	0.39
2nd Floor End Frame	0.051	0.13	0.25	0.66	0.37
1st Floor End Frame	0.048	0.15	0.28	0.58	0.37
Peak Ground Accel.	0.05	0.50	1.0	2.0	0.37

Table 4
Calculated Peak Horizontal Response Accelerations
Non Linear Response w/URS

Location	Acceleration (g) SEGM-2 Time History				Accel (g) James Road Time History		
	0.10	1.0	2.0	4.0	1.0	mod 1	mod 2
Scaling Factor	0.10	1.0	2.0	4.0	1.0	mod 1	mod 2
Roof End Frame	0.05	0.22	0.42	1.22	0.60	0.56	1.13
2nd Floor End Frame	0.05	0.13	0.25	0.87	0.52	0.58	0.94
1st Floor End Frame	0.05	0.15	0.28	1.06	0.54	0.56	1.09

Base isolation response to extreme ground motions/

PIPELINE FAULT-CROSSING DESIGN STRATEGY

T. L. Anderson (I)

SUMMARY

This paper presents the elements in a pipeline fault-crossing design strategy for the case where the likelihood of large fault offsets at the pipeline crossing during the design life is low. Where acceptable risk of pipeline rupture has not been defined, it is recommended that the design decision be fundamentally based upon an economic analysis, to achieve minimum total cost during the design life, as well as other considerations. An outline of the strategy is given.

INTRODUCTION

Although an active fault crossing is the greatest seismic hazard for oil and natural gas transmission pipelines, they have been specifically designed for this hazard only in recent years (Refs. 1, 2 and 3). A decision to use special fault-crossing designs for pipelines should consider fault activity, length of fault-crossing zone, rupture consequences, environmental impacts, exposure to other hazards and cost.

Two approaches to the design of a pipeline crossing an active fault or active fault zone can be considered for the situation where the probability of large fault displacements during the design life is low. The first approach, often voiced by regulatory requirements, is deterministic and requires a design to resist pipeline failure resulting in rupture caused by fault displacements expected to occur during the design life. For example, the federal stipulations governing the Trans Alaska Pipeline system (Ref. 4) state that where the pipeline crosses an active fault zone, the pipeline shall as a minimum be designed to resist failure resulting in leakage from two feet of horizontal and/or vertical fault displacement. The second method, and the one coming more into use, is probabilistic. It requires accepting a low risk of pipeline rupture and the associated consequences, and taking steps to minimize them.

Strategy Choice

In situations where the probability of fault movement is comparable to the level selected for design for other hazards such as ground shaking, or where in populated areas even a low level of risk cannot be accepted, the deterministic method is appropriate. On the other hand, when the probability of fault movement during the design life is very small, for example, an order of magnitude less than the ground shaking hazard and in remote areas where the consequences of fault-induced rupture are not great, the probabilistic approach appears to be preferable.

When the probability of fault movement is between these two bounds, the

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fault-crossing design decision should be fundamentally based upon a cost analysis as well as consideration of the consequences of pipeline rupture and associated impacts particular to specific fault-crossing designs.

In many cases the probabilistic approach is applicable and preferable. It requires a probabilistic assessment of the several fault parameters, the most important of which is the probability of occurrence of fault movements of various amounts of displacement at the pipeline crossing during the design life.

Ideally, given such a fault-hazard assessment, the total probability of occurrence of all amounts of fault movement at the pipeline can be compared with the acceptable risks previously established for the project for other natural and man-made hazards. On this basis it may be possible to show that the fault-displacement risk is acceptable and needs no further consideration in design. In this situation, however, measures need to be taken to minimize the consequences of rupture including the following: 1) Development of an emergency response plan, 2) Stockpiling of repair materials, 3) Use of heavy wall, high-toughness pipe at the fault crossing to reduce the possibility of pipe rupture and minimize its effects should it occur, and 4) Placement of mainline block valves on either side of the fault-crossing zone with automatic, remote shutoff capabilities.

If it is shown that the fault displacement risk is unacceptable, or if no definition of acceptable risk has been established, additional evaluation to properly select a design mode for the fault crossing is warranted. The following sections address the essential elements of such an evaluation as might be applied to an oil or gas transmission pipeline.

FAULT-CROSSING DESIGNS

For a given fault offset, the probability of rupture for buried pipe depends upon the burial depth, fault-crossing geometry, type and strength of surrounding soil and pipe material properties. In most soils (other than rock or well-bonded frozen soils in arctic areas) conventional pipe burial and favorable fault-crossing geometry can be shown to accommodate fault offsets of several feet without rupture (Ref. 3). At increased burial depths and in rock cuts conventional pipe burial has reduced capacity to withstand fault offset and, in extreme conditions, has essentially no such capacity. In these cases, special fault-crossing design modes may be considered such as (Ref. 5):

- Placement of pipe in an aboveground embankment constructed of low-strength soil
- Placement of pipe in an oversized ditch surrounded by low-strength, crushable material or selected backfill with enhanced slip planes
- Burial of pipe within a sacrificial culvert or oversize conduit
- Placement of pipe on aboveground sliding supports.

It should be noted that the cost of constructing sacrificial culverts or conduits can be quite high since they are sized internally to accommodate the

pipe with sufficient free rattle space to allow for fault displacement. In fact, any special fault-crossing mode is generally more costly than conventional burial, although special design modes will reduce the probability of pipeline rupture given a fault offset. Further, the environmental consequences may be more severe for the aboveground modes than is the risk of rupture for a buried mode, even with no consideration given to fault design capability. These issues are discussed in later sections.

In order to select a design mode for a fault crossing, it is first necessary to estimate the conditional probability of pipeline rupture for various amounts of fault movement for all fault-crossing modes considered appropriate. Because analytical or test data do not currently exist to accurately define such probability estimates for all conditions, the values may be determined qualitatively by judiciously assessing how well each fault-crossing mode would perform relative to a conventionally buried pipeline. To account for uncertainties in this assessment, both best-estimate and upper-bound conditional rupture probabilities should be determined for later use in a sensitivity evaluation.

The product of the fault displacement probability and the conditional rupture probability is equal to the probability of pipeline failure during the design life for each fault-crossing mode considered, including conventional burial. If the pipeline crosses more than one active fault or fault zone, the total probability for more than one occurrence of pipeline rupture from all fault movements during the design life should be evaluated and factored into the evaluations. The pipeline failure probability as computed above for each fault-crossing mode is used in the cost analysis described next.

COST ANALYSIS

The cost of each fault-crossing design alternative, including conventional burial, is determined and should include the total cost of construction and those costs associated with the occurrence of a fault-induced pipe rupture during the design life. For the comparative cost analysis, each special fault-crossing design is evaluated on the basis of total added cost (above a no-mitigation design case) equal to the cost of construction of the mitigative design plus the present value of future fault-induced probabilistic losses arising from pipeline repair, revenue loss and gas loss minus the corresponding costs for a no-mitigation design. The analysis should include growth, inflation and discount rates. Losses may be assumed to be equally likely to occur in any year of the design life. Repair costs include field mobilization and demobilization of crews, equipment and materials; excavation or removal of ruptured pipe; replacement, welding, inspection, purging and equalizing pressure of the pipe; removal and replacement of culvert or conduit, embankment or aboveground structure, if necessary; backfilling; and restoration.

The present value of each of the probable losses (PVL) due to fault movement may be determined using an approach suggested by Oppenheim (Ref. 6), modified to account for growth rate, as follows:

$$PVL = AEL (P) \sum_{i=1}^t (1 + GR)^i (1 + MARR)^{-i}$$

where: AEL = annual expected loss = loss/design life in years,
 i = index over each year of pipeline life,
 GR = annual growth rate associated with a loss,
 MARR = discount rate or expected rate of return,
 t = pipeline design life in years, and
 P = probability of pipe rupture in t years.

A cost comparison is then made between construction cost plus the PVL of each loss, evaluated for conventional burial and each special fault-crossing design mode under consideration. Assuming the conventional burial and all of the mitigative designs evaluated are otherwise acceptable, the lowest total cost design is preferable.

Cost Sensitivity

Before drawing hard conclusions from such analyses, studies should be conducted to assess the sensitivity of the results for realistic ranges of the important input variables such as probability of occurrence of the loss; initial construction costs; and general growth, inflation and discount rates. Factors affecting the probability of occurrence of the loss include the following:

- Probability of occurrence of the fault movement
- Uncertainty of best-estimate conditional rupture probabilities
- Burial depth
- Percentage of the pipeline alignment surrounded by high-strength material such as well-bonded frozen soil in arctic areas or bedrock
- Spacing of fault-crossing isolation valves.

The results of the cost analysis previously outlined will tend to show that conventional pipe burial for fault crossing is preferable when: 1) The length of a fault-crossing zone is great (on the order of several miles), which dramatically increases the construction cost differentials between conventional burial and special fault-crossing designs, and 2) The probabilities of pipe rupture are sufficiently small, which reduces the influence of the probable losses. The length of the fault crossing can be great where the definition of the fault location is uncertain. In this case, construction costs tend to dominate over PVL costs. For well-defined, very short fault-crossing zones, the length of fault crossing becomes much less critical to the outcome of the cost analysis.

CONSEQUENCES OF FAULT-CROSSING DESIGNS

In addition to the cost analysis, the selection of a fault-crossing design mode should consider the other consequences of constructing the alternative modes. This evaluation might be considered in three categories: 1) Public safety effects, 2) Environmental effects, and 3) Susceptibility to other hazards.

A qualitative approach can be used to make the comparison between the various fault-crossing modes in each of these categories. This can be done by first evaluating the consequences associated with each design mode and then ranking the various design modes relative to each other with respect to each consequence. Factors to consider in this evaluation are discussed in the following paragraphs.

Safety Consequences

The major effect on the health and safety of the public due to rupture of an oil or gas pipeline is the release of contents within a localized area. However, the same amount of product will be released for any of the fault-crossing modes. Hence, all modes are considered to have an equal impact from a health standpoint due to pipe rupture.

The rupture of a gas pipeline requires the consideration of safety aspects. Both pipe whip, caused by a sudden release of pressure, and the possibility of fire need to be considered for the various fault-crossing modes. The degree of overburden or aboveground structure restraint and potential for flying debris should be assessed for each of the fault-crossing modes with their varying degree of restraint capability including modes which use sacrificial culverts or conduits. These evaluations should also recognize the remoteness of the pipeline alignment and the potential numbers of the public which may be exposed to the hazards should a pipe rupture occur.

Environmental Consequences

The environment may be affected by the various modes of fault-crossing by their influence on drainage patterns, visual impact, requirements for mineral materials and the need for greater restoration. The various fault-crossing modes will require differing quality and amounts of mineral materials, both for embankment and backfill and for concrete aggregates. Additional drainage culverts may be required for embankment modes in order to maintain drainage patterns and allow for fish passage. The extra width of backfill area should also be considered as it affects surface drainage.

The rupture of an oil pipeline presents additional considerations, especially in environmentally sensitive areas. Impacts on fish, wildlife and plant life should be assessed for each of the fault-crossing modes with their varying ability to mitigate the release of or otherwise confine the spread of oil.

Other Hazards

Depending on the fault-crossing mode adopted, the pipeline will be subjected to other natural or man-made hazards. These include:

- Vehicle impact, including accidental collisions by construction equipment, automobiles and aircraft . . .
- Vandalism and sabotage
- Foundation failures, including, for example, stability problems caused by liquefaction and slope instability.

All buried modes are preferable with respect to both vehicular collision, vandalism and sabotage. The location of the pipeline has low visibility and the pipe itself is inaccessible to vehicles. An aboveground embankment mode has greater visibility and may be more susceptible to vandalism or sabotage. It may also be possible for heavy construction equipment to damage the pipeline. The pipe placed on aboveground supports is highly visible, which will increase its exposure to vandalism, sabotage and vehicle impact.

Liquefaction and slope stability problems, if any, will most probably be induced by a seismic event and will tend to have an equally detrimental effect on any fault-crossing mode. Furthermore, slope stability and liquefaction problems induced by seismic events could have more effect on the pipeline behavior than a fault movement.

MITIGATION MEASURES

If the results of the cost and consequence analyses described in the foregoing favor crossing the fault or fault zone with conventional pipe burial, measures should be taken to mitigate potential damages and losses. These might include:

- Use of heavy wall, high-toughness pipe
- Installation of fault zone isolation valves
- Automatic line break controls
- Minimum burial depth in the fault zone
- Offsetting the pipeline alignment from nearby structures and public-use areas in the fault zone
- Emergency repair plan
- Seismic monitoring system

Some, if not all, of these measures might well be considered for modes other than conventional pipe burial.

CONCLUSIONS

Special fault-crossing designs can greatly minimize, but not completely eliminate the possibility of fault-induced pipe rupture or damage during the design life because of possible uncertainties in fault zone definition, fault movement parameters and the reliability of the designs themselves. Accordingly,

in remote areas where the likelihood of large fault movements at a pipeline crossing during the design life is low, the final decision to construct special fault crossings or the selection of a specific fault-crossing mode should be fundamentally based upon the results of the economic analysis described with lesser consideration given to the results of the consequence analysis. In cases where the economic analysis yields no clearcut benefit between candidate fault-crossing modes, the decision should be based upon the consequence analysis.

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SEISMIC ISOLATION DESIGN AND CONSTRUCTION PRACTICE

T. L. Anderson¹

ABSTRACT

Analysis, design and construction details of the seismic isolation system used for the Los Angeles County Fire Department's Fire Command and Control Facility are described. Measured isolator properties are reported and the first-of-a-kind engineered displacement control system is explained, including the design and the engagement response behavior as measured by full-scale validation tests. Design parameters, special structural reinforcement considerations and isolator-framing erection sequence are also reviewed. Reporting of these project-specific implementation details are believed critical to the increased acceptance of seismic isolation into wide spread design practice.

INTRODUCTION

The adoption of seismic isolation into the mainstream of structural design practice in the United States lags the application of this technology in other countries, especially Japan [4, 9]. The reasons given for the relatively slow pace of implementation in the United States vary from the need for additional research [5] to general reluctance to accept any new concept [7].

Efforts are underway to accelerate the use of seismic isolation through research, education, code development, public policy and related activities. In addition, an increasing number of technical papers discussing case studies and current applications are being presented at specialty seminars [3, 6] and special sessions of major conventions. Recently, the evaluation, selection and design implementation process was described by Anderson [1] for the seismic isolated Fire Command and Control Facility (FCCF) project designed by Fluor Daniel and currently nearing construction completion for the Los Angeles County Fire Department. In that paper a complete performance-based comparison is explained which allowed the selection of seismic isolation as the preferred design approach to be made on a rational, consistent criteria basis considering cost and performance. Additional such comparisons need to be reported to further encourage and guide new users to this technology.

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This paper discusses design details and special aspects of the seismic analysis and erection process for the base isolated FCCF project. It is hoped that by describing steps in the analysis and design process other potential practitioners will be encouraged to pursue seismic isolation as a viable seismic design alternative.

DESIGN DETAILS

The detailed, nonlinear dynamic verification analyses of the FCCF structure were undertaken at the completion of design, after all test data were available from the final isolator verification tests. Those analyses are reported by Bachman and Gomez [2].

Prior to completion of design, however, preliminary design analyses were conducted in order to size members, estimate in-structure accelerations and otherwise support the overall design effort. These calculations were necessarily conservative in order to avoid having to make what could have been impractical changes later in design. For example, a need to later widen the rattle space surrounding the isolated building could seriously impact the location of foundations already in place.

These preliminary analyses relied upon the then available seismic design guidelines [8], supplemented with the application of added safety margins.

Design Parameters

A worst case design maximum displacement of 9.6 inches was calculated, corresponding to the total motion of a corner of the building. This value was the result of the calculated displacement at the building C.G. plus an assumed torsión due to five percent accidental eccentricity of base shear and combining horizontal components of motion along both building axes using a 100-30 rule. This displacement was conservatively taken to be applicable to all isolator units, regardless of location.

The ultimate isolator displacement was calculated, using preliminary isolator bearing design parameters and worst-case assumptions, as 15.5 inches, the value at which a pin-pinned bearing was calculated to "roll out" or become unstable. Two subsequent developments, however, precluded instability at this displacement. Nevertheless, the 15.5 inch displacement (taken in any horizontal direction) remained the size of the rattle space which all architectural, equipment, structural and utility components, crossing this isolation plane, had to accommodate without distress.

The 15.5 inch rattle space proved later to be conservative because: 1) a fixed-fixed bearing design was selected for use which eliminated the roll out instability condition and 2) the ultimate restraint system was later designed to engage at a displacement of about 12 inches which will limit building displacements to less than the 15.5 inch value.

Wind drift response was calculated to be on the order of 0.4 inch under the maximum design wind. This value was used as the upper bound or endurance limit displacement for all systems which crossed the isolation plane and might be sensitive to fatigue loading.

Vertical isolator bearing deflection was calculated to be equal to 0.12 inch elastic plus 0.06 inch long term creep, for a total of 0.18 inch. Most elastic deflection will be realized before utility and nonstructural systems are installed across the isolation plane. Hence, the amount of permanent vertical displacement imposed upon systems crossing the isolation plane was concluded to be negligible.

BASE SHEAR

Base shear of structural and nonstructural elements depends upon their location, either isolated or non-isolated. Design values were specified as given in the following paragraphs.

Structural Elements At and Below Interface

The isolator supports and foundations were designed for a base shear of $V = 0.18W$ at code allowable stresses without the one-third increase usually permitted for earthquake load combinations. This base shear was distributed to the isolation bearings in proportion to their relative stiffness. Note that this is a conservative step designed to ensure damage free, elastic response of the foundation considering the isolation system was specified to limit the building base shear to no more than $0.15W$ under the maximum design earthquake.

In addition, the anchorage, local structure and foundations for the ultimate restraint system isolators were designed to resist the normal contribution of design base shear they would receive based upon their relative stiffness plus the reactive force they will provide in limiting the building displacement. This total force, F , is given by Eq. 1.

$$F = 0.15W \frac{K^*}{\sum_{i=1} T} + 1.25(0.15W) \frac{K^*}{\sum_{i=1} N} \quad (1)$$

where:

- F = Total force in a displacement control (ultimate restraint) isolator
- K^* = Stiffness of a displacement control isolator bearing at design displacement
- K = Stiffness of any isolator bearing at design displacement
- W = Weight of building and fixed equipment
- N = Number of displacement control isolator bearings
- T = Total number of bearings

The 1.25 factor in the second term on the right hand side of Eq. 1 reflects the 125 percent additional base shear resistance specified to be provided by the ultimate restraint system (see Reference 1).

The above loads go well beyond the intent of building codes and, hence, the code allowable stresses and load factors were relaxed somewhat to enable more of the available material safety margins to be utilized to resist these highly unlikely load levels. Accordingly, the above loads are resisted at yield stress levels in steel, using unity load factors for strength design in reinforced concrete and using an appropriate increase factor (on the order of two-thirds) for lateral resistance of foundations.

Nonstructural Elements At and Below Interface

Nonstructural components at and below the isolator interface, viz., items not isolated and subject to full ground motion, were conservatively designed for a seismic force of $F_p = 1.0 W_p$ at code allowable stresses, including the one-third increase. This represents the peak of the five percent damping design response spectrum for the free field. Although dynamic analyses might justify use of a smaller value, the design goal was to maintain elastic response of utility systems, telecommunication links and other vital lifelines in an earthquake. In this way functional operability was easier to ensure.

Structural Elements Above Interface

Structural components above the isolation interface, viz., the isolated structure itself, were designed for a base shear of $V = 0.15W$ at code allowable stresses, including the one-third increase for seismic effects. This value stems from the same base shear limit specified for the isolation system itself.

Nonstructural Elements Above the Interface

Nonstructural components above the isolation interface should qualify for the benefits of reduced in-structure accelerations and corresponding relaxation of bracing/anchorage requirements. Calculated typical maximum in-structure accelerations for equipment design were on the order of 0.15 g. However, in the absence of code language for in-structure equipment, a decision was made to design and anchor nonstructural components for a seismic force of $F_p = 0.3W_p$ at code allowables, in accordance with the Los Angeles County Code Seismic provisions for conventional fixed base buildings. It is believed that a formally promulgated seismic code, specifically dealing with isolation, would establish more realistic equipment design values, fully accounting for the potential benefit of seismic isolation in substantially reducing in-structure acceleration levels.

SUPERSTRUCTURE DETAILS

When witnessing a seismic isolator test to full lateral design displacement the observer is made keenly aware of the large P-Delta moment developed by the product of the laterally displaced system and the vertical column load. This moment adds to the lateral isolator reaction acting through its vertical lever arm to produce what can be very large bending moments in the building column above the isolator.

This section of structure could easily become the weak link in the lateral force resisting system if it is not properly reinforced to carry these forces and redistribute them into the beams and girders framing into the column at the level just above the isolator. For the FCCF project, the decision was made to reinforce these areas so as to limit stresses to yield levels and, thereby, keep the structure damage free. The resulting structure is heavily reinforced as seen in Fig. 1. In addition, Fig. 1 also shows a second set of vertical stiffeners, placed outboard of the first. Their purpose is to act as locations for jacks that will be used to lift the building frame in order to remove the load from the isolator and permit removal and replacement of the seismic isolation bearing units for inspection purposes as needed.

ISOLATOR PROPERTIES

The project performance specification [1] required a system vibration period in the range of 1.5 to 2.5 seconds for motions at the design displacement. The design displacement was specified as a function of equivalent viscous damping, as measured at the design displacement.

Measured properties of isolation bearings, taken during initial tests of prototype bearings and later verification tests of production units scheduled for installation in the building, were consistently good. Stiffnesses (horizontal and vertical), equivalent viscous damping, stability of hysteretic loops, etc., were all within project specifications. Load-deflection data are shown in Fig. 2 for a typical test sequence in which isolator displacements were increased in increments after each three cycles of motion. The stiffness and energy dissipation properties of the isolators are stable and very repeatable, settling down to a steady-state value in no more than three cycles.

An equivalent viscous damping value of 15 percent of critical, taken at the design displacement, was used for seismic analyses as a result of such tests. Seismic analyses based upon viscous and upon hysteretic damping are discussed in a separate paper by Bachman and Gomez [2].

DISPLACEMENT CONTROL

The function of the FCCF operation will be to receive and dispatch fire and medical aid in response to 911 emergency calls for Los Angeles County. As project architect-engineers it was Fluor Daniel's mission to use best-available-technology to meet this goal and to take steps to mitigate loss of facility function should even the very conservative design levels be exceeded.

The seismic isolator design criteria drew upon the fundamental structural design principles of providing both ductility and reserve strength to resist overload conditions and preclude sudden collapse. For the isolator system the "ductility" is provided by a large margin of additional isolator displacement capacity (about 80 percent margin) above the maximum calculated design requirement. Reserve strength is provided by a "strain hardening" displacement control device built into all isolator units located on the building perimeter.

The displacement control isolator is a patented device made up of a length of chain attached at each end to the top and bottom end plates of the isolator and located in a central cylindrical hole. The chain remains slack under most lateral displacements, even up to and well above the design lateral displacement.

When the lateral displacement of the isolator reaches about 12.5 inches, the slack in the chain is taken up (see Fig. 3) and the tensile force in the chain now provides additional resistance to further horizontal shear displacement of the bearing. Because of the inherent load-deflection properties of chain links as they take up load, combined with the secondary vertical shortening of the isolator as the chain is tensioned, the resulting four-fold increase in the lateral stiffness of the isolator is very smooth and without impact as the displacement control chain is engaged (see Fig. 4).

The displacement control devices were designed to provide an additional 125 percent of base shear capacity. Superstructure isolator attachments and foundations were designed to carry these forces without damage as well. Because of the large margin of displacement capacity built into the bearing before the displacement control is engaged, it is highly unlikely this ultimate restraint system will ever be loaded.

SEISMIC RESPONSE UNDER EXTREME SHAKING LEVELS

The minimum displacement safety margin available in the isolator units is the difference between the calculated maximum design displacement of 5.1 inches at the building c.g. and the point at which the displacement control chain begins to engage, at 12.5 inches. Accounting for two directions of horizontal ground motion plus accidental and real torsion results in a maximum differential displacement of 7.1 inches at a building corner. Bidirectional and torsional response will cause a different pattern and magnitude of motion at each isolator. Assuming a representative value is given by the average of the c.g. (5.1 in.) and maximum (7.1 in.) displacements, results in a 100 percent displacement safety margin above the 500-year design earthquake.

An additional seismic performance requirement was to accommodate a 1,000-year event [1]. The site-specific geoseismic study [10] indicates a 19 percent increase in spectral displacement between the 500-year and 1000-year events. Using the above argument for representative displacement results in a displacement safety margin of at least 70 percent above the 1,000-year event displacement demand.

Linear extrapolation using the probabilistic hazard data for the site [10] suggests that it would require in excess of 4,000-year return period ground motions to fully displace the structure so as to engage the displacement control backup system. Even larger ground motions would be required to load up the displacement control bearings to their design level of an added 125 percent of base shear resistance. These would be very large but low probability of occurrence motions. Building response under such extreme conditions will be reported in the future at the completion of ongoing nonlinear studies.

ISOLATOR AND SUPERSTRUCTURE ERECTION

Erection of a base isolated structure is a most interesting challenge for which there seems to be no help available in the literature. The Fluor Daniel design team, building general contractor, steel fabricator and seismic isolation system supplier jointly worked out a methodology, starting from scratch, based upon a combination of traditional building erection practices and innovation. After considerable deliberation it was decided to erect the structure by first attaching the isolation bearings, with their mating end plates, to the superstructure columns in the steel fabrication plant where bolt torques and weldments could be made under controlled shop conditions. The assembled base plate, isolator and column elements were trucked to the site as a unit and readily erected by crane onto preset anchor bolts, then held in place with anchor nuts (see Fig. 5). The process took no more than 20 minutes for each two-story column-isolator assembly.

Horizontal framing members were added next. Fit up was much easier than with conventional fixed-base steel construction due to the flexibility of the isolators which provided the ironworkers with an element of "softness" they had never experienced before.

Horizontal framing completion was followed by placement of vertical bracing members to complete the steel erection process. Much to the surprise of the building erector, the actual total time for structure erection was about half of the estimate made during planning (two weeks total compared to a four week estimate). The lesson learned from this experience is that, for steel framed structures, the seismic isolator units can be considered as an extension of the column base plate with the assembly erected no differently than "traditional" fixed-base construction.

Following frame erection, the balance of erection and installation of structural, utility and architectural components followed conventional practices. It should be pointed out, however, that conventional practices for layout, entry and construction of building utilities typically are not consistent with the need to maintain an unobstructed "rattle space" for free movement of the superstructure during earthquake ground motions. Building officials, inspectors and public works engineers need to adjust their thinking to accept large relative building displacements as the "norm" for a seismically isolated structure. Furthermore, all utility systems which across the isolation interface need to freely accommodate the design motion without distress. A wide variety of design and hardware options are readily available to make this possible for all utilities, including electrical ducts and sanitary sewer lines, which typically are constructed as relatively rigid systems.

CONCLUDING REMARKS

Widespread use of seismic isolation as an accepted design alternative is likely several years off. Yet the benefits to be derived in maintaining functionality and protecting capital investment while also meeting the traditional code objective of collapse protection of buildings cannot be ignored. Clearly, the maintenance of function and protection of community lifelines (transportation, communication, utilities, etc.) was given a heightened level of public awareness by the October 17, 1989 Loma Prieta earthquake. A move to add performance requirements to seismic building codes would greatly accelerate the implementation of seismic isolation and other earthquake damage mitigation technologies.

This paper describes design and construction details and special analyses undertaken in the implementation of seismic isolation for the Los Angeles County Fire Command and Control Facility. Reporting of these details is considered essential in recruiting first time users to this technology and the accelerated and expanded use of seismic isolation on all projects where it is justified, applicable and feasible.

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FIGURE 1. Installed seismic isolation bearing showing superstructure reinforcement.

FIGURE 2. Typical measured lateral load-deflection curves for a pair of isolation bearings.

FIGURE 3. Isolation bearing deflected to engagement of displacement control system.

FIGURE 4. Measured lateral stiffness of displacement control isolator bearing at large displacements.

FIGURE 5. Erection of isolator-column assembly as a unit.

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Seismic Isolation for the
Los Angeles County
Fire Command and Control Facility

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ABSTRACT

The use of seismic isolation is becoming an increasingly important consideration in improving the earthquake lifelines of a community. This paper identifies the major problems encountered in the seismic isolation selection and implementation process, describes in detail the approach followed on the Los Angeles County Fire Department Fire Command and Control Facility Project, and provides a set of recommended guidelines for future projects.

INTRODUCTION

Seismic protection of function has become increasingly important for such projects as hospitals, utilities, pipelines and emergency facilities. Historically, the need to protect critical operational systems or components was met with seismic hardening (of structures) and qualification (of equipment) programs. These activities were, and are, an important part of the evolving field of lifeline earthquake engineering. Recently, however, the well established concept of vibration isolation has been applied as a means to provide this protection. A summary of the development and use of seismic isolation since its inception near the turn of the century is presented by Kelly, 1986. There has been little discussion in the literature of how one approaches the possible use and selection of seismic isolation for a major, critical facility project.

This paper describes the seismic isolation evaluation, selection and implementation process for the Los Angeles County Fire Department Fire Command and

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Control Facility (FCCF). Fixed base and base isolated concepts were compared during design development, both on the basis of functional integrity and lifecycle cost. The selection of a base isolation approach was followed by a formalized decision analysis process which defined the specific project needs and led to a detailed performance specification. Because of the critical lifeline role of the facility, a double contingency approach to safety margin was adopted. This led to the incorporation of an ultimate restraint system as an integral part of the base isolation design. The design process benefitted from the helpful feedback of a peer review panel and close cooperation with both fire department and building department officials. It is hoped that this paper will provide encouragement and guidance to the design and user community and, thereby, accelerate the adoption of this technology to the welfare of the public.

FOUNDATION CONCEPTS

First principles, as applied to vibration engineering, dictate use of either a fixed-base foundation or vibration isolation for control of all vibrations. The latter is widely used whenever the response exceeds threshold values based upon equipment operability, human discomfort or other suitable criteria. Base isolation of equipment is standard practice in virtually every commercial and industrial building. Further, many entire buildings in Europe and the United Kingdom have been base isolated to successfully decouple them from harmful or annoying ambient vibrations arising from nearby railroad operations (Kelly, 1986). In addition, on the order of one hundred buildings and structures worldwide (schools, office buildings, nuclear power plants, bridges, etc.) have been founded on base isolation (Tarics, 1987) to decouple them from earthquake motions. A "boom" in construction of base-isolated buildings is currently underway in Japan, where more than ten buildings have been constructed using seismic base isolation, with nearly that same number of buildings under planning or in design (Yamahara, 1987).

Operability Requirements.

With this heightened interest in seismic isolation, it was not surprising that seismic isolation was considered as an alternate design concept in parallel with a fixed base design during the design development phase of the project. A key project requirement, stated

throughout the tender documents, was the goal to obtain a facility that would perform its intended function during and following all likely earthquake events. This requirement is driven by the fact that the FCCF will house the consolidated dispatching operations of the Los Angeles County Fire Department. It will be an indispensable link in the County earthquake lifelines, directing county-wide fire and medical emergency relief services following a major earthquake and its aftershocks. It will house computer-aided dispatch operations for response to 911 calls, a variety of data and voice communications systems, administrative offices and self-contained living quarters. This critical service demanded use of the best available technology in structural dynamics.

Problem Areas.

Fundamentally, it is rather straight forward to understand the detuning that seismic isolation provides and the resulting trade-off of displacement for acceleration -- a principle that applies to all dynamic disturbances. Problems, however, arise in a number of areas for the first-time practitioner.

Nonlinear behavior of the materials involved and high damping levels can rule out the use of our conventional rules-of-thumb and can render our normal intuition based upon previous linear experience invalid.

Lack of definitive design manuals, applications catalogs and documented performance data, as is abundantly found in equipment vibration isolation design applications, is a severe handicap.

The absence of code-approved design guidelines which can be adopted by building officials as a basis for their plan checking, and the uncertain need for third party review can be unsettling. A related need is for purchase specification information in the form of performance requirements, and QC/material requirements. Clearly the technical community disagrees on the level of dynamic analysis required to support a seismic isolation design. Isolator vendor claims and counterclaims regarding their products can also be confusing.

Surprisingly, engineered back-up features are generally lacking on virtually all seismic isolation systems installed to date. Several ultimate restraint system concepts have been developed and/or tested, but

the requirement of an ultimate restraint system that takes over in a controlled way, in the event the design ground motions are exceeded, has not generally been employed. In the author's opinion this is an unsettling attitude and seems inconsistent with the proven approach taken in fixed base design where overcapacity is ensured through a combination of strength and ductility.

All of these elements taken together, from the sparsity of design and application information to uncertain guidance for purchase specifications, are the real constraint to integrating seismic isolation into the design process. Most issues are addressed in the following sections by describing the approach used on the FCCF project.

DESIGN DEVELOPMENT

Project Seismic Criteria.

In order to develop both fixed base and base isolated designs in parallel for comparative purposes, consistent criteria covering both concepts were developed. The purpose of the seismic design criteria was to achieve a design of the building and its contents to withstand the effects of earthquakes, without loss of mission function, in a safe, balanced and economical manner. Furthermore, the criteria were selected in a very conservative manner such that the risk of functional loss of the facility was judged to be consistent with the consequences associated with such a failure.

The seismic criteria incorporate the requirements of the Los Angeles County Building Code (LACC, 1987). However, such building code seismic provisions are generally intended to assure life safety, not continued operability. Operability requirements demand design levels greater than the code and invoke detailed consideration of non-structural building contents to be certain they are properly anchored or braced to ensure their performance and prevent sliding, tipping and falling hazards. In all cases, the operability-based seismic criteria far exceed the code requirements.

Because of the importance of the FCCF mission, it could be argued that even a low level of risk cannot be accepted. Invoking this argument results in the use of a deterministic design criteria, one that requires design to one or more postulated earthquakes expected to occur during the facility life. Choice of the

preferred approach, probabilistic or deterministic, has been the subject of continuing debate in the technical community. Accordingly, both were used.

While it is believed that the probabilistic method for setting design criteria is both sound and applicable to this project, it does require accepting a risk (however low it may be) that the actual earthquake ground motions could be greater than the level used for design (especially at longer periods comparable to the isolation period). To mitigate this occurrence, the design criteria require the facility to have substantial reserve margin and to employ an ultimate restraint system for a base isolated case.

In brief, the seismic design criteria was based upon both the LACC and a site-specific design criteria, whichever governed. The site-specific hazard was determined for a specified risk level, well in excess of implied code values, and it was used in conjunction with elastic design of the facility and seismic hardening of contents. Further, the probabilistically-determined seismic shaking levels were not permitted to fall below the effect of estimated maximum earthquakes on the most significant faults whose motion most affect both long and short-period structures at the site. The site-specific seismic exposure includes all of the well known faults in and around the south coast basin of Southern California, e.g., the Whittier at 6 miles, the Newport-Inglewood at 12 miles, and the San Andreas (South Central) at 32 miles.

The seismic criteria are as follows:

1. Fixed-Base Design

- o 1987 LAC Building Code.
- o Site-specific ground motions for a 487-year return period and design to code-allowable stresses.

2. Base-Isolated Design

- o 1986 Structural Engineers Association of Northern California, "Tentative Seismic Isolation Design Requirements," (SEAONC, 1986).
- o Site-specific ground motions for a 500-year return period and design to code allowable

stresses, not to be less than 1000-year return period ground motions at yield threshold.

The site-specific ground motion criteria govern design for both cases, and it can be shown that the functional limit for the facility corresponds to about a two percent probability of exceedance during the 25-year design life for both design cases.

Fixed Base and Base Isolation Compared.

A detailed study was undertaken in order to define the expected seismic performance of both fixed-base and base-isolated buildings from the perspective of building response, equipment performance and contents safety.

Building. Given that the seismic design criteria require the building to remain essentially elastic during the maximum design earthquakes, the seismic performance of the building frame can best be judged by examining the calculated interstory drifts (Ferritto, 1984 and Scawthorn, et al, 1981). The estimated fundamental periods, base shears and drifts for the steel braced frame building as base isolated and fixed base are shown in Table 1.

Table 1. First Mode Period and Calculated Drifts for Base-Isolated and Fixed-Base FCCF

	Base-Isolated	Fixed-Base
First Mode Period (sec)	2.0	0.2
Base Shear	0.14W	0.57W
Drift		
2 - R	0.00006	0.0004
1 - 2	0.00020	0.0018

Detailed studies of damage evaluation (for example, Anderson and Nyman, 1977) indicate that the threshold of damage related to interstory drift is about 0.001. It can be seen from Table 1 that both foundation schemes generally exhibit drifts below this threshold. Thus, it is concluded that both fixed-base and base-isolated building frames would not experience drift-caused damage in an earthquake. Note, however, that the fixed-base building frame is sized to resist a lateral force about four times greater than the base-isolated frame.

Equipment. The seismic design criteria for equipment would require all fixed equipment, piping, HVAC ducts, cable trays, computer access floors, and the like to be braced, anchored or otherwise suitably supported (seismically hardened) to resist maximum expected accelerations from earthquakes. Preliminary estimates of these accelerations are given in Table 2 for both fixed-base and base-isolated concepts.

Table 2. Floor Accelerations for Base-Isolated and Fixed-Base FCCF

	Maximum Accelerations (g)	
	Base-Isolated	Fixed-Base
Roof	0.08	1.05
Second Floor	0.08	0.85
Ground Floor	0.08	0.38

The floor accelerations are seen to be amplified and increase with height for the fixed-base case. This is typical for buildings and also true for the base-isolated case. However, the amplification is non-existent, and the increase with height is negligible for the latter.

Clearly, the seismic hardening requirements for equipment in this fixed-base facility are high -- approaching one g. For comparison, typical hardening of non Category I equipment for nuclear power plants in California is to a level of about 0.5g lateral force. The LACC would require anchorage of major nonstructural elements to 0.3g to 0.45g.

By contrast, the acceleration requirements for the base-isolated case are of such a low level that they could be met without the need to undertake detailed anchorage analysis of each item. Rather, conventional code practice would be more than adequate. This is so because the base-isolated foundation reduces the accelerations reaching the building to a level of about 20 percent of the ground accelerations.

Experience from past destructive earthquakes has shown that equipment which remains anchored during ground shaking is not functionally damaged (Anderson and Nyman, 1977). This positive experience, combined

with observations of extensive seismic qualification testing, is convincing evidence that seismic hardening for realistic in-structure acceleration design levels coupled with appropriate design criteria can ensure the functional integrity of fixed equipment in the design earthquake. Hence, it is believed that the risk of loss of function can be made essentially nil for either fixed-base or base-isolated concepts.

Contents Safety. In a well earthquake-engineered facility the greatest seismic hazard is represented by unanchored equipment, furnishings and materials which are subject to sliding and overturning or being thrown from storage shelves in a strong earthquake. This is a potential safety hazard to personnel and may damage nearby critical systems which are properly anchored.

The reasonable solution to this potential problem is to implement administrative type earthquake damage prevention measures. Examples would include, in addition to installing positive anchorage wherever practical, storage of heavy and/or critical items and materials on lower shelves; installation of face bars to restrain shelved items; minimizing ad hoc storage of materials in areas where their tipping or sliding could pose a safety or impact hazard; and storage of portable (wheel-mounted) work benches, when not in use, in an area where they can be restrained from overturning.

Clearly, the importance of carrying out such recommendations is directly related to the shaking levels expected within the building. Regardless of design effort, it is believed that some readily repairable nonstructural damage and nuisance interrupts could occur in strong earthquake shaking. However, the potential for this likelihood is much reduced for a base-isolated building.

Intangibles. The foregoing reviewed expected functional integrity of fixed-base and base-isolated foundation concepts. For those aspects that can be quantified, it is concluded that both concepts can be designed to provide the integrity needed. There are, however, certain intangible aspects of using seismic base isolation that should be noted. Their influence on a design decision is subjective.

It is firmly believed that seismic base isolation provides an extra measure of functional integrity insurance. Stated another way, it means seismic hardening of building contents is less of a concern.

Furthermore, the reduced level of shaking in a base-isolated facility should greatly reduce the potential for impaired operator performance. Human reaction and skill level during and following strong shaking is not well defined. In fact, it may be necessary in a base-isolated building to install a special alarm, triggered from outside the building, to alert occupants to the fact that a great earthquake has just occurred.

Seismic Loss Estimate.

Advances in probabilistic assessment of earthquake hazards has made it possible to implement the acceptable risk approach in design decision analysis. It has been applied on a number of critical projects (Anderson, 1984 and Spellman, et al, 1984) to aid design decisions on an economic basis. It was used on this project to evaluate probable future seismic dollar losses associated with fixed-base and base-isolated designs. The present value of these future losses added to the initial construction cost estimates equals the estimated life cycle cost for each design alternative. Assuming each alternative is otherwise acceptable, the lowest life cycle cost design is preferable.

Use of conditional damage probability recognizes the fact that the probability of failure is far less than the probability of occurrence of a prescribed event as a result of the use of safety factors, modeling simplifications, minimum material strengths and other conservatisms (Warner, 1981). Values for conditional damage probabilities are difficult to quantify and always require use of expert opinion in their formulation.

In the emerging field of seismic damage evaluation, conditional damage probability is approximately represented by the term damage ratio. A recent, very detailed study of seismic design costs and loss estimation was completed for the U.S. Navy (Ferritto, 1984) for low-rise steel framed buildings. While it is believed that such values are very conservative for a well engineered facility, their common basis of development makes them quite suitable for comparative studies.

Using this methodology applied to the floor accelerations given in Table 2, the present value of expected seismic damage losses was computed for fixed-base and base-isolated cases. The results reflected a 40 times reduction in seismic losses with base isolation.

Before drawing hard conclusions from such analyses, the sensitivity of the results to realistic variation of the input variables must be made. Such side calculations were made, varying initial costs and cost element distributions, damage ratios and event probabilities. In all cases, base isolation showed the lowest value for the present value of estimated future seismic losses.

As cautioned earlier, numerical quantification of predicted cost of earthquake damage is difficult, and the results lack the test of extensive field correlation. From a comparative point of view, however, the reduction achieved with base isolation in seismic losses translates into a significant improvement in the FCCF to provide continuous service in and after an earthquake.

Construction Cost.

In developing the construction cost estimates for both the fixed base design and the base isolated design, a non-biased approach was taken. Each system was evaluated separately by developing preliminary structural calculations for each design concept. The basic building layout, floor-to-floor heights, and site location remained the same for both estimates. When evaluating the cost of the base isolators, preliminary quotes from five base isolator vendors on five different types of isolators were obtained. The most conservative approach was taken by using the highest quote and then adding installation costs.

A cost comparison showed the base isolation concept to be about six percent less than the fixed base design. The primary reason for the cost difference was the savings realized when seismic hardening and certification of the HVAC and electrical equipment was not required for the base isolated design.

Due to the contractual commitment to design the FCCF to remain operational after a major earthquake, certification of the mechanical and electrical equipment was required in the fixed base design to ensure that the equipment would meet the seismic functional requirements. Accordingly, seismic hardening multipliers were developed based upon past experience and manufacturers' recommendations for essentially Class I nuclear design. Typical values used in establishing equipment cost are as follows:

- o Computer Room HVAC Unit = 1.2
- o Chillers = 3.0
- o Air Handling Units = 1.5
- o Cooling Tower = 1.5
- o Boiler = 3.0
- o Electrical Equipment = 1.15

For example, normal equipment cost (used in the base isolated case) would be factored by these multipliers to establish the hardened cost for the fixed base case.

Design Recommendation.

All of the above technical and economic evaluation factors clearly favored seismic isolation as the preferred design approach. Accordingly, the detailed engineering phase was started with this decision in place.

It should be pointed out, however, that this project brings together a unique set of project-specific objectives and conditions (for example, bedrock, site, high seismicity, damage-sensitive building contents, critical facility function and risk avoidance) seldom encountered in practice. Other conditions could result in comparative studies which favor a fixed base design or are inconclusive. Regardless of outcome, all such studies must be recognized as only an aid to judgement.

DETAILED DESIGN

Base Isolation Selection.

The next step in the design process was to undertake preliminary design evaluation of candidate seismic isolation systems with an eye towards selection of a "best" or optimum system, if possible. It soon became apparent that generic design information and applications guidelines were extremely difficult, if not impossible, to locate. Although seismic isolation suppliers are eager to provide materials supporting their products, there is no consistency of this information between suppliers. Consequently, direct comparisons between product technologies are difficult to make.

Further, some isolator suppliers also offer advisory services geared to helping the first-time practitioner evaluate the applicability of seismic isolation to his or her project, and selecting a suitable system. The latter service seems to be in conflict with the former.

Finally, for the first-time user, there are no codified design guides or requirements, and there is no apparent agreement on either the complexity of analysis required for the design verification of a seismic isolation system or on the specific computer programs one should use to perform such analysis.

Given enough time and effort, one can move up the learning curve to a point where a sufficient level of understanding is achieved to draw conclusions, make critical comparisons, carry out the design process, recognize performance limits, and understand the several design-performance trade-offs. Our project was fortunate to have sufficient time for this learning process to take place. Other projects may not be so blessed.

During the information gathering and preliminary design phase, several requirements evolved for the seismic isolation system. Some were "must" requirements, others were "needs". The three "must" requirements of a candidate base isolation system were: 1) Reduce base shear, 2) Reduce accelerations, and 3) Be shake table tested. The latter requirement was critical, in our view, in order to quantify the frequency characteristics of the vibratory motion which an isolated building actually experiences. In addition, the shake table tests should reflect similar conditions with respect to isolator design, isolation period, fundamental structure period, effective damping level, and intensity of input motions including levels above design values to check "overload" response.

On this basis, five systems were considered for detailed evaluation. They were:

- o Elastomeric high loss rubber (EHLR)
- o Elastomeric high loss rubber with displacement control (DC)
- o Elastomeric with lead core (EWLC)
- o Friction pendulum (FP)
- o Friction slider with elastomeric bearings (SLIDER)

The displacement control isolator bearing (DC) is identical to the EHLR bearing, except it adds an internal link which is engaged at a pre-selected lateral displacement of the bearing. Shake table tests of this

isolator confirm its performance as identical to a high loss rubber bearing (EHLR) up to the point the displacement control is engaged.

In the course of schematic design and construction cost/schedule development for the project, preliminary seismic isolation system performance specifications were drafted to accompany a cost estimate inquiry to several seismic isolation vendors. Prior to and during the course of this inquiry effort, and as a result of later requests to the vendors and several research laboratories for additional technical information, a dossier was compiled on each seismic isolation system including applications, design procedures, field experience, test data, and performance characteristics. Investigative efforts also included in-depth discussions with vendor representatives; evaluation of a large volume of correspondence, published test reports and technical journal articles; and inquiries into several seismic isolation projects under construction or completed.

Measured Behavior.

An examination of instructure response spectra, developed from shake table tests of the candidate isolation systems, reveals some common behavior and some unusual response characteristics. All systems reflect the frequency shift to a lengthened isolated period. Highly nonlinear systems, e.g., EWLC, seem to be somewhat less effective in lengthening the period than are the more linear systems, e.g., EHLR.

As a result of the period shift, all systems provide a reduction in seismic base shear reflected by the detuning of the structure from the most intense earthquake ground motions. The ground motion components, whose frequency are at or near the base isolation frequency, are amplified, but the effective damping levels are sufficient to limit the amplified response to acceptable values.

Interestingly enough, the friction based systems have the very beneficial effect of spreading out the response frequencies over a wide band. Hence, "resonance" is not possible with any frequency. However, the sliding systems can "generate" response at frequencies that are not within the input motion itself. This stick-slip behavior produces instructure motion frequencies in the range from 10 Hz and above, the region of greatest fragility for computers and related electronic equipment.

A comparison of instructure spectra for high loss rubber and lead rubber bearings, indicates roughly equal reduction of motions across all input frequencies, with an apparent slight edge given to a purely rubber system (Kelly and Tsai, 1984). It also has been noted in tests that highly nonlinear base isolators are somewhat less effective in removing the higher frequency ground motions (Kelly, et al, 1987). This behavior is evidenced in shake table tests by higher mode response of a supported structure.

Ultimate Restraint.

Because of the critical nature of the FCCF facility function, it was deemed essential that the base isolation be backed up by at least one and preferably two overload protection systems. It is believed important to protect against the highly unlikely, but nevertheless possible, event that the earthquake ground motions could be greater than that used for design. It was decided to specify that the isolation system have a reserve displacement capability to accommodate a displacement demand 50 percent greater than the maximum, calculated design demand. If effect, this condition would be represented by about a 7000-year earthquake for this site. Beyond this margin, it was further decided to add a requirement for an ultimate restraint system that would engage and arrest further displacements in a smooth and controlled manner, without impact.

Ultimate restraint systems have been developed and tested (see, for example, Kelly, et al, 1987). Shake table tests have demonstrated a smooth transition in resistance as the displacement control is engaged. This behavior has the beneficial effect of minimizing collision-type excitation and, hence, leads to lower accelerations of the building, housed equipment and personnel, all of which are highly desirable in even this double contingency scenario. The ultimate restraint system was specified to provide an additional 125 percent of base shear capacity beyond the factor of safety of 1.5 specified for displacement. That is, the ultimate force capacity of the isolation system would be 225 percent of design base shear.

KT Decision Analysis.

The comparatively small installed cost of isolation systems relative to the cost of the building and contents, coupled with the vital lifeline importance of the facility function, made it clear that the seismic

isolation system selection should be made on the basis of technical merit, choosing that which best meets the project engineering requirements. If all systems have equal performance, cost should be the deciding factor. Otherwise, cost is a subordinate factor to performance. The objective of the decision analysis was to select the preferred base isolation system.

Fluor Daniel experience in seismic hardening/qualification of equipment, coupled with the critical need to maintain the functional capability of the FCCF, made it evident that protection of sensitive equipment was the primary purpose of the seismic isolation system. The low fragility levels of computers and communications equipment in the 5 to 30 Hz range causes one to be very conscious of the higher frequency attenuation characteristics of the various isolator types. It is believed that this is the first project to look at this property of isolators in such depth.

In addition to high frequency attenuation, there were a number of other "wants" (such as simplicity of operation, wind drift, temporal stability, etc.) for the isolator system, each with varying degrees of importance. In order to properly account for these various wants and avoid the temptation to make a decision based upon emotion or intuition, the design team employed the widely-practiced Kepner Tregoe (KT) decision analysis methodology (Kepner-Tregoe, 1979). It provides a systematic, logical approach to decision making which is considered to be complete and rigorous.

The KT process involves judgements as to (in this case) the degree to which each isolator satisfies a want objective. In brief, one establishes a list of want objectives ("wants"), characteristics or attributes one seeks in an ideal solution. These are assigned weights to reflect relative importance. The most important want (high frequency attenuation in our case) is given a 10 (highest rank); all other wants are then assigned a relative weight value from 0 to 10. Cost is eliminated by assigning it a weight of zero.

The next step in the process is to score each candidate isolation system for each want. For each want, the system that best meets the want objective is given a 10, regardless of how well the want is met. Each of the other systems are then given a comparative score (0 to 10) for that want.

The scoring matrix is shown in Table 3. Most wants reflect positive attributes where a high number reflects good performance relative to the want. Several wants, however, are in reality undesirable characteristics which one seeks to minimize or avoid. For example, "stick-slip" is an undesirable want for our case, hence the absence of this characteristic demands a score of 10.

Table 3. KT Scoring Matrix for
Seismic Isolation Solution

WANTS	CANDIDATE SYSTEMS					
	Wt	EHLR	DC	EWLC	FP	SLIDER
Simplicity	7					
Ultimate						
Restraint	5					
Wind Drift	8					
Temporal						
Stability	9					
Previous Bldg.						
Use	6					
High Frequency						
Attenuation	10					
Stick-Slip	6					
Dwell	7					
Vendor Support	4					
Cost	0					
WEIGHTED TOTALS		XXX	XXX	XXX	XXX	XXX

The following subsections provide examples of the rationale for the weighting and ranking of the candidate isolation systems on several want objectives.

- o Simplicity. A slider-based system composed of two slide plates is judged to be the most simple and, correspondingly, easy to understand. Hence, it would rate a 10. The motivation for simplicity is that simple systems are more clearly understood and, likely, more accurately modeled. On the other hand, the DC system with its internal displacement control link is judged to be the least simple of the candidates.
- o Ultimate Restraint. This want was weighted at a 5 because its use will only come into play in the highly unlikely event that an earthquake causes ground motions 50 percent greater than the already conservative design values. A system, especially engineered just for this condition, could rate a 10.

- o High Frequency Attenuation. This is the most important want. As mentioned earlier, dynamic shake table test results must be examined in order to assess the high frequency motions of the isolated structure. Friction-based systems seem to perform poorly in this area, as discussed in the previous section, and, accordingly, would be rated down on this basis.

The individual scores are entered into the matrix and multiplied by the weight corresponding to that want; the products are summed to give the weighted totals.

Candidate Systems.

On the basis of the foregoing evaluations, three systems (EHRL, DC, and EWLC) were selected for further analysis. Strictly on the basis of the numerical totals, one might select the highest scoring system. However, for this project the numerical results at this stage were rather used to short-list the field by eliminating the friction-based systems from further consideration because of their low total scores. The weighted totals of the remaining three systems were about equal.

The next step in the decision analysis process was to identify those alternatives that are unsatisfactory. This effort is called adverse consequence analysis and is designed to lead to the identification of the optimum choice.

Adverse Consequence Analysis.

In order to estimate the consequences of choosing any of the alternative candidate systems, one evaluates the likelihood that a consequence will occur and the evaluates the degree of impact of this consequence if it should happen. For example, an adverse consequence might be stated as, "If bearings fail to meet specified performance tests, the construction schedule could be impacted."

A simple index scale of high (H), medium (M) and low (L) is used for both the probability of occurrence of a consequence and the seriousness of occurrence. One then looks at the combined probability and seriousness scores as a measure of the threat of an adverse consequence. For example, an H/H adverse consequence score is generally considered grounds for rejection of that alternative. Scores of H/M or M/H must be judged

carefully as the risk with such an alternative is obviously greater than for L/L scores (or even H/L or L/H) which reflect nuisance level or low threat consequences of little concern.

The process described in the foregoing is a rather lengthy one. Much time is spent in postulating realistic consequence scenarios; the scoring then follows. The results are not shown, however, no H/H scores were identified for any of the three candidate systems.

Selected System.

Based upon the must screening, the high scoring in the want analysis, and the absence of serious adverse consequences, it was possible to recommend a single isolation system as best meeting the project engineering performance objectives. Such a recommendation was put forward and reviewed by an independent peer review panel assembled for that purpose. The panel, however, found it difficult to embrace the KT methodology applied to this kind of decision making, although they were not familiar with it. Accordingly, based upon their recommendation, the base isolation system was to be selected on the basis of responses to a request for proposals containing specific performance requirements.

Performance Requirements.

Having gone through the previous steps, it was a straight-forward next step to formulate the seismic base isolation system requirements in terms of performance, materials and QC requirements. This was so because each of these issues had been fully examined and desirable ranges of behavior established during the earlier steps. An outline of the base isolation specification is given in Figure 1. This specification was used as the basis for procurement of the seismic isolation system.

When the isolator bearing properties become available from the prototype testing program, final detailed dynamic seismic analyses will be undertaken to confirm the design values.

FIGURE 1. SEISMIC BASE ISOLATION SYSTEM REQUIREMENTS

1. Scope

- o Design basis
- o Laminated steel-rubber bearings
- o Sliding system not permitted

FIGURE 1
(Continued)

2. References

3. Requirements

- o Building layout
- o Building fixed base period = 0.2 sec.
- o Bearing vertical loads defined (DL, LL, E)
- o Vertical stiffness (10 Hz) and long term creep limit
- o Required design displacement (D) as a function of damping ratio (all bearings must accommodate 150 percent of stated value without loss of stability)
- o Isolation period = 1.5 to 2.5 seconds
- o Isolation system damping range (10 to 25 percent of critical)
- o Base shear limited to $V < 0.15W$ using site-specific spectrum and stick model provided
- o Wind drift limit = 0.4 in.
- o Ultimate restraint system to engage at 1.4D and supply a resistance of 1.25V
- o Self-restoring capability
- o Safety factors at various levels of vertical load and lateral displacement
- o Bearing attachment requirements (shimming and removal/replacement provisions)
- o Maintenance program
 - Inspection (10 year minimum)
 - Testing
 - Replacement

4. QA Provisions

- o Dynamic shake table test (including ultimate restraint system)
- o Validation testing of prototype bearings (including ultimate restraint system)
- o Determination of dynamic properties
- o Acceptance criteria
- o QC and material tests
 - Batch requirements
 - Compound tests
 - Completed bearing tests
 - * Dimensional tolerance
 - * Sustained compression (all bearings)
 - * Compression stiffness
 - * Combined compression and shear (20 percent of bearings)
 - * Acceptance criteria

5. Marking and Certification

RECOMMENDED GUIDELINES FOR FUTURE PROJECTS

The major problems encountered in the decision to use seismic isolation and the selection of a system for the Los Angeles County Fire Department FCCF project has been unique in several respects. First, the project has very stringent requirements on the protection of facility function; likely greater than the safe shutdown and containment requirements found in the nuclear power industry. Coupled with this requirement was the high capital cost of equipment housed in the facility (a factor of several times the cost of the building itself). This resulted in high conventional seismic hardening costs. Second, all parties involved in the base-isolation decision process -- County steering committee, review panel, County Fire Department officials, building officials, and project management -- have been very supportive of the extra effort involved in considering the use of seismic isolation. Next, it is believed the project served to advance the technology of seismic isolation by incorporating an ultimate restraint system as an integral part of the design requirement, serving as a back-up to the safety margin reflected in the reserve displacement capability of the bearings. In addition, for the first time a seismic isolation system was selected on the basis of high frequency attenuation characteristics (as opposed to simply a base shear reduction). Finally, the KT decision analysis methodology was applied to the isolator selection process and, although it was not used as the final selection basis, its use proved to be invaluable in clearly defining the performance requirements needed for the isolation system procurement.

It is believed that the concepts and procedures utilized on the FCCF project are suitable for other critical or lifeline engineering projects where the risk of operational loss must be kept low. Accordingly, the major points made in this paper are summarized below as recommended guidelines:

1. At an early stage, generic seismic design criteria should be established applicable to both fixed-base and base-isolated design concepts. These criteria should incorporate project-acceptable risk levels or, if none have been established, they should be based upon risk levels consistent with other environmental loads and failure consequences.

2. During the schematic design phase, construction cost estimates should be developed for fixed-base and base-isolated designs. Costs should reflect equal protection levels for critical facility contents and functions. Damage scenarios and life cycle costs should also be described.
3. If the results of each of the elements of Step 2 favor either fixed-base or base-isolated design, the design recommendation is relatively straightforward. If the results are mixed, as is the most likely case, then a KT type decision analysis (Ref. 13) is in order.
4. Contingent upon the outcome of Step 3, one may elect to use seismic isolation. Assuming this to be the case, the next step would be to develop a list of project "musts" and "wants" for the system. At this stage, careful consideration must be given to those system parameters that must be specified, e.g., maximum base shear or use of an ultimate restraint system, and those system parameters that need not be specified, e.g., maximum displacement or materials of construction.
5. A performance specification for the isolation system can now be developed based upon the outcome of Step 4 and supplemented with QC and material requirements, as applicable.
6. Using the requirements of Step 5, preliminary design analyses should be carried out for several types of isolation systems. Simple, single degree-of-freedom, equivalent linear studies generally should be adequate at this stage. Results of these design exercises will identify parameter sensitivities, general bounds of workable solutions, approximate isolator sizes, damping-base shear-displacement trade-offs available, etc., and suggest material limitations or necessary QC tests. Recycling of these studies with Steps 4 and 5 above will result in a refined and solidly based set of performance requirements.
7. Assuming more than one bid is received in response to the base isolation request for proposal, it is recommended that the technical selection be based upon a "go-no go" rating of

each system against performance requirements. If it is judged that a sufficiently precise performance specification has not been developed, then a KT decision analysis is definitely called for in order to quantify comparisons.

8. After purchase order award, detailed follow-up and review of the engineering design of the seismic isolation system is a must. Only in this way can the engineer compare actual design and test data with his preliminary design and performance limits and, thereby, validate the procedures used in the previous steps and adjust them where necessary for future reference.

CONCLUDING REMARKS

In the last several years, a good deal of technical activity, training, research and related emphasis has been placed upon seismic isolation as the "new wave" in earthquake resistant design. However, much less attention has been given to the development of design guides, codification of design standards or publication of hardware application catalogs. The result is a void in which the first time practitioner can flounder, without direction, in finding his own way. We must give attention to bridging this gap. Otherwise, the benefits of seismic isolation for the public welfare will be needlessly delayed.

ACKNOWLEDGEMENT

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The cooperation and support of Dave Rembert, Matt Gomez and Bill Parker in implementing the philosophy discussed herein is gratefully acknowledged. The work of the independent review panel made up of R. Olson, N. Donovan, R. Anderson and R. Sharpe, is also acknowledged. Special thanks are due to the Fire Department and Fluor Daniel Project Management Team for giving the author the authority and freedom of scope necessary to develop and implement the activities described.

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FLUOR DANIEL

INTEROFFICE CORRESPONDENCE

To: Bill Johnson

Date: July 18, 1991

Location:

Reference:

From: Bob Bachman

Location: 533S

Client:

Extension: 4950

Subject: Structural Response Of The
Los Angeles County Fire
Command and Control
Facility To The Sierra
Madre Earthquake

cc: Tom Anderson, 533T
Matt Gomez, 513E

On June 28 at 7:43 a.m. the Sierra Madre Earthquake struck with an estimated magnitude of 5.8 ML. The earthquake epicenter was approximately 6 miles north of Sierra Madre and 17 miles north-east of the base isolated LA County Fire Command and Control Facility. The location of the earthquake epicenter relative to the L.A. Command Center is shown in attached Figure 1. This was the first significant earthquake ground motion that the facility has experienced since it was completed in 1990. Reports from those occupying the building at the time of the earthquake indicate a rolling motion was felt, but a strong jolt was not felt. No damage of any kind was reported or observed and the building continued to function during and following the earthquake.

The building is instrumental to measure ground motion and structural response by the California Division of Mine and Geology (CDMG). A quake report of the measured motions dated by July 1, 1991, was received by Fluor Daniel on July 5, 1991.

The CDMG sensor's location, orientation, and the corresponding peak measured accelerations are shown in attached Figure 2. The measured acceleration time histories are presented in attached Figure 3. Response spectra will not be available until the later part of July or early August. In summary, the free field peak horizontal acceleration measured was 0.12g and the peak vertical acceleration was 0.06g. The measured horizontal acceleration at the foundation was 0.08g, while the peak vertical acceleration was 0.05g. The peak horizontal building response was 0.06g at the edge of the first floor, 0.07g at the edge of the roof and 0.09g at the center of the flexible roof diaphragm. The peak vertical building response

REB:sdp
(reb60)

was 0.06g at the edge of the first floor and 0.09g at the center of the 2nd floor. The estimated horizontal first mode response is observed to be a rigid body response with a fundamental period of approximately 0.9-1.0 seconds. The fundamental period of the second floor vertical response is approximately 0.1 seconds. Based on the measured fundamental period and acceleration, it is estimated that maximum isolator displacement was 0.6 inch.

There were no surprises in the measured building response. The building fundamental periods were consistent with verification analysis results and measured accelerations are also consistent with predicted values. The building period was predicted to be about 1.0 seconds to accelerations level of up to 0.1g and is predicted to shift to 2.2 seconds for acceleration levels of 0.5g. Both the higher horizontal accelerations at the center of roof and higher vertical acceleration at the center of the second floor had been predicted. In fact, it was the analysis results which prompted the instrument locations. By way of comparison, the horizontal accelerations at the edge of the non-base isolated building at the roof level are typically 2 to 3 times greater than those measured at the foundation level.

The measured acceleration levels are well below those which would cause inelastic deformations within the building. Therefore, the measured accelerations are consistent with the observation of no damage.

Once response spectra are obtained from CDMG a more detailed analysis of the building will be performed and properties of the dynamic model will be tuned as required to match the measured response. We will prepare a report describing this analysis and its results.

Figure 1. Selected CSMIP stations in the Los Angeles area.

● Stations for which records are included.

○ Stations for which peak accelerations are listed in the table.

Los Angeles - Fire Command Center

(CSMIP Station No. 24580)

SENSOR LOCATIONS

Los Angeles - Fire Command Center
(CSMIP Station 24580)

FIGURE 3. Measured Acceleration Time Histories

For presentation at the
Fourth U.S. National Conference on Earthquake Engineering
May 20 - 24, 1990, Palm Springs, CA.

VERIFICATION ANALYSIS OF THE BASE ISOLATED
LOS ANGELES COUNTY FIRE COMMAND AND CONTROL FACILITY

Robert E. Bachman^I, Matthew J. Gomez^{II}, and K. C. Chang^{II}

ABSTRACT

Verification seismic analyses of the Los Angeles County Fire Department's Fire Command and Control Facility (FCCF) are described. The FCCF is an essential facility which must remain operational during and following the most severe earthquake ground motions which can be reasonably postulated for its site. To meet this criterion, the FCCF was designed utilizing base isolation which incorporates a unique ultimate restraint system. The FCCF is a two story steel, braced frame rectangular shaped building, 188 feet long and 84 feet wide.

To verify the dynamic response of the FCCF, 2-dimensional elastic response spectra and time history analyses were performed utilizing equivalent viscous damping based on test data. These were compared with 2-dimensional non-linear time history analyses performed with DRAIN-2D in which the isolators were modeled assuming hysteretic energy absorption. Three dimensional elastic dynamic analyses were also performed. The results compare equivalent elastic and non-linear analyses for base isolated structures. Floor response spectra are also compared and the effects upon response of the FCCF caused by vertical ground motion is also evaluated and reported.

INTRODUCTION

The Los Angeles County Fire Command and Control Facility (FCCF) is a base isolated two story steel frame structure designed by Fluor Daniel, Inc. and currently under construction in the County of Los Angeles. The mission function of the FCCF is to receive emergency 911 telephone calls and to coordinate and route emergency response vehicles to needed locations. This response will be of critical importance following a major earthquake occurring in the Los Angeles Basin. Therefore, the intended purpose of the seismic design criteria for the FCCF is to achieve a design of the building and its contents which will withstand the effects of earthquakes, without loss of mission function, in a safe, balanced, and economical manner. To meet this criteria, base isolation was incorporated into the design. A discussion of the process utilized to select base isolation and the actual isolators used is described in a paper by

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Anderson[1]. Because of a short duration design and construction schedule, conservative assumptions were made regarding the properties of the isolators and the resulting design loads applied to the building structure. After the isolators were tested and building construction started, verification analyses were performed to illustrate the degree of conservatism utilized in the design of isolators and the building structural system.

PHYSICAL DESCRIPTION OF THE FCCF

The FCCF is a two story braced frame rectangular structure, 188 feet long and 84 feet wide. The building is supported by 32 steel wide flange columns which are in turn supported by 32 natural rubber bearing isolators 28 feet apart in the transverse direction and either 26 or 28 feet apart in the longitudinal direction. The first and second floors are concrete slabs supported on metal decking. The decking is puddle welded to the steel floor system to provide rigid diaphragm action. The roof consists of metal decking which in turn supports an insulating concrete slab. The roof deck was treated in design as a flexible diaphragm.

Above the isolators, the building is supported laterally by inverted chevron tube bracing located about the building perimeter. The bracing work points are offset from the horizontal beam centerlines to allow for the architectural exterior building paneling which accentuates the structural system and the base isolator system. The resulting eccentric connection moments are considered in design. A photograph of the building under construction showing the structural framing is presented in Figure 1. A rendering of the completed building is presented in Figure 2. The girders supporting the first floor are connected to the columns with full moment connections in order to resist the large seismic moments transmitted into the columns by the horizontal shear and vertical P-delta components of the base isolators.

FCCF SEISMIC DESIGN CRITERIA

The seismic design criteria for the FCCF was based in part upon the tentative recommendations of the Base Isolation Subcommittee of the Seismology Committee of the Structural Engineers Association of Northern California[2] and the 1987 Los Angeles County Building Code[3]. Since an adopted seismic code for base isolated structures did not yet exist at the time of the design, the recommendations were accepted as the best available guidance. Additional details regarding the development of the seismic design criteria are provided by Anderson[1].

The seismic design criteria adopted for the FCCF design specifies that the stresses in the structural members above the isolator shall not exceed code allowable stresses (including the normal 1/3 increase allowed for seismic loads) when subjected to site specific ground motions with a return period of 500 years. It also specifies that above isolator member stresses not exceed yield threshold levels when subjected to 1000 year return period site specific ground motions. For design purposes, the equivalent base shear coefficient corresponding to a 500 year event was conservatively taken as 0.15g. The maximum corner displacement of the building used for design was also conservatively taken as 9.6 inches. The lateral force distribution method used in the design of the FCCF is also described by Anderson[1].

Site specific design ground motion elastic response spectra[3] were developed for the FCCF by Woodward Clyde, Inc. at the request of Fluor Daniel for both 500 year and 1000 year return periods. The 500 year spectra has a zero period acceleration 0.42g, while 1000 year spectra had a zero period acceleration of 0.50g. The site is characterized as stiff clay fill underlain by siltstone bedrock. The site is characterized seismically as S1[3]. Two time history ground motions[4] corresponding to the 1000 year return period design ground motion spectra were also developed by Woodward Clyde at the request of Fluor Daniel. These time histories are based upon the horizontal scaled components of the 13 April 1949 Western Washington Earthquake. For purposes of this paper, the record identified in Reference[4] as SEGM-2 was utilized for all time history analyses performed. This record has a peak ground acceleration of 0.50g. The elastic response spectra of the SEGM-2 time history for various values of damping is provided in Figure 3 along with a comparison with the 5% of damped 500 year and 1000 year return design response spectra.

DYNAMIC MODELS

The FCCF was designed utilizing elastic plane-frame analyses in which forces were distributed assuming a constant acceleration of 0.16g (0.15g center of gravity response + 0.01g for torsion effects) at all floor levels. In addition, the P-delta moments associated with a 9.6 inch displacement were imposed at the isolator column base plate connections. The building was designed to remain within code allowables (including the normal 1/3 increase) when subjected to these loads. Analyses were performed in both the transverse and longitudinal directions.

To verify the FCCF seismic response, several different dynamic models were utilized. Plane frame dynamic models were developed in both the transverse and longitudinal directions. Computer plots of the models utilized are shown in Figures 5 and 6. Each dynamic model represents a symmetric half of the building in the direction under consideration. The transverse model includes the effects of a flexible roof diaphragm. The modeling of the isolator was complex to allow the same model to be used for both linear and full non-linear dynamic analyses. Plane frame models also allowed the response to both vertical and horizontal ground motions to be evaluated. In addition a three-dimensional model was developed for the typical building analysis approach in which vertical dynamic ground motion effects are ignored and the rigid diaphragm action assumption is utilized to reduce the number of dynamic degrees of freedom.

The dynamic mass of the building was concentrated at the floors and the roof. The idealized first floor mass, considering dead and appropriate live loads, is 1665 kips. The second floor mass is 1775 kips and the roof mass is 790 kips. In the 3-D building-type analyses, a range of mass eccentricities were analyzed in order to bound possible torsional dynamic effects.

For performing elastic dynamic analyses, the isolators were idealized as having effective lateral stiffness of 3 kips/in. Vertically the exterior isolators were idealized as having a stiffness of 1470 kips/in., while interior isolators are idealized as having a stiffness of 2438 kips/in. For performing non-linear analyses, both the interior and exterior isolator bearings were idealized with the horizontal force deflection properties shown in Figure 4. Vertical stiffnesses continued to be taken as elastic. Based on measured test

data, equivalent viscous damping of the isolators at design displacements is slightly greater than 15% for horizontal motions and 5% for vertical motions. These were the values used in the analyses performed. For purposes of comparison, analyses were performed assuming the isolators were infinitely rigid. Modeling and analysis results considering the engagement of the ultimate restraint system will be reported at a later date.

The computer code used to perform the elastic plane frame dynamic analysis was the Fluor Daniel version of SAP IV [5]. The computer code used to perform the non-linear dynamic analyses was DRAIN-2D [6]. The computer code used to perform the building type 3-D analyses was SK-COMBAT [7].

MODE SHAPES AND FREQUENCIES

The natural frequencies of the various dynamic models are summarized in Table 1. As expected, the fundamental mode horizontally is a rigid body first mode response. The fundamental mode vertically is more complex and is a combination of isolator and building beam responses. Mode shapes for the transverse dynamic models for both horizontal and vertical directions are presented in Figures 7 and 8. Also shown are the mode shapes and frequencies of the system assuming the isolators are infinitely rigid.

ANALYSES RESULTS

Both response spectra and time history analyses were performed on the linear elastic dynamic models, considering both horizontal only and horizontal plus vertical design ground motions. The vertical ground motion was taken 2/3 of the horizontal when considered. For these analyses, the first mode damping was taken as 15%, while the damping for all other modes was taken as 5%.

To obtain an estimate of torsional response effects, a range of mass eccentricities were considered in the 3-D building analysis. These eccentricities and resulting increase in corner displacements are plotted in Figure 9. Best estimates of gross eccentricities (approximately 3.4% of long dimension) indicate that center of gravity displacements and acceleration should be increased by 40% at the building corners to account for torsional response effects, while the transverse lateral bracing forces increased by 12%.

The peak response displacements are summarized and compared in Table 2. It can be seen that peak lateral displacements, not considering torsional response effects, are approximately 5.1 inches for the 500 year earthquake ground motion. For the 1000 year earthquake, these displacements increase to approximately 6.1 inches. Considering torsional effects, this displacement is 8.6 inches which is less than the 9.6 inches used as the basis for design.

The peak response accelerations from the time history analyses for the 1000 year ground motion are compared in Table 3. Horizontal response accelerations resulting from horizontal ground motion input do not exceed 0.15g at the first and second floor levels, which indicates that the performance matches closely to what was anticipated in preliminary design. When vertical ground motions are considered, resulting response accelerations are as great as 0.15g horizontally at the first and second floor levels and 1.25g vertically.

The results of the response spectra analyses in which the 500 year design spectra are compared with the 0.15g design analysis for various structural members is provided in Table 4. The horizontal spectra results have been scaled by 1.12 to account for torsional effects. The results show that for the horizontal only analysis, the computed member forces are comparable with those used for design and within design allowables considering the 1/3 increase permitted for seismic loads. It also shows that for horizontal plus vertical analysis, member forces are within design allowables considering a 1/3 increase.

Horizontal instructure response spectra were computed for the dynamic models, both considering and ignoring isolator effects. The instructure spectra are computed for 2% damping. Second floor spectra are plotted together in Figure 10. As expected, the analyses show the isolators greatly reduce and shift the instructure response to much lower frequencies. However, the analysis also shows that items which have fundamental frequencies less than 1 Hz should be properly braced since their response could be significantly amplified by the base isolation response.

CONCLUDING REMARKS

The analyses performed verify that the FCCF will meet its design criteria objectives. The forces induced in the building structural system by the 500 year horizontal design ground motion satisfy provisional code requirements. The analyses also point out the importance of the consideration of torsional effects in the evaluation of base isolated structures. Vertical ground motion structural response may be of greater significance in base isolated structures than conventional construction. Further research into the response and damping of base isolated structures considering vertical ground motion effects is recommended.

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TABLE 1. Fundamental Frequencies for Linear Elastic Model

Response Mode	Plane Frame Analysis (SAPIV)				3-D Building Model (COMBAT)	
	Transverse Model		Longitudinal Model		No	With
	No Isolator	With Isolator	No Isolator	With Isolator	Isolator	Isolator
Longitudinal	-	-	2.8 Hz	0.46 Hz	2.8 Hz	0.46 Hz
Transverse	2.5 Hz	0.46 Hz	-	-	3.1 Hz	0.46 Hz
Vertical	13.3 Hz	8.9 Hz	13.6 Hz	9.1 Hz	-	-
Torsional	-	-	-	-	7.8 Hz	0.54 Hz

TABLE 2. Calculated Horizontal Peak Relative Displacement at Center of Floor

<u>Analysis Model</u>	<u>Input Motion*</u>	<u>Horizontal Displacement (inches)</u>		
		<u>First Floor</u>	<u>Second Floor</u>	<u>Roof</u>
		(33)	(20)	(5)
<u>Transverse</u>				
Elastic w/o Isolator	500 year spectra	0	0.61	0.89
Elastic w/o Isolator	1000 year time history	0	0.82	1.1
Elastic w/Isolator	500 year spectra	4.9	5.0	5.0
Elastic w/Isolator	1000 year time history	5.5	5.6	5.6
Non-Linear w/Isolator	1000 year time history	3.9	4.0	4.0

* Horizontal displacement is not significantly affected by vertical ground motion.

TABLE 3. Calculated Peak Response Accelerations at Center of Floor - Transverse Model**
1000 Year Time History Input Ground Motion

Location		Horizontal Time History Input			Horizontal and Vertical		
		Horizontal		Non-Linear	Horizontal and Vertical		Non-Linear
		w/o Isolator	w/ Isolator	w/ Isolator	w/o Isolator	w/ Isolator	w/ Isolator
Roof - Edge	Horiz	1.2 g	0.12 g	0.14 g	1.2 g	0.16 g	0.22 g
2nd Floor - Edge	Horiz	1.1 g	0.12 g	0.12 g	1.1 g	0.12 g	0.13 g
1st Floor - Center	Horiz	0.50 g	0.12 g	0.14 g	0.50 g	0.12 g	0.15 g
Roof - Edge	Vert	0	0	0	0.45 g	0.70 g	0.86 g
2nd Floor - Edge	Vert	0	0	0	0.38 g	0.46 g	0.79 g
1st Floor - Center	Vert	0.1 g	0	0	0.45 g	0.70 g	1.25 g

** Transverse Model had greater response accelerations than Longitudinal Model at all locations.

TABLE 4. Member Force Summary

		MEMBER FORCE SUMMARY									
		Static Analysis				Dynamic Analysis***					
		Allow. Stress Code Capacity**		D + L		D + L + S		D + L + E _H *		D+L+E _H * +E _V	
Member I.D.	Member Force	Long.	Trans.	Long.	Trans.	Long.	Trans.	Long.	Trans.	Long.	Trans.
1	P	91.1	91.1	29.7	10.9	77.2	58.3	71.4	47.2	74.0	47.8
2	M	236.8	104.3	14.4	7.4	36.5	34.4	38.4	33.9	52.0	42.5
3	M	224.2	127.1	41.4	9.9	94.2	40.1	126.3	61.3	130.7	64.8
4	M	462.6	406.3	312.9	47.2	322.4	67.9	351.9	81.5	352.0	94.6
5	M	595.0	595.0	0.0	0.0	169.8	166.0	169.4	102.7	176.2	102.7
6	P	N/A	N/A	188.9	255.9	242.2	257.2	212.4	279.6	292.0	470.2
7	V	N/A	N/A	0.0	0.0	20.1	20.1	16.5	16.5	16.5	16.5
8	P	303.3	303.3	91.3	111.5	106.3	112.2	107.2	127.1	235.6	399.4
9	M	35.0	55.0	27.8	13.4	26.5	11.5	42.1	31.0	43.5	47.5
10	M	613.0	613.0	0.0	0.0	125.2	133.2	59.9	137.3	72.7	137.3

** Does not include 1/3 increase.

*** E_H and E_V effects combined by SRSS.

Abbreviations

D = Dead Load
 L = Live Load
 S = Static Seismic Load
 E_H = Horizontal Dynamic Load
 E_V = Vertical Dynamic Load
 * = Increased 12% to account
 for torsion
 P = Axial (kips)
 M = Moment (kip-feet)
 V = Shear (kips)

Member I.D.	Description
1	Brace
2	Roof Beam
3	2nd Floor Beam
4	1st Floor Beam
5	Link Beam
6	Isolator
7	Isolator
8	Column
9	Column
10	Column Extension

FIGURE 1. FCCF Building under construction showing steel framing system (photo taken October 1989).

FIGURE 2. 3D Computer rendering of completed FCCF building.

FIGURE 3. Comparison of 500 year and 1000 year Design Response Spectra with elastic spectra obtained from the Design Ground Motion Time History, SEGM-2.

FIGURE 4. Idealized base isolator force deflection properties.

FIGURE 5. Transverse plane frame dynamic model.

FIGURE 6. Longitudinal plane frame dynamic model.

FIGURE 7. Fundamental horizontal mode of transverse dynamic model (T = 2.2 seconds).

FIGURE 8. Fundamental vertical mode of transverse dynamic model (T = 0.11 seconds).

FIGURE 9. Plot of differential increase in corner displacements and transverse bracing shear force as a function of gross mass eccentricity.

FIGURE 10. Comparison of horizontal 2% damped in-structure response spectra of 2nd floor for base isolated and fixed base transverse dynamic models.

Workshop on Seismic Base Isolation
for Department of Energy Facilities

TESTING OF LAMINATED ELASTOMER BEARINGS

by

Y. W. Chang and R. F. Kulak

Argonne National Laboratory

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Marina Del Rey Marriott
Marina Del Rey, California

Testing of Laminated Elastomer Bearings

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Introduction

Base isolation is a seismic design strategy for reducing the effects of earthquake ground motion on structures by uncoupling the motion of the structure from that of the horizontal components of the earthquake. Base isolated buildings that have experienced earthquakes demonstrated that seismic base isolation can reduce the effects of earthquake ground motion and protect people and equipment from damage. Thus, the use of seismic base isolation is growing very rapidly worldwide as a design alternative for a wide range of critical and important facilities such as hospitals, emergency command centers, computer centers, etc.

Research on base isolation systems began in the early 1970s. In the USA, the National Science Foundation (NSF) has supported fundamental research on base isolation systems for mitigation of the effects on earthquakes on structures and equipment. Following the NSF effort, other U.S. government entities have sponsored base isolation research. This research activity was performed at universities and national laboratories. In Japan, the large construction companies have taken the lead role in developing base isolation technology. Recently, R&D in the U.S.A. has produced an enormous amount of test data on base isolation systems, particularly on laminated elastomer bearings [1-6].

Many types of base isolation systems have been used throughout the world: laminated elastomer bearings, sliding plates, combination of sliding plates and elastomer bearings, friction pendulum sliders, rollers, helical springs, etc. The most commonly used seismic base isolation system appears to be the laminated elastomer bearings. This is mainly due to simplicity of design and reliability of performance under various loading conditions. With developments in elastomer technology and availability of experimental test data, the use of laminated elastomer

bearings for seismic base isolation is rapidly gaining acceptance worldwide to enhance the structural seismic safety margin.

Laminated elastomer bearings have been used in many base isolated buildings. The first building in the USA to use elastomer bearings is the Foothill Community Law and Justice Center in San Bernadino County, which was completed in December 1985. Laminated elastomer bearings have also been used in two French designed nuclear power plants: Cruas, France, and Koeberg, South Africa. The current U.S. advanced liquid metal reactor (ALMR) plant design, the PRISM reactor plant, uses laminated elastomer bearings for base isolation in the reference design.

The laminated elastomer bearing is constructed from alternating layers of thin elastomers and steel plates (shims) that are bonded together during the vulcanization process. The elastomer is usually a carbon filled natural rubber or a blend of natural and synthetic rubbers. There are two types of end connections used to attach bearings to the structure: doweled and bolted. In the dowel-type connection, mounting plates containing dowel pins are bolted to the structure and foundation. The dowel pins fit inside dowel holes located in the end plates of the bearing. This design tends to minimize the occurrence of tension in the elastomer, but allows the bearing to roll out under large displacements. In the bolted-type connection, the mounting plates are bolted to the bearing as well as the structure and the foundation. Tensile stresses can develop in certain regions of the bearing, but bearings cannot roll out. However, under large horizontal displacements, the bearing may exhibit buckling-type instabilities.

Experimental Tests of Elastomer Bearings

This section describes the testing of the laminated elastomer bearings. The objectives of experimental tests are to determine the performance characteristics of the laminated elastomer bearings under a wide range of loadings (performance tests); to determine failure modes and failure loads (safety margin tests); to establish a data base for the development of design formulas (R&D tests); to validate the analytical models and analyses (validation tests); and to assure that

the bearings manufactured by vendors have met the design specifications and have no detrimental flaws in the bearing (acceptance tests). Tests can be performed on scale-size or full-size bearings. In addition, elastomer specimen testing should also be performed to determine the elastomer material properties.

A. Mechanical Properties

Mechanical properties that determine the dynamic characteristics of the bearing response are the shape factor, hysteresis loop, horizontal stiffness, vertical stiffness and damping. They are briefly described below.

1. Shape Factor

The shape factor, S , is defined as the ratio of one loaded area to the total force-free area as shown in Fig. 1.

$$S = \frac{\text{loaded area}}{\text{force free area}}$$

$$= \frac{LB}{2t(L+B)}$$

Figure 1

For a square bearing, S is given by

$$S = \frac{a}{4t} \quad (1)$$

where a is the plan dimension of a square bearing, and t is the thickness of the elastomer layer.

For a circular bearing

$$S = \frac{\phi}{4t} \quad (2)$$

where ϕ is the diameter of the bearing, and t is the thickness of the elastomer layer.

Values for the shape factor can range from 3 to 30. This range can be further divided into three groups:

Low shape factor bearings	$S = 3$ to 6
Medium shape factor bearings	$S = 9$ to 18
High shape factor bearings	$S = 20$ to 30

2. Hysteresis Loop

The hysteresis loop is a curve representing the load-displacement relationship during a complete cycle of the loading and unloading process. It can be expressed in force-displacement or stress-strain form. The work represented by the area between the loading and unloading curves (loop area) is called hysteresis. Figure 2 shows a typical hysteresis loop of an elastomer bearing at a strain level less than 100 percent. For natural rubber containing no fillers, there is very little hysteresis up to moderate strains. At high strains, the hysteresis becomes much larger. For high damping rubber, the hysteresis loops are considerably larger than those for natural rubber.

Figure 2. A Typical Hysteresis Loop of an Elastomer Bearing at a Strain Level Less than 100 Percent

Note: The majority of rubbers used for engineering applications contain a filler. Generally, the filler is a carbon black, which may comprise up to one-third of the total volume of the vulcanizate. These black fillers fall into two groups: (1) reinforcing blacks which improve the tear and abrasion properties of the unfilled gum rubber as well as increase the Young's modulus, hysteresis and creep; and (2) non-reinforcing blacks which have little effect on tear and abrasion and give only moderate increases in modulus, hysteresis and creep.

3. Horizontal Stiffness

Horizontal stiffness is the key mechanical property that determines the horizontal frequency of the base isolated system. The horizontal shear stiffness, K_H , of the bearing is defined as

$$K_H = \frac{F_x}{u_x} = \frac{GA}{t_r} \quad (3)$$

where F_x = shear force
 u_x = shear displacement
 G = shear modulus of elastomer
 A = area of bearing
 t_r = total rubber thickness

The horizontal stiffness of the bearing can be determined from the hysteresis loop obtained from either the bearing tests or elastomer specimen tests. Figure 3 illustrates how the shear modulus is determined from the hysteresis loop. The shear modulus determined is the effective shear modulus, G_{eff} , which is defined to be

$$G_{eff} = \frac{\Delta\tau}{\Delta\gamma} = \frac{\tau_{max}^+ - \tau_{max}^-}{\gamma_{max}^+ - \gamma_{max}^-} \quad (4)$$

where τ_{max}^+ and τ_{max}^- are the maximum positive and negative shear stress, respectively, and γ_{max}^+ and γ_{max}^- are the maximum positive and negative shear strain, respectively. It should be mentioned that there are some variations in the peak amplitude of the hysteresis loops among different hysteric cycles. Figure 4 illustrates the variations in hysteresis loops during the first 40 cycles of the loading and unloading process. The first cycle of hysteresis loop is always larger than the second and subsequent cycles of hysteresis loops. It is the common practice to use the third cycle of the hysteresis loop for the determination of the horizontal stiffness.

Figure 3. Definitions of Shear Moduli. Storage modulus shown as dashed line and effective modulus shown as solid line.

ARGONNE - LTV BEARING TEST
FATIGUE - 150% SHEAR STRAIN - 0.5 HZ

FILENAME: 901108.03
BEARING TYPE: 1
BEARING ID.: ED4866-S/N 07
VERT PRESSURE: 695 PSI
TEST No.: 1

Cycle	Stiffness	Shear Modulus	Damping
1	4.13 Kips/in	101 psi	8.28 %
40	3.24 Kips/in	79 psi	6.31 %

Figure 4. Variations in Hysteresis Loops During the First 40 Cycles of the Loading and Unloading Process

Damping

Damping is another quantity that is used to characterize the elastomer bearings. The effective damping ratio, $\bar{\zeta}$, is defined as

$$\bar{\zeta} = \frac{U_D}{2\pi U_S} \quad (5)$$

where U_S is the energy stored during a loading and unloading cycle and U_D is the energy dissipated during the cycle. Figure 5 illustrates these quantities.

Figure 5. Energy Dissipated/Stored in a Loading-Unloading Cycle

5. Vertical Stiffness

The vertical stiffness, K_v , of the bearing is given by

$$K_v = \frac{F_z}{u_z} = \frac{E_c A}{t_r} \quad (6)$$

where F_z = the axial force
 u_z = the axial deformation
 E_c = compression modulus
 A = cross-sectional area
 t_r = total rubber thickness

The value of the compression modulus, E_c depends on the shape factor S . It can be shown [7] that

$$E_c = E(1 + 2kS^2) \quad (7)$$

where E = Young's modulus
 S = shape factor
 k = a numerical factor. Table 1 lists the values of k as given in Ref. 7.

C. J. Derham [8] performed extensive tests on the compression of bearings. He found that a good approximation for both square and circular bearings was to take

$$E_c = 5.6 GS^2 \quad (8)$$

Table 1. Hardness, Elastic Moduli and k

Durometer	Young's Modulus lb/in ²	Shear Modulus lb/in ²	k	Bulk Modulus E _∞ lb/in ²
35	168	53	0.89	142,000
45	256	76	0.80	142,000
55	460	115	0.64	154,000
65	830	195	0.54	171,000
75	1340	317	0.52	189,000

B. Types of Tests

1. Elastomer Specimen Tests

Testing of elastomeric specimens [9] is performed to determine basic material properties (elastic constants, viscoelastic constants, damage parameters) and performance characteristics (stiffness, damping, creep, temperature effects, aging and environmental effects). These small specimen tests can provide data for strain ranges up to failure and at frequencies relevant to a seismic event. Compression tests are performed on thin disks specimens to determine the bulk modulus. Three bar lap shear specimens are used to determine the shear modulus and the damping ratio for each elastomer. Combined compression-shear tests are performed to determine the elastomer behavior under the complex loading states expected during an earthquake. The above tests determine the performance characteristic of each elastomer and provide the needed material constants for the constitutive models used in the numerical simulations.

The variation of the mechanical properties of the bearing over the life of the plant must also be estimated. This can be accomplished by performing accelerated aging tests in which specimens are exposed to air at elevated temperatures for predetermined time periods. The

retention properties at room temperature can be obtained from the Arrhenius plot. Tensile, shear, and damping properties are measured.

2. Horizontal Stiffness Tests

Since the main purpose of the seismic bearing is to isolate the structure against severe horizontal motion, the horizontal stiffness tests are extremely important. The objectives of these tests are: (1) to determine the horizontal stiffness and damping under a variety of horizontal motions at fixed vertical load, (2) to verify scaling rules between full-scale and reduced-scale models, and (3) to provide data to validate computer codes and analytical models.

2.1 Constant Amplitude

A series of cyclical horizontal stiffness tests with constant amplitude, sinusoidal wave form, and fully reversed horizontal displacements are performed. The vertical load is held constant during each test, but may vary from test to test. The tests are conducted at shear strain levels of ± 10 , ± 25 , ± 50 , ± 100 , ± 150 and ± 200 percent. The vertical load is set at 50, 100, and 150 percent of the design load. The tests are conducted at frequencies of 0.2, 0.5, 0.75 and 2.0 Hz with a sinusoidal wave form. Each test is run for 20 complete cycles. For the purpose of allowing the bearings to recover, a minimum of 15 minutes shall be provided between tests.

2.2 Variable Amplitude

This series of tests is conducted to study the response of the isolators to complex wave forms. The results are compared to similar tests conducted with three-bar lap shear specimens to see if results obtained from specimen tests can predict the behavior of full scale and reduced scale bearings. The vertical load is maintained at the design level during the tests. In the first test, the bearing is subjected to nine groups of five constant-amplitude sine waves. The amplitude and sequence of shear strains (percent) is as follows: ± 30 , ± 60 , ± 90 ,

± 120 , ± 150 , ± 120 , ± 90 , ± 60 and ± 30 . In the second test the bearing is subjected to three groups of five constant-amplitude sine waves with the following shear strain amplitudes (percent): ± 100 , $+50$ to $+100$, ± 100 . For the third test, the testing apparatus is preprogrammed to subject the bearing to selected observed earthquake time histories as well as synthetic earthquake time histories. The results will show bearing performance under complex loadings more typical of what they will experience during an earthquake.

2.3 Sustained Compression

Since it is highly likely that the isolation system will be under the structure for a long period of time before the occurrence of an earthquake, the sustained compression loads may have some effect on the response of the bearings during an earthquake. The following sustained compression test is aimed at simulating this condition. The bearing is subjected to a continuous compressive load, which is equal to the design load, for 72 hours prior to the horizontal cyclical tests. The cyclical test is performed under a constant vertical load equal to the design load. The bearings is then subjected to 20 cycles of fully reversed sinusoidal loading. The amplitude is equal to the design displacement. Comparisons between results from this test and previous tests will show the effects of the pre-test sustained compressive load on the horizontal response characteristics of the bearings.

3. Static Recovery Tests

After an earthquake it will be necessary to have the isolation bearings return the structure to its centered position. The following test is designed to evaluate the self-centering capability of laminated isolation bearings. The bearing is subjected to 20 cycles of 100% shear at a frequency of 0.5 Hz. At the peak amplitude of the last cycle, the horizontal force is removed allowing the bearing to freely return to an equilibrium position. The time history of the horizontal displacement is recorded along with any permanent offset. The displacement history is recorded for eight hours.

4. Vertical Stiffness Tests

During a seismic event, the isolation bearing will undergo relative horizontal displacements that will change the vertical load distribution within the bearing. The result will be a decrease in the vertical stiffness of the bearing and possibly a change in the hysteresis characteristics of the isolator. This test determines the variation in stiffness and damping as a function of horizontal displacement. The tests is conducted at vertical load levels equal to 50, 100 and 150% of the design load. The stiffness and damping ratio at each level is obtained by cycling the vertical load by $\pm 30\%$ for five cycles. The above test sequence is performed at horizontal offsets equivalent to the following shear strains (percent): 0, 25, 50, 100, 150 and 200.

5. Failure Tests

The goals of the failure tests are to obtain failure limits and failure modes so that safety margins can be evaluated. The following test sequence is designed to provide a data base of failure information from the various loading configurations expected during an earthquake.

5.1 Vertical Tension/Compression

This test is designed to determine the breaking strength and failure mechanism of bearings subjected to simple tension/compression. The deformation characteristics near failure are monitored. The tests is performed in a testing machine with a capacity large enough to break the bearings. Scaling laws are verified by testing bearings at different scales (e.g., full size, half size, etc.). This test can provide data to evaluate the safety margins that the bearings have against failure during uplift and compression.

5.2 Combined Tension-Shear and Compression-Shear

A series of tests are carried out to determine the effects of combined tension-shear and combined compression-shear loading on the failure limits and failure mode of the

bearings. Each bearing is subjected to a prescribed horizontal shear strain and then loaded in tension or compression until failure occurs. The resulting data is used to generate a tension shear-compression shear failure curve that can be used by designers to establish safety margins.

6. Fire Tests

Full or scale model bearings are subjected to fire tests [10] to determine their performance during and after a fire event. Mechanical response characteristic tests are performed prior to the fire tests to establish reference values. The bearings are placed in a large multipurpose furnace and loaded to the vertical design load. The bearings are subjected to the standard prescribed time-temperature history for four hours. Temperature histories are recorded with thermocouples placed at predetermined locations within the bearing to monitor the spatial and temporal variation of the temperature field. Afterwards, post-fire mechanical tests are performed on the bearing assemblies to determine the results of thermal degradation. The bearings are sectioned to examine the internal state of the elastomer. Results from these tests can be used to determine the adequacy of these bearings as a structural load bearing component after a fire.

7. Manufacturing Quality Assurance Tests

Upon completion of the manufacturing process of each laminated isolation bearing, it is necessary to test the bearing to assure that it meets the design specifications and does not contain performance effecting flaws.

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**CONSTRUCTION WITH BASE ISOLATION SYSTEMS
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**REMARKS DELIVERED AT THE
WORKSHOP ON SEISMIC BASE ISOLATION
FOR DEPARTMENT OF ENERGY FACILITIES
LOS ANGELES, CALIFORNIA
MAY 15, 1992**

I shall cover two distinct -- very different -- topics today. The first is the construction of new buildings with seismic base isolation. The second is the retrofitting of existing buildings using base isolation.

NEW BUILDINGS:

You have heard a great deal in the past day-and-a-half about the design considerations associated with seismic base isolation. Design is where the action is in new base-isolated buildings. The field construction work not greatly different from conventional construction, but these things should be noted:

1. There will be flexible connections for utilities, and expansion/contraction joints in floors, walls, ceilings, approach walks and driveways spanning the plane of isolation. Special problems in waterproofing and fireproofing can be expected as a result.
2. The contractor should consider the seismic behavior of the partially-built structure, should a quake occur during construction. Behavior will change as dead load increases. Temporary utility connections during construction should be flexible.
3. There will be a new sub-trade to consider, in the budget and in the scheduling of the job: The supplier of the isolators themselves. (These sometimes are pre-purchased by the owner, and hence not in the contractor's budget, but the delivery of the isolators to the site is certainly a critical path item in the contractor's schedule.)

Contracting procedures can be largely "as usual", except that the added complexity will call for these changes:

1. Input from a person or organization familiar with construction, during the design phase, concerning the practicality of construction details.
2. Pre-qualification of a limited number of general contract bidders. They may not have done seismic base isolation work, but each should have a demonstrated record of successful completion of technically complex structures, personnel with this experience available, and the necessary financial strength.

Inexperienced and unqualified contractors will have difficulty estimating this complex work, and early over-optimism may lead to dispute or default. Experienced and qualified contractors are likely to withdraw from a competition that involves a very large number of competitors (more than five or six), or competitors who are clearly unqualified.

RETROFIT OF EXISTING BUILDINGS:

Seismic retrofit, especially if it involves base isolation, is very different from new construction, and can be akin to home remodeling on a very large scale. Because of the complexities and uncertainties of the work, a contractor should be involved in the planning process. It is best to arrange for early selection of the contractor who will do the work. It won't be possible to do it on a price basis, because there won't be a completed design on which to bid.

We do not advise it, but if the project must be bid on a lump sum basis, because of government regulations, then it is especially important to:

- ◆ Prequalify contractors very carefully.
- ◆ Give them bid drawings and specifications which are as complete as possible.
- ◆ Hold pre-bid conferences to iron out uncertainties and answer questions -- with written addenda to clarify bid instructions.
- ◆ Analyze bids carefully, giving weight to exceptions, conditions and ambiguities which may indicate lack of understanding, and to thoroughness which may indicate competence.
- ◆ Have an established system for handling changes and for dispute resolution.

The decision with regard to a general contractor should be more nearly like a decision with regard to design consultant on complex technical work, with heavy emphasis on compe-

tence, experience, and demonstrated ability to work as part of a team. Part of the agreement with the contractor would be some proposed method for determining the cost of the work, once the design and scope of work have been set, and perhaps an agreed method of third party review and mediation.

The work is best performed by construction project managers and superintendents with an understanding of structural design, and with a willingness to take a special level of care in the work. This talk reviews the relationships of the parties, and lists particular items which the contractor must consider as he (or she) prepares for and executes the work.

THE CONTRACTOR'S RELATIONSHIPS:

If there is an experienced contractor at the planning table, then he can review design details as they are developed and propose alternate methods which may be available. Iterations of the budget can provide progressively better estimates of the cost. Possible forms of contract can be discussed at this time. (In our experience the most successful form of contract has been a lump sum agreement, but one arrived at after extensive discussion of methods and budgets.)

During construction there is an especially strong need for quick response and prompt decisions, as the contractor encounters unexpected conditions within an existing building. This need for prompt response applies to the architect and his engineers and to the owner. We ask that a specific person be designated as decision maker (with a back-up person also named) and with one or the other to be reachable at all hours. Home telephone numbers are exchanged. Much of the work in this field is done at night, on weekends and at other unusual times.

If you are subject to the jurisdiction of local building code officials, an atmosphere of trust and open communication is important. The issues involved in seismic retrofit are often new to the building jurisdictions, especially in smaller communities, and much of the work we have done is "beyond the code". Advanced consultation with the permitting and inspection departments of the city is generally helpful.

THE STRUCTURE ITSELF:

The contractor, along with the owner and his design team, starts with the need to know what the structure is now. Are the original drawings and specifications available? If they are, what has been done to change the structure during the intervening years, and is there any record of that?

It is our experience (with public agencies and private owners) that changes during intervening years are generally in the area of electrical, mechanical, H.V.A.C., sprinklers and such work. Very rarely has this work been added to the original drawings or are in any way related to them. Many surprises result from this practice. I suspect that you at D.O.E. are much more careful in recording changes and remodeling work, and that this would be less of a problem in your facilities.

Advanced investigation of the structure in the field is most important, and can reduce the number of changes which will occur during construction. However, it is our experience that no amount of preliminary investigation can eliminate fully the surprises in the field.

What is the structure to become, and what will be its structural condition and stability at intermediate stages? As columns and walls are cut, and isolators added progressively to an existing building, the stiffness of the building gradually changes. Its dynamic characteristics are altered. Therefore, the work must be scheduled with an eye toward maintaining stability and torsional balance, and avoiding overload of non-isolated elements, in case an earthquake occurs during construction. I would recommend that parameters be set by the engineer, but detailed schedule and sequence left to the contractor.

During our seismic base isolation retrofit work at the Rockwell building in Seal Beach, California, the sequence of column cutting and isolator installation was studied carefully. The number and proximity of cuts at any given time were limited. Isolators around the perimeter of the building were "locked" by large bolted steel channels, so that they were immobilized until all isolators were in place. These restrainer channels were then removed quickly in a pre-ordained sequence.

Access which is available, for the structural elements to be modified, often dictates the methods to be used. Elevators, stairs and heavy mechanical equipment often are involved, or in the way. Usually, the building must remain in service, and the design must take into account this fact. If concrete coring, sandblasting, guniting and other messy work is to take place within executive offices, computer rooms and medical laboratories, then the hours and conditions of work must be rigidly controlled. Sometimes the method chosen for doing the work will be governed by these conditions.

At the 9595 Wilshire building in Beverly Hills (not a base-isolated building), we pump-placed the concrete for the shear walls because it was less messy than a shotcreting operation would have been, inside suites of lawyers' offices. All work in these offices was done in the middle of the night. The offices were kept fully operative between 7:00 A.M. and 10:00 P.M., Monday through Saturday. On the other hand, shotcreting was used for shear walls

inside an AT&T office/service building, because workers could be moved out of the immediate area and protective shrouds could be placed around the work for its duration.

OTHER SYSTEMS TO BE MODERNIZED:

It has been our experience that, while work is triggered by seismic concerns, we very often do significant amounts of other work at the same time, or in related phases. This most commonly involves work related to fire and life safety, such as sprinklers, backup power and pumps, and annunciator systems. A close second would be upgrades related to handicapped access. Asbestos frequently is removed or encapsulated by a separate, specially licensed contractor, prior to our arrival on the site.

In a directly related but usually separate operation, seismic anchorage of ceiling systems is often accomplished, and we encourage the owner and his tenants to engage in a systematic effort to anchor bookcases, file cabinets, computers, pumps and generators, elevator equipment and other critical, delicate and/or hazardous equipment.

Finally, after looking at the budget for all of this work, owners shrug their shoulders and decide on an architectural upgrade for front entries, lobbies, corridors, elevators, bathrooms, and exterior landscaping, etc. Related non-seismic work almost always exceeds in cost the seismic strengthening which triggered the project in the first place.

CARE AND HANDLING OF OCCUPANTS:

Human relations are a major concern, because every building we have strengthened has been occupied in some fashion at the time -- some of them 100% full. The contractor and the owner must approach this issue together. The plan of action depends on the nature of the occupants, and the owner's relationship to them.

The easiest situation is in a parking structure, where certain stalls can be vacated and isolated during the work. In a one-tenant building, occupied by the owner's employees, the workers can be instructed to change hours, shift offices or otherwise accommodate the construction work. (I suspect that most of your D.O.E. structures would fall into this category.) At the other end of the spectrum in our experience are commercial office buildings, 100% occupied by high-paying tenants, with seismic work going on at tax time in March and April. (We haven't yet strengthened a hospital.) Among the methods we have used to assist in tenant relations are these:

- ◆ A preparatory meeting (it was once a cocktail party) conducted by the owner, for all the tenants, with the architects, engineers and contractor

on the program. The work was described, the schedule laid out and the contractor's personnel introduced so that they would be known later.

- ◆ A distinctive personal identification lapel badge for each construction worker with his or her name and photo. (This was already required at Rockwell in Seal Beach, and may be standard procedure at your facilities.)
- ◆ Bulletins to tenants and in the elevators during the course of the work, describing progress of the work and updating the schedule. These bulletins can also give times of utility changeovers if required.
- ◆ Prompt and personal response to complaints from tenants, by the contractor's supervisory personnel.
- ◆ In two dormitory projects at UCLA, students resident in the building were involved in a barricade painting contest, creating images and logos related to the seismic upgrade. The winning design was used on a tee-shirt distributed to construction workers and to all occupants in the dorm.
- ◆ Special preparation was made for the changeover of work at the UCLA dormitories from one wing to the other. This involved a mass movement of students between wings over one weekend. A special barbecue was held in conjunction with this event and it was the university's and contractor's intention to make it seem more like a party than a chore.

Nothing can prevent this from being a time of difficulty and stress for occupants of the structure. The emphasis is on good communication of intentions, prompt response to complaints, and the maintenance of good will among the parties.

SCHEDULE:

Given the complexity of the project, as described above, and the close interrelationship among the occupants and the working parties, scheduling is critical. Presidences and interrelationships have to be studied thoroughly. Some contingency has to be built in to accommodate those problems as yet undiscovered. It is our experience that an overall schedule, properly prepared, can be maintained in most cases, but that the week-to-week adjustments are frequent. At weekly jobsite meetings, the best projections are usually those

which extend out three or four weeks into the future, and it is based on these that bulletins have been issued to building occupants.

In empty spaces, or spaces easily vacated, work can be done during normal construction hours, Monday through Friday. However, for the occupancies discussed above which are very sensitive to interruption, work on the evening or graveyard shift is quite common. We worked inside the 9595 Wilshire building in Beverly Hills for approximately eight months on a "Rumpelstiltskin" basis, moving in about 10:30 P.M. every night and covering up our tracks as we vacated each morning about 6:30 A.M.

Two blocks down the street at 9720 Wilshire, most of the work was done between the hours of 3:00 P.M. and 11:00 P.M. It was a compromise reached in a partially-occupied office building immediately adjacent to some very sensitive condominium owners. In Beverly Hills, expensive condominiums, with litigious residents, abut the commercial district. The residents don't want you to work at night, and the office tenants resent your work in the daytime. It calls for diplomacy and planning.

COSTS, BUDGETS, CONTRACT:

As discussed above, it is ordinary on negotiated contracts to progress from preliminary budgets through a series of iterations to a more precise figure. It has been our experience that most owners, and most lenders and financial institutions, are more comfortable with a resulting lump sum agreement than with the cost plus fee agreement which is quite common on new private construction. An experienced and financially strong general contractor can give such a lump sum figure after sufficient study (at least on all the projects with which we have been acquainted to date). It is common for the price to be conditioned, because there are always some unknowns and surprises found as work progresses and as the building's innards are exposed.

As in all construction, changes occur. These can be handled expeditiously and with good feelings on all sides, if they are given prompt attention and if the parties on all sides of the table are skilled and fair. Human relationships among the leaders and among their managing subordinates in the field are critical.

The cost of our seismic retrofit work alone has ranged from \$2.50 per square foot of building (for the addition of shear walls to a parking structure) to approximately \$40 per square foot (for base isolation, a new ductile perimeter frame, and extensive foundation work at a fully-occupied office building). These figures do not include architectural upgrades, fire and life safety, nor other non-seismic construction.

MISCELLANEOUS CONSIDERATIONS:

It is especially important that Builders Risk/Course of Construction Insurance be carried on the structure while seismic rehabilitation work is on its way. In our experience, this is most commonly carried by the owner. Personal injury, property damage, and other liability coverages are essentially standard. Guarantee and warranty periods are the same as for new structures.

In one instance, we as a contractor were asked to assume responsibility for the effective performance of the retrofit design. This crept quietly into the contract form, inserted by the owner's attorneys. We declined to take the job on this basis, because the risk was far too great when compared to our potential gain from performance of the work (particularly so since the type of work being performed was a world first). As the contractor, on seismic rehabilitation work as on all work, we try to limit our liability to the complete and competent performance of the work as called for in the project drawings and specifications. Unless it is a design/build contract (which none of these seismic rehabilitation jobs has been to date) we avoid taking responsibility for the design itself.

CONCLUSION, SUMMARY:

I am an engineer and a contractor, and have spent all of my working life in seismically active areas. The seismic rehabilitation projects on which this paper is based have been among the most challenging of my professional career. Their successful completion has been satisfying, and I shall feel even better about it if the structures perform well when "The Big One" comes. Each project is a puzzle, and almost all of the puzzles are different from each other. As awareness of seismic vulnerability increases across the country, more and more of us will have the opportunity to solve these puzzles, and I am sure that you as professional engineers will enjoy meeting the challenge.

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C. TERRY DOOLEY

Terry Dooley is Senior Vice President of Morley Construction Company, Santa Monica, California. This 45 year old firm operates only in Southern California, but is currently #152 on the ENR list of general contractors in the USA. A significant portion of the firm's business is the building of complex new reinforced concrete structures. During the past few years, it also has done eight seismic strengthening and rehabilitation projects inside existing buildings.

Terry is a native of Hammond, Indiana, and grew up in South Bend. He graduated from St. John's University (Minnesota, BA, English); from the University of Illinois (Urbana, BS, Civil Engineering); and from the University of California (Los Angeles, ME, Engineering Executive Program). Before joining Morley in 1981, he was Contracting Manager for Bethlehem Steel Corporation in Los Angeles.

Terry is a member of the newly-formed ACI Committee 369 on Seismic Rehabilitation, and of three other ACI national committees. He is a Fellow of ASCE, and is active in the Structural Engineers Association of Southern California and in other technical, professional and educational organizations.

In a world-first, seismic isolators are being implanted in the columns of an existing eight-story concrete building.

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SEISMIC SURGERY

By installing seismic isolators in the columns of an eight-story concrete building in Seal Beach, Calif., and by strengthening the existing frame, engineers expect to meet the owner's directive: The building must survive an expected major earthquake with no downtime for the building systems, contents and occupants. (The Newport-Inglewood fault is just 0.7 mi away, and a shock of Richter magnitude 7.0 is considered credible there.) Structural damage must be limited, and minor in nature. The elevators must resume operation promptly.

Among the constraints set forth by the owner, Rockwell International Corp.: Complete the project without disrupting round-the-clock operations by 600 employees; limit utility changeover work during all space shuttle launches and flights; and abide by security access restrictions affecting all subcontractors and suppliers.

Work began in November 1989, and most of the isolators are now in place. Work under the contract will be complete early in 1991.

Rockwell's ownership and sole occupancy did have several advantages, especially during the planning stage when its engineers worked with the design engineers, architects and other consultants. The owner relocated employees around the work area and arranged for storage and assembly areas on the site, but outside the building. The final advantage: in July, Rockwell announced that its corporate headquarters will be moved from a leased building in El Segundo, to this building in 1991.

The eight-story, concrete frame/waffle-slab structure had been built for engineering offices in the mid-1960s. After it had been filled with the sensitive and critical equipment, Rockwell officials realized that any interruption by an earthquake would be unacceptable.

The retrofit includes a ductile re-

inforced concrete frame added to strengthen the building's exterior. This was complicated by the presence of a loading dock and cafeteria extending beyond the exterior column lines, outside the tower footprint. The plane of isolation follows an irregular route, skirting a stairwell and elevator core.

The isolators were installed midway between the first and second floors. The usual position for isolators would be at the foundations, but because of the one-story basement (full of equipment that could not be disturbed) and the geometry of the building, this would be too difficult. The first floor, however, was lightly occupied. The building is the first in the world with isolators in this position; in the U.S., it is the first high-rise seismic retrofit.

Two types of isolators are being installed: single rubber/steel laminated units for the 26 interior columns, and double units with lead cores for the 24 intermediate exterior columns. The four corner col-

Steel yokes take the building load through four 500 ton hydraulic jacks while the column is cut and the isolator is inserted near the first-floor ceiling. Below, after an initial coring, a plunge saw cuts through the 28 in. reinforced concrete column.

Photos by Ronald Moore

umns have single isolators with lead cores. In addition to damping the earthquake energy once the building starts to move, the lead cores prevent vibration from wind. "Without the lead cores, the building would quiver in the breeze. We're using two sizes of lead cores to account for torsional differences in the building," says Dennis J. Drag of Englekirk & Hart Inc. (EHI), prime consultant and structural engineer on the project.

Explaining how the columns are expected to behave in an earthquake, he adds, "The isolator dampens the horizontal forces, so that the column above it will move less than the column below it. We checked the columns as-built and determined that they are sound. The interior columns carry very little, if any, of the seismic load. The exterior columns carry more than 90%, and they are being beefed up."

Calculations showed that the interior columns each could carry from 1,000 to 1,300 kips, and the exterior columns from 1,500 to 2,000 kips. "The owner wished to increase the factor of safety even more to carry these loads, and ordered the strengthening of the exterior frame," says Drag. To resist the added forces, 28 drilled, cast-in-place piles 3 ft in diameter were placed 50 ft into the sandy soil adjacent to the existing foundation. They have been joined to the existing piles at the pile caps.

INTERIOR COLUMNS

Isolating the interior columns took place while the exterior frame was being strengthened. The isolation installation, in addition to being unique, was risky. A building is weaker when it loses part of its structural system, but those columns did have to be cut. Crews from Morley and specialty subcontractors removed obstructing partitions and utilities. Structural steel yokes gripped each column by friction above and below the isolator level, and then four temporary 500 ton hydraulic jacks relieved the load on the column.

Cutting began with a 10 in. diameter core hole at the bottom of the future isolator and continued laterally, using a plunge saw, until the cut was smooth and complete. Control of cooling water was critical during cutting because of busi-

ness operations in the immediate area. A similar cut was made where the top of the isolator would be, and workers removed the concrete "plug" using a specially fitted forklift. (The first plug was displayed in the building's entrance lobby.)

Steel bearing plates were installed at the top and bottom of the cut, and the isolator assembly was moved into place. Steel shims preserved space for a flat jack, while the top and bottom bearing plates were connected to the existing column with nonshrink cement grout. Finally the Freysinner flat jack was moved in as part of the permanent installation.

A critical time at each column was the transfer of load from the temporary hydraulic jack/yoke assembly to the permanent isolator. Pressure in the flat jack rose as high-strength epoxy was injected, and pressure in the peripheral hydraulic jacks dropped simultaneously. Because the precise load in each column is indeterminate, loading of the flat jack ceased when the first of several criteria was reached: (1) load in the temporary jacks dropped to zero; (2) load in the permanent flat jack exceeded significantly the estimated column load; or (3) the floor above began to respond to the pressure. Care and judgment were essential.

When load transfer was judged complete, the flat jack was capped, and the temporary jack/yoke assembly snubbed off and left in place as a backup support until the epoxy in the flat jack had cured completely. The temporary assembly then was moved to another column. (Two nonadjacent interior columns could be under construction at any one time. There were similar exterior restrictions.)

EXTERIOR MOMENT FRAME

At the perimeter of the building the existing C-shaped intermediate columns are 3 ft 6 in. by 7 ft. The remodeled columns will be 5 ft by 7 ft with the C filled in. Bonding the new columns to the old required more than 16,000 dowels (No. 6 rebar 16 in. long), epoxy-grouted into holes drilled 8 in. into the existing concrete, and protruding into the new concrete.

To reduce the noise interference with Rockwell operations, drilling the holes and sandblasting the

FIGURE 1.
SOUTH ELEVATION

Ductile concrete frame added to original structure carries two isolators at each column. North wall is similar to south, but east and west walls have frame to sixth floor at all columns.

bond surface were done in the evening. Because dimensional stability of the rebar was important, the column cages were preassembled on-site from custom-fabricated welded wire fabric.

The new spandrel beams were poured parallel to, but independent of, the existing spandrels. A gap was left between the old beam and the new 18 in. thick member. The strengthened frame had to be complete up to the third floor at any given bay before the column could be cut and new isolators installed at that location.

Different jacking methods were used for the exterior columns. Two temporary wide flange structural steel columns were extended upward from the pile cap next to each column base to support the hydraulic jacks so that column loads could be relieved. The sequence of operations was similar to that used in the interior, but there are two lead-core isolators at each of the intermediate exterior columns. The bearing plates are much heavier. The temporary structural steel columns are cut off after each column is complete, with stubs left in place below grade.

None of the exterior isolators will be operable, however, until all are in place. Temporary restraining plates, $\frac{3}{4}$ in. thick, are bolted to the bearing plates to deactivate each isolator. This prevents individual isolated columns from moving laterally and overloading nonisolated columns, should an earthquake occur during construction.

(The Upland quake occurred on Feb. 28, 1990, with a magnitude of 5.5 and an epicenter some 35 mi from the site. This was early in the construction cycle, before any exterior isolators had been installed. No activities were interrupted. Lights swung but there was no damage, nor did any of the jacks slip.)

When all columns are completed, when work at the elevator/stair core is complete and when all utilities have been made flexible at the plane of isolation, then the restraints will be removed. In an operation akin to "swinging" a bridge that has been on falsework during construction, the building will be put into its new mode as a seismically isolated structure.

THE CORE

Despite the complexity of inserting the isolators into the columns, the elevators presented the biggest challenge on this project. Six traction elevators occupy two rectangular concrete cores, which pass through the isolator plane and continue into the basement. These elevators must be operable following an earthquake, and thus the guide rails must be uninterrupted. The elevator core wall could not be cut for the installation of isolators, because of the expected 14 in. differential movement at the time of the design earthquake.

Taking a different approach, the design engineers decided to sever each elevator core on top of the footing and to insert teflon bear-

FIGURE 2.
SCHEMATIC SECTION

ing plates. Although they provide no damping, these low-friction surfaces will allow the core to shift horizontally along with the upper building. In effect, the elevator cores are hung from the upper structure.

The elevator work was done little by little, like an underpinning job, with enlarged foundations that provide a shelf for the core when it moves. Finally, a new wall 18 in. from the core creates a moat. Flexible expansion joints will cover this moat and a similar space on the first floor between the elevator shaft and the floor. These joints are not designed to absorb energy.

The building's maze of utilities and services ranges from important and sensitive equipment to low-tech but bulky utility lines. These had to be moved away from the isolators and made flexible to handle future differential motions where they pass through the plane of isolation, and had to be rebuilt to meet architectural appearance requirements as well as the building code.

Architectural matters have been a challenge, requiring careful planning of utilities, fireproofing, sprinkler systems and weather protection. The differential movement of floors, walls and ceilings also had to be considered by the owner, designers and contractors meeting frequently as a team.

THE CODE PROBLEM

Seismic isolation is still outside the provisions of the building

codes, which neither require nor prohibit it. A general clause allows any technique that can be proven to be effective. "Rockwell could have rejected the design and so could the city of Seal Beach," explains Englekirk & Hart's Dennis Drag. "We made the application for the permit with help from Dynamic Isolation Systems, the isolator consultant. Because there were so many knowledgeable people involved who could provide the necessary proof, Seal Beach officials approved the concept but retained an outside design firm to do the review for plan check."

Even then there were questions, he recalls. "We spent a great deal of time explaining seismic isolation technology. The closest standard we had was that for California hospitals (CE November 1989, page 66) and the Rockwell project was designed by a factor of two over that standard. In all, the plan check took five or six months plus the time we spent with them prior to submittal.

"Designers can expect to spend more time with isolation projects than with conventional projects in getting permits. Even with the new codes, acceptance will be difficult. The codes will define the criteria and how the work will be done, but designers still will face building departments that are unfamiliar with the technology. Someone has to be able to evaluate a design in order to approve it, and there aren't many people with seismic isolation experience.

CHECKING IT OUT

The Rockwell project, however, did add to the experience of a number of people. Each member of the team was backed up by a designated reviewer. (All firms are in California.) Rockwell chose Englekirk & Hart's isolator proposal in combination with conventional strengthening of the exterior as proposed by Dames & Moore (D&M). The A&E award went to EHI for the design, and D&M was named reviewer. Both the Los Angeles and Irvine offices of EHI were involved, as were the Los Angeles, San Francisco and Seattle offices of D&M.

Rockwell engineers under the direction of Thomas W. Wilson, program manager, and Michael R. Baddley, manager of critical systems and environmental support for the Information System Center, developed design criteria and specifications for the project.

Morley Construction Co., Santa Monica, is general contractor under a lump-sum agreement with the owner. Adams & Smith, Concord, is the key subcontractor for isolator installation. Macco Constructors, Inc., Paramount, did preconstruction cost estimating and reviewed details of the construction proposals.

D&M did foundation studies and developed design criteria, reviewed by Woodward-Clyde Consultants, Santa Ana, and special vibration studies, reviewed by Ewing & Associates, Rancho Palos Verdes. Accu-Test Engineering Laboratories, Canoga Park, performed in-place material strength testing, which was reviewed by EHI/D&M.

The seismic isolation design was by EHI, with assistance from Dynamic Isolation Systems, Berkeley. This work was, in turn, reviewed by D&M. Mechanical engineering by Meckler Engineers Group, Encino, and electrical engineering by Patrick Byrne & Associates, Reseda, were reviewed by EHI/D&M.

Primary field inspection and testing are by Smith-Emery Co., Los Angeles, with Twining Laboratories, Long Beach, handling testing of epoxy-grouted dowels. ◊

C. Terry Dooley, F.ASCE, is senior vice president at Morley Construction Co., Santa Monica, Calif.

THE KEY TO CONSTRUCTION SAFETY AND SYSTEMIC IMPROVEMENT

SEISMIC

TESTING

at the

Energy Technology

Engineering Center

SEISMIC FRAGILITY TESTING

Conservative earthquake design requirements and procedures have been adopted in response to rapidly evolving and increasingly stringent regulatory environment.

Design and construction of seismic features to protect existing and advanced plants have significantly increased costs and licensing delays. Retrofits to older buildings and bridges are also costly and time-consuming.

Current standards are based largely on analytical methods of predicting resistance to earthquake effects. Seismic testing of systems and component hardware to the failure point (called fragility testing) identifies overly conservative design practices and can lead to reductions in construction costs and licensing delays.

Typical fragility test setup.

Hydraulic shaker tables mounted on seismic mass test bed.

Six-inch diameter piping, ready for fragility testing.

Energy Technology Engineering Center

The Energy Technology Engineering Center (ETEC) is a multiprogram field laboratory operated for the U.S. Department of Energy (DOE) by the Rocketdyne Division of Rockwell International. The site is 40 miles northwest of Los Angeles.

ETEC's primary mission is applied engineering development of emerging technologies including solar, conservation, geothermal, fossil, fusion, and fission energy. ETEC performs test facility design, manages construction, and manages and conducts system and component development test programs.

DOE has assigned ETEC the technical lead and designated ETEC as the Center-of-Excellence for component test engineering and seismic survivability testing.

This brochure describes ETEC capabilities for seismic fragility and seismic isolation testing. For questions about using these facilities, or to obtain information on other ETEC programs and testing capabilities, please contact:

ETEC
P.O. Box 1449
Canoga Park, CA 91303
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SEISMIC ISOLATION TESTING

Earthquake effects on critical plant components and systems can be mitigated by isolating buildings and hardware with elastomeric devices, friction devices, or hybrids that incorporate both features.

Testing of candidate devices is needed to determine the effects of traveling seismic waves and vertical forces, resistance to uplift and overturning, and reliability and maintainability.

The ability of various isolation devices to attenuate maximum ground accelerations will be measured. Performance envelopes, defined and failure modes determined.

Seismic Isolator Bearing Test Fixture.

Bearing test fixture with half-scale test article installed.

Bearing test fixture and hydraulic connections.

PERFORMANCE ENVELOPES

Seismic Isolation Testing

8 Vertical Hydraulic Actuators: One Horizontal

Vertical Load (T) 750

Horizontal Load (T) Dynamic 120

Station 150

Horizontal Stroke (in) 15 @ 0.75 Hz

Seismic Fragility Testing

4 Synchronous, Uniaxial Tables

Payload (lbs/table) 750

Velocity (in/sec) 250

Acceleration (G) 30

FOR THE
Committee, M. ASCE

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CONCLUSION

[illegible][illegible]

15. Project Engr., Naval Civ. Engrs. 93043.

Note.—Discussion of this paper must be filed with the paper as a written request. The manuscript for this paper is part of the journal publication. The manuscript on May 19, 1983. ©ASCE, 1984. ISSN 0733-944X. Possible publication on Vol. 110, December, 1984. Paper No. 19377.

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SEISMIC DESIGN AND ANALYSIS

This study focuses on new construction. The structure selected was redesigned, and it was assumed that it was newly constructed and located in a seismically active area. Seismic design concepts were typical of West Coast standard engineering design practice. The structure was designed for six levels of peak ground acceleration—0.1 g to 0.35 g with 0.05 g increments. Elastic design spectra utilizing Newmark standard spectral shapes were implemented. Five concepts of seismic lateral resistance were individually utilized: (1) Steel moment frame; (2) steel braced frame; (3) steel frame and concrete shear wall; (4) concrete moment frame and (5) concrete frame and shear wall. For the purposes of this analysis to facilitate design comparisons, the performance level of the structure under the specified spectra was set at a ductility equal to 1.0 design that members would be at yield. This performance level was set for several reasons. First, a ductility of 1.0 produces the same response as a ductility value greater than 1.0 at higher acceleration. Second, the use of ductility equaling the 1.0 criteria allows the structural design engineer to use all elastic computer codes without conducting a nonlinear analysis; further, nonlinear spectral techniques did not need to be implemented.

The required designs for the five concepts of lateral resistance and six load levels were performed under contract with a firm having significant experience in seismic design. As part of that effort, the contractor was required to provide detailed cost estimates for seismic lateral resistance systems by treating the structure as new construction, using the available cost data on the existing structure, and making adjustments for West Coast construction practice. Results of the designs are presented in Ref. 5.

Cost of Seismic Strengthening.—The detailed structural costs estimated were based on the results of the six design cases for the five concepts of strengthening. The cost of the existing exterior frame construction was deducted from the total building cost, and the cost of each new seismic framing system was then added to obtain a new total building cost. Concrete or masonry seismic shear wall configurations, when utilized, were assumed to replace the existing 6 in. (15 cm) concrete block. The cost of foundation redesign was included, and all costs were adjusted to 1981 Los Angeles levels. Fig. 1(a) shows the seismic strengthening

(a)

FIG. 1.—Seismic Strengthening Concepts and Cost Increases

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for Each Type of Strengthening: (a) Concepts; and (b) Costs

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TABLE 1.—Damage Ratios—Drift Steel Structure

Element (1)	Cost, in dollars (2)	Repair multi- plier (3)	Interstory Drift									
			0.001 (4)	0.005 (5)	0.010 (6)	0.020 (7)	0.030 (8)	0.040 (9)	0.070 (10)	0.100 (11)	0.140 (12)	
a Rigid frames	117,500	2.0	0	0.01	0.02	0.05	0.10	0.25	0.35	0.50	1.00	
b Braced frames with (c Shear walls ; Non-seismic structural frame	625,500 417,600	2.0 2.0	0 0 0	0.03 0.05	0.14 0.30	0.22 0.30	0.40 0.60	0.85 0.85	1.00 1.00	1.0 1.0	1.0 1.0	
d Masonry	417,600	2.0	0	0.10	0.20	0.50	1.00	1.0				
e Windows and frames	120,600	1.5	0	0.30	0.80	1.00						
f Partitions, architectural elements	276,200	1.25	0	0.10	0.30	1.00						
g Floor	301,200	1.5	0	0.01	0.04	0.12	0.20	0.35	0.80	1.00	1.0	
h Foundation	412,100	1.5	0	0.01	0.04	0.10	0.25	0.30	0.50	1.0	1.0	
i Building equipment and plumbing	731,600	1.25	0	0.02	0.07	0.15	0.35	0.45	0.80	1.0	1.0	
j Contents	500,000	1.00	0	0.02	0.07	0.15	0.35	0.45	0.80	1.0	1.0	

FIG. 2.—First Mode Period

ening concepts, and Fig. 1(b) the increase in cost for seismic strengthening. Details of the cost estimating are given in Ref. 4. Fig. 2 gives the first-mode periods of the structure for the strengthening concepts as a function of design level. The moment-frame period shows greatest variation with design acceleration.

Damage Evaluations.—Damage to structural frame members, shear walls and other elements associated with displacement are influenced by interstory drift. Elements tied to the floors, such as equipment or contents, are influenced by floor acceleration. Ref. 2 is a detailed study of work previously done in damage evaluation, as well as an extensive literature review.

The detailed cost estimate was formulated to identify key elements of the structure to which dollar values could be assigned. Repair factors for damage were estimated, with key elements divided into drift- or acceleration-sensitive components, and values of drift and acceleration then related to damage for each element.

Tables 1 and 2 give damage ratios for each key element for the steel framing concepts as a function of drift and acceleration experienced by the structure. A similar table was developed for the concrete-framing concepts. For example, if a masonry element were to experience 0.01 drift, damage could equal 20%. Note that a value is assigned to contents and that the utilization of repair multipliers, to be applied to original costs for the repair of damage from seismic shaking, can result in costs exceeding the total cost of the structure. This is reasonable, since demolition and removal costs would be required for major repairs.

To evaluate the damage expected to occur to the structure, each of the six design loading levels for each of the five lateral resistance design concepts was analyzed for a series of applied seismic load levels. Non

linear finite element techniques were employed. The program DRAIN-TABS was used to conduct the analysis. Fig. 3 gives the damping used for the analysis, based on engineering practice; damping increased with the ratio of applied load to design level. Drift and floor acceleration time-history responses were computed in the analysis. Effective response levels representing sustained loading were selected at 65% of peak values and used in the damage prediction. The rate of 65% has been used in past studies to equate effective peak ground acceleration to sustained levels of loading.

Tables 1 and 2, used in conjunction with the drift and acceleration values computed from the nonlinear analysis, resulted in Tables 3-7, which present damage matrices giving damage to the structure, noted as a function of design level, and applied loading summing up all the elements shown in Tables 1 and 2. Included in the damage matrix is damage to the structure and contents using the noted repair factors. Thus,

TABLE 2.—Damage Ratios—Acceleration Steel Structure

Element (1)	Cost (2)	Repair multi- plier (3)	Floor Acceleration, in g's				
			0.08 (4)	0.18 (5)	0.50 (6)	1.2 (7)	1.4 (8)
1. Floor and roof system	301,200	1.5	0.01	0.02	0.10	0.50	1.0
2. Ceilings and lights	288,500	1.25	0.01	0.10	0.60	0.95	1.0
3. Building equipment and plumbing	731,600	1.25	0.01	0.10	0.45	0.60	1.0
4. Elevators	57,000	1.5	0.01	0.10	0.50	0.70	1.0
5. Foundations (slab on grade, sitework)	412,100	1.5	0.01	0.02	0.10	0.50	1.0
6. Contents	500,000	1.05	0.05	0.20	0.60	0.90	1.0

FIG. 3.—Damping

Tables 3-7 show structural, nonstructural, mechanical and electrical damages, as well as damage to contents, and the cost of restoring the building to operation.

Steel Moment Frame.—The response of the moment frame structure is in the constant velocity region of the spectra for all six design ranges. Note that as the structure is stiffened, displacement decreases while acceleration increases. Damage is contingent upon both displacement and acceleration. Note also that for a given applied load level, each of the six design cases is at a different damping level, with the weakest structure being most heavily damped. In the low applied loading level, the strong structures are lightly damped, responding elastically with floor accelerations higher than those of weaker designs at the same load level. The weaker structures are more heavily damped, responding inelastically with lower floor accelerations than stiffer and stronger structures.

TABLE 4.—Damage Ratio—Steel Braced Frame

Applied load, in g's (1)	Design Acceleration, in g's							
	0.00 (2)	0.10 (3)	0.15 (4)	0.20 (5)	0.25 (6)	0.30 (7)	0.35 (8)	
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
0.10	0.10	0.08	0.08	0.07	0.07	0.07	0.06	
0.20	0.17	0.14	0.14	0.14	0.12	0.12	0.11	
0.30	0.24	0.23	0.21	0.20	0.17	0.16	0.16	
0.40	0.37	0.28	0.28	0.27	0.24	0.23	0.23	
0.50	0.56	0.32	0.31	0.30	0.27	0.26	0.16	
0.60	0.82	0.34	0.34	0.32	0.29	0.29	0.28	
0.70	1.03	0.37	0.37	0.35	0.31	0.34	0.35	
0.80	1.13	0.42	0.41	0.41	0.36	0.37	0.39	
0.90	1.25	0.45	0.43	0.43	0.47	0.45	0.52	
1.00	1.50	0.49	0.48	0.47	0.56	0.61	0.71	

TABLE 5.—Damage Ratio—Steel Frame and Concrete Shear Wall

TABLE 3.—Damage Index		Design Acceleration, in g's							
Applied load, in g's (1)									
	0.00 (2)	0.10 (3)	0.15 (4)	0.20 (5)	0.25 (6)	0.30 (7)	0.35 (8)		
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0.10	0.10	0.04	0.04	0.04	0.04	0.02	0.02		
0.20	0.17	0.08	0.07	0.07	0.07	0.07	0.07		
0.30	0.24	0.11	0.11	0.20	0.09	0.07	0.07		
0.40	0.37	0.15	0.14	0.13	0.12	0.11	0.11		
0.50	0.56	0.20	0.18	0.16	0.14	0.11	0.11		
0.60	0.82	0.24	0.23	0.18	0.17	0.14	0.14		
0.70	1.03	0.26	0.26	0.20	0.17	0.17	0.17		
0.80	1.13	0.29	0.28	0.24	0.21	0.12	0.20		
0.90	1.25	1.20	0.31	0.25	0.23	0.20	0.19		
1.00	1.50	1.50	0.32	0.28	0.25	0.23	0.22		

TABLE 6.—Damage Ratio—Concrete Moment Frame

TABLE 6. — Drainage Ratio — Concrete Moment Frame								
Applied load, in g's (1)	Design Acceleration, in g's							
	0.00 (2)	0.10 (3)	0.15 (4)	0.20 (5)	0.25 (6)	0.30 (7)	0.35 (8)	
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
0.10	0.05	0.01	0.01	0.03	0.04	0.05	0.06	
0.20	0.06	0.05	0.04	0.04	0.06	0.07	0.08	
0.30	0.18	0.11	0.11	0.10	0.10	0.10	0.09	
0.40	0.22	0.14	0.14	0.12	0.12	0.12	0.12	
0.50	0.25	0.18	0.18	0.16	0.15	0.14	0.14	
0.60	0.35	0.24	0.23	0.19	0.18	0.17	0.17	
0.70	0.46	0.28	0.26	0.25	0.21	0.21	0.20	
0.80	0.60	0.33	0.30	0.26	0.24	0.23	0.23	
0.90	0.82	0.39	0.36	0.30	0.27	0.27	0.26	
1.00	1.05	0.44	0.42	0.34	0.30	0.29	0.28	

TABLE 3.—Damage Ratio—Steel Moment Frame

Applied load, In g's (1)	Design Acceleration, In g's						
	0.00 (2)	0.10 (3)	0.15 (4)	0.20 (5)	0.25 (6)	0.30 (7)	0.35 (8)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.10	0.10	0.08	0.08	0.09	0.11	0.11	0.11
0.20	0.17	0.11	0.13	0.15	0.17	0.19	0.21
0.30	0.24	0.16	0.20	0.23	0.22	0.24	0.24
0.40	0.37	0.24	0.26	0.27	0.27	0.26	0.29
0.50	0.56	0.37	0.29	0.32	0.29	0.29	0.31
0.60	0.82	0.55	0.35	0.36	0.33	0.33	0.32
0.70	1.03	0.68	0.40	0.39	0.35	0.34	0.35
0.80	1.13	0.75	0.43	0.43	0.38	0.38	0.38
0.90	1.25	0.83	0.46	0.45	0.42	0.40	0.39
1.00	1.50	0.89	0.50	0.48	0.46	0.42	0.41

TABLE 7.—Damage Ratio—Concrete Frame and Shear Wall

Applied load, in g's (1)	Design Acceleration, in g's							
	0.00 (2)	0.10 (3)	0.15 (4)	0.20 (5)	0.25 (6)	0.30 (7)	0.35 (8)	
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
0.10	0.05	0.01	0.04	0.03	0.03	0.03	0.03	
0.20	0.06	0.04	0.06	0.05	0.10	0.10	0.09	
0.30	0.18	0.07	0.08	0.10	0.10	0.09	0.09	
0.40	0.22	0.11	0.13	0.14	0.15	0.14	0.13	
0.50	0.25	0.15	0.15	0.17	0.16	0.15	0.15	
0.60	0.35	0.21	0.20	0.18	0.20	0.19	0.19	
0.70	0.46	0.26	0.25	0.22	0.20	0.23	0.23	
0.80	0.60	0.31	0.29	0.27	0.24	0.23	0.28	
0.90	0.82	0.35	0.34	0.31	0.28	0.27	0.24	
1.00	1.05	0.40	0.38	0.35	0.32	0.30	0.29	

at the same load level. In this range, stiffer structures receive greater damage, a condition which exists to about 0.5 g for the range of structures studied. Over 0.5 g the stiffer structures experience lower levels of damage, as might be expected. The use of a single time-history event with its unique frequency content results in minor response variations. Any single time history has unique frequency gaps and high points. Since the period of the structure changes with strengthening (stiffening), secondary interactions occur between the frequency high points and structure periods so that responses at a particular design level may be slightly reduced or amplified over the response of an ideal time history without gaps and high points. Further, the six design cases are not exact multiples, but depend on the human selection of available structural shapes. These factors induce very minor dispersion in the results. A clear conclusion, however, is that stiffening in the low applied acceleration region does not reduce total damage. Fig. 4 shows a plot of damage ratio as a function of applied load to design level. The data illustrates the effects of variation, in period of the structure, on the response. The damage ratio is a complex function of period, damping, range of nonlinear behavior, and the mix of total damage caused by drift and acceleration.

Steel Braced Frame.—The response of the braced-frame structure is in the constant acceleration region of the spectra for all six design ranges. The structure in its basic configuration with bracing is much stiffer than the corresponding moment frame, pushing the response from the constant velocity region to the acceleration region. The resulting floor accelerations, produced by the applied loading, are higher than those of the moment frame, while story drifts are reduced. In the medium- and low-level applied loading range, damage decreases with stiffening; however, at high load levels the acceleration dominates, resulting in higher damage with stiffening. Again, note that damping varies with the ratio of applied to design load level. Note also that three of the designs utilized two-bay bracing, and three of the designs utilized three-bay bracing.

Steel Shear Wall.—The shear wall/frame structure was the stiffest of

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FIG. 4.—Damage Ratio as Function of Design and Load, Moment Frame

the three steel concepts studied. Damage was generally least with this structure; however, collapse did occur for the 0.1 g design at the 0.9 g applied load. The brittle nature and sudden shear failure are shown by the 0.29 damage ratio at 0.8 g loading, and the collapse at 0.9 g loading (Table 5). In general, because of the low period of the structure, floor acceleration resulting from amplification of base motion was least, and in high applied acceleration load levels, attenuation occurred.

Concrete Moment Frame.—The response of this structure is similar to the steel moment frame. In the low applied load range, stiffer structures receive greater damage; this condition exists to about 0.25 g for the range of structures studied. Over 0.25 g, the stiffer structures exhibit lower damage, as might be expected.

Concrete Shear Wall.—The shear wall/frame structure was stiffer than the concrete moment frame. Less damage was incurred, generally, with this structure. In general, because of the low period of the structure, floor acceleration resulted from the amplification of base motion. In high applied acceleration load levels, attenuation occurred.

Site Seismic Probability.—The Navy developed an automated procedure to conduct a seismic analysis using available historic and geologic data. The objective of a seismicity study is to determine the probability of acceleration at the site. To do this, site coordinates and study bounds are specified in terms of latitude and longitude. A regional study is performed first, in which all the historic epicenters are used with an attenuation relationship to compute site acceleration for all historic earth-

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quakes. A regression analysis is conducted to obtain regional recurrence coefficients, and a map of epicenters is plotted. The regional recurrence can be used to compute the probability of site acceleration for randomly located events in the study area when individual faults are not well enough known to be specified.

When individual fault zones can be specified, individual subsets of historic data for a specific fault are used, in conjunction with geological data for that fault, to determine fault recurrence coefficients in a manner similar to that used for the region. These are used to compute the probability of site acceleration from individual fault sources. The total risk is then determined for all faults specified. Confidence bounds are given on the site acceleration as a function of probability of not being exceeded. The results of five case studies were used for this work.

NAVY ECONOMIC ANALYSIS

Ref. 1 specifies procedures for the economic analysis of facilities. This analysis must take into consideration the fact that earthquake strengthening is expressed as a current cost increased to protect against future dollar losses. The real world is complicated by cost increases through inflation; to repair or replace a damaged building will cost more in the future than today. Work in the previous sections expresses the costs of seismic design levels and damage as a percentage of (new) building value in order to maintain a common reference. That premise recognized increased value of the building and increased costs of repair as damage occurs. In an economic sense, this may be expressed as letting the discount rate (the value of return on investment) equal the inflation rate.

The government has placed a value on money in time. NAVFAC P44. (1) and DODINST 7041.3 set the discount rate at 10%. This rate is differential in addition to inflation.

When the present worth of the annual expected damage is considered using a discount rate of 10%, the present worth estimate of damage would be effectively reduced by a factor of about five. In other words, the earthquake could occur at any point during the life of the structure; the best estimate considers an equivalent series of annual expected losses. The assumed life of the structure is 50 years, based on NAVFAC seismic design criteria. The present worth of this accumulated loss series can be computed, and its value is about one-fifth the total expected loss.

Note that the discount rate specified for use is actually a differential rate of 10% over the rate of inflation. One could use the differential rate without taking inflation into account, or one could consider the rate of inflation when projecting increased repair costs, and then discount that cost using a rate of 10% plus inflation. The results for modest inflation rates are approximately the same. The differential cost approach was used in this study.

Included in the economic analysis is a value for injury and loss of life. There may be considerable difficulty in assigning value to a life; however, an economic analysis must represent all elements of the problem in common terms in order to ensure completeness (see, e.g., Ref. 3). The value assigned to the loss of life is \$300,000 in the analysis. As mentioned in later sections, results were not sensitive to the value selected

for moderate damage levels, neither loss of life nor injury was found to be high, based on a review of previous earthquakes with similar construction.

SITE STUDIES

Five sites—Bremerton, Wash.; Memphis, Tenn.; San Diego, Port Hueme, and Long Beach, Calif.—were examined in light of the cost and damage data presented earlier, and in light of the probability of site acceleration distributions. Fig. 5 gives typical results of the increased cost of seismic design levels on new construction for the steel moment frame structure. The moment frame is the most expensive lateral resistance system. Costs for the braced frame and shear wall systems do not vary significantly with design level.

Fig. 6 indicates the least-cost design acceleration in terms of the 225-yr return-time acceleration (80% probability of not being exceeded in 50 yr), based on the probability distribution data from the five sites.

ANALYSIS

Seismic strengthening costs are assumed to be dependent on the type of lateral resistance system utilized; damage is correlated both to drift and acceleration. Stiffening alone limits drift damage but increases acceleration damage. Damage to a structure is complex, and is influenced by damping level, degree of inelastic behavior, acceleration and drift levels, and the spectral region of response. The most cost-effective design

FIG. 5.—Increase in Cost in Various Locations for Moment Frame Structure

FIG. 6.—Least Cost Design Acceleration for Important Structures (Ductility = 1)

acceleration is a function of construction type and site seismic exposure. Since acceleration produces a significant amount of damage, special care should be taken to design ceilings and lights that will withstand it. Acceleration causes equipment to overturn—a factor accounting for mechanical and electrical losses. Since stiffening produces increased utilization of isolation techniques, should be given to the development and utilization of isolation techniques. A value of design acceleration with a 60 to 100-yr return time appears reasonable. A 100-yr return-time acceleration would represent a design level of about 70% of the 225-yr return time level, and a 60-yr return-time acceleration would be about 50% of the 225-yr return-time level.

Fig. 7 shows a histogram of the probability distribution of acceleration normalized on the basis that the 225-yr return-time acceleration is 0.25 g. This data is based on a composite of a number of sites studied. Also shown in the figure is the damage ratio for a steel-moment-frame structure. An examination of the computed results of the probabilistic expected damage analysis over the life of the structure (probability times damage in increments) shows that most damage comes from the ex-

FIG. 7.—Histogram of Probability Distribution of Acceleration and Distribution of Damage Ratio with Acceleration

posure to low-level acceleration. Structures which respond elastically in this range, having been designed for high acceleration, exhibit high floor accelerations causing much of the damage. As noted, strengthening produces little or no reduction in damage at low acceleration levels, which are most probable since floor acceleration increase with the resulting stiffening of the structure.

CONCLUSION

This paper presented data on the cost of seismic design levels for new construction, and examined techniques for estimating damage. An analysis of a three-story structure with five seismic concepts of lateral resistance was conducted and evaluated under a range of loadings. Results show a design acceleration with a return time of 60- to 100-yr to be cost-effective. These results are limited to the type of low-rise (three-story) structure studied and are shown to be a function of the type of seismic strengthening concept used.

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PROBABILISTIC ANALYSIS OF TIMBER BRIDGE DECKS

By Andrzej S. Nowak,¹ A. M. ASCE and Medhat K. Boutros²

Abstract: Reliability indices are calculated for single timber beams and timber bridges with deck planks and stringers. Load and resistance models are developed on the basis of the truck survey and material test data available. Three common groups of species and two material grades are considered. The reliability indices are calculated using the Rackwitz and Fiessler procedure for the single beams, and Monte Carlo simulations for the whole bridges. The reliability indices for single beams are about 15–40% lower than those for whole bridges. The difference can be attributed to a distribution of load between the stringers because of the mechanical properties of wood, in particular the modulus of elasticity, modulus of rupture and their correlation.

INTRODUCTION

Wood is used more frequently for short-span bridge decks. Its low cost and availability, especially in the northern U.S. and Canada, make timber an attractive material for future construction (5). In the present AASHTO specifications, the design (12) and evaluation (6) of timber bridges are based on allowable stresses. However, this approach does not allow for the determination of the actual safety reserve in the structure.

Wood has high variability in its properties, such as modulus of rupture (MOR), modulus of elasticity (MOE), dimensions, etc. Therefore, the AASHTO allowable stresses for stringers are specified at about 20–25% of the mean MOR. But a bridge is a structural system of interconnected members. Due to load sharing and a partial correlation between MOR and MOE, the reliability of the bridge is usually higher than that of its components (stringers).

This paper develops a method for the evaluation of the reliability of timber bridge deck systems. Its approach follows the direction indicated by Bakht (2). Load and resistance are modeled from the available load survey and material test data. The components' mechanical properties are generated using the Monte Carlo method. In each run, the safety margin is evaluated for the entire structure, using a specially developed computer program, based on a finite strip model. Safety is measured in terms of a reliability index (4). For comparison, reliability is also evaluated for individual bridge components.

The approach is applied to timber deck systems with transverse deck planks supported by longitudinal stringers. The typical spans for such structures are from 10–24 ft (3–7 m), with the maximum span of 30 ft (10 m). The stringers, usually from 6 × 12 in. (150 × 300 mm) to 8 × 24 in. (200 × 600 mm), are spaced at 15–24 in. (370–600 mm). The deck

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AN OVERVIEW OF THE NAVY PROGRAM IN EARTHQUAKE ENGINEERING

J. M. Ferritto^I

ABSTRACT

This paper will discuss the risks faced by the U.S. Navy to earthquakes and the mitigation measures underway. An extensive program of research and investigation has been conducted over the last decade. Over 14,000 buildings have been evaluated, finding about 750 structures requiring upgrading. In conjunction with this investigation, numerous procedures have been developed and guidance documents produced. The paper will discuss the various elements of the Navy program.

INTRODUCTION

The Navy has 55 major bases in seismically active regions whose facilities are worth \$25 billion. This includes utilities, roads, bridges, dams, and over 14,000 major buildings.

There presently exists a concern to the Navy if major fleet support facilities are damaged. The threat is especially great for West Coast activities where a single earthquake could cause damage to several bases. Many of the buildings at these installations were built without adherence to current seismic standards. The Navy, because of its waterfront mission, must site its facilities on low-lying coastal areas where fine-grained soil conditions are usually marginal and extremely susceptible to large deformation under earthquake loads. High seismicity combined with marginal bearing and saturated soil conditions in areas susceptible to flooding make them especially vulnerable to major damage.

Recent earthquakes, particularly those in Alaska, Japan, and Chile have emphasized the high damage threat the soil liquefaction phenomenon poses to waterfront structures. In the 1960 Chilean earthquake (magnitude 7) quaywalls, sheet piles, and sea walls were damaged by liquefaction of loose, fine, sandy soils. In the 1964 Alaskan earthquake (magnitude 8.4) severe damage to Anchorage, Cordova, and Valdez occurred, including large-scale land slides, as a result of liquefaction. Japanese earthquakes (Niigata, 1964, magnitude 7.3; Miyagi-Ken-Oki, 1978, magnitude 7.4) caused severe waterfront damage to wharfs, bulkheads, quaywalls, piers, and conventional structures. The majority of the damage sustained in waterfront areas was primarily from liquefaction of loose, cohesionless sands [13].

Each year there is a 5 percent risk of a major earthquake in the Southern California area. Risk to other sites on the West Coast may be only slightly

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less. Risk also exists to Eastern sites such as Charleston. Each year the expected loss from seismic damage could approach \$0.5 to 1.0 billion. The conclusions are:

- a. The risk exposure to an earthquake affecting Navy installations is very high.
- b. Given that the earthquake occurs, waterfront facilities are especially vulnerable.

For the above reasons, the Navy has been conducting a research program to develop the technologies necessary to understand the response of structures at the waterfront to the ground motions induced by earthquakes. The large number of unique structures not found in the civilian community makes the task challenging.

THE EARTHQUAKE HAZARDS REDUCTION ACT

The Earthquake Hazards Reduction Act of 1977, and subsequent acts, intended to reduce the risks to life and property from earthquakes, directs Federal agencies to develop technical standards for critical facilities and to implement earthquake safety practices. The Naval Facilities Engineering Command (NAVFAC) tasked the Office of Naval Research (ONR) to form a panel of recognized experts to advise the Navy on techniques for providing protection against earthquake hazards. The panel stated a need for a planned program for upgrading seismic resistance of selected facilities located in areas of high seismic activity to reduce the threat of loss of operational readiness.

This early study set the direction for future Navy studies. NAVFAC responded to this study by conducting an extensive engineering investigation program of its facilities and by sponsoring a research program at the Naval Civil Engineering Laboratory (NCEL). Additionally, the Department of Defense (DOD) established a requirement, stated in DOD Construction Manual 4270.1, which requires a systematic program for investigating and upgrading existing structures for earthquake safety. NAVFAC provided three directives. The first advisory, in nature, established expedient measures to improve earthquake safety. The second mandated that modernization and repair projects include investigation of earthquake safety and remedial measures when the project cost exceeds 10 percent of the replacement cost of the facility. The third directive established a program to investigate existing structures in seismic zones 3 and 4.

ENGINEERING INVESTIGATION PROGRAM

Several alternatives were examined for determining how to evaluate the safety of the 14,000 major buildings located in seismically active areas. Both meeting mission requirements and protecting life were equally important. The results of pilot studies were presented to the staff of the Chief of Naval Operations; NAVFAC was authorized to submit a budget item to conduct the earthquake safety investigations.

The procedures for conducting an earthquake safety investigation of a base included [12]:

1. Screening a computerized inventory of base facilities and consulting with base personnel to establish a list of base facilities to be reviewed.
2. A site seismicity study to establish probability of earthquake ground motion and response spectra.
3. An evaluation of site hazards.
4. A field screening and data collection for structures and utilities.
5. A preliminary rapid structural analysis.
6. An evaluation of preliminary findings.
7. A follow-up investigation of site hazards and utilities.
8. A detailed analysis of structures indicated to have excessive ductility demands from preliminary analysis.
9. Recommendations for mitigation measures as required.
10. A final evaluation report including cost estimates and programming budget data.

In the process of conducting the investigations, Navy seismic research developed a number of tools for conducting site seismicity studies, performing rapid analysis, etc., which are presented in the following sections. The Tri-Service manual "Seismic Design of Buildings," NAVFAC P-355 [11] was updated. An additional volume, "Design Manual for Essential Structures," NAVFAC P355.1 [10] was developed to insure mission essential facilities would contain increased seismic resistance for acceptable performance.

At this time, essentially all of the bases and all of the Navy structures intended to be screened have been at a cost of about \$14 million. Site hazards, particularly liquefaction have been identified at 10 bases. About 5 percent of the existing structures, about 750, require structural upgrading. Over 130 structures have been upgraded as part of planned modernization. Sixty additional buildings have been upgraded as a result of the seismic investigation. Two hundred projects are pending. Some minor improvements have been made to utility systems. It is estimated that total construction costs of structure upgrades, excluding hospitals, could reach \$300 million. The cost of the preliminary analysis has been about \$2 thousand per building; the cost of the detailed evaluation may vary widely, but on average has been about \$25 thousand per building [12].

Through the process of screening the Navy has been able to evaluate the potential for damage. The Naval Facilities Engineering Command is not in a better position to prioritize the construction spending to eliminate the greatest risks. The following sections will present some of the technology developed as part of the process to evaluate and control expected damage from earthquakes.

SITE SEISMICITY STUDIES

NAVFAC guidelines specify that buildings designated as "critical buildings," shall have a site seismicity study performed in lieu of the lateral load coefficients provided in NAVFAC P355, Seismic Design of Buildings. Acceleration response spectra developed from the study shall be used to determine the magnitude of the lateral loads. Furthermore, the level of design earthquake is suggested to be based on a probabilistic assessment of activity.

The objective of a seismicity study is to quantify the level and characteristics of the earthquake ground motion which pose a risk to the site of interest. The approach taken in this work was to use the historical epicenter data base in conjunction with geologic data where available to best estimate the probability of site acceleration levels. This was the basis for definition of response spectra suitable for use in structural design and analysis.

An automated procedure has been developed by NCEL [2] to perform a site seismicity analysis using historic and geologic data. The specific region of the study is specified in terms of bounding latitude and longitude. The general data file of earthquakes is sorted and the subset of earthquakes in the region becomes the data base for the specific study. The location of the site is specified. The program determines the distance between the site and each epicenter in the study data base and, using the earthquake average magnitude and the attenuation relationship, computes the site acceleration for all historic earthquakes; a histogram is created. For a regional analysis, the earthquake magnitudes are then plotted, and a least-squares analysis is used to obtain recurrence coefficients; a map of epicenters is plotted.

Further study can be made by specifying the location of a fault in terms of coordinates of several points defining line segments. All earthquakes within a specified distance or boundary from the fault line under study are made a subset, and the recurrence of the fault is calculated. A probability analysis is then performed to calculate expected site acceleration and causative earthquake magnitude and epicentral distance. The program randomly selects the epicenter of an earthquake. Using the fault recurrence data in terms of Richter coefficients and maximum earthquake magnitude associated with the fault, the program determines an earthquake magnitude and the length of fault break (assumed centered on the epicenter), and then calculates the closest distance of the site to the fault break (the epicentral distance and the hypocentral distance). These distances, along with the acceleration magnitude attenuation relationship, give the site acceleration. The process is repeated to produce a list of site accelerations and related causative magnitude and epicentral distance thus defining the site's probability distribution. Random location, magnitude, and acceleration level are all considered in the determination of site acceleration. Provisions are included to use recurrence data from slip analysis or regional seismicity in lieu of fault specific data. Further provision is included for floating earthquakes to study a region when specific faults are not known. The program determines, tabulates, and plots the probability of not exceeding various levels of acceleration in the time period specified. These data are available for the regions as a whole, for individual faults, and for all faults acting together. The determination of total risk to the site is of importance for establishing design levels.

RAPID ANALYSIS

A "rapid analysis" procedure was developed by the firm of John Blume/URS, under Navy contract, to assess vulnerability. In this method the key structural elements are identified and their capacities to resist earthquake forces estimated. A resistance curve for the structure can then be compared to the earthquake demand. Demand is determined from site response spectra (for various degrees of damping) developed from the seismicity study.

The calculations can be done fairly simply and the results quickly obtained by a graph showing the demand and resistance line as shown in Figure 1. The ordinate is the spectral acceleration and the abscissa is the period. When the initial yield is reached, damage is assumed to be zero. When the ultimate capacity is reached (collapse mechanism developed) damage is assumed to be 100 percent. The period of the structure will lengthen and its ductility changes as it experiences inelastic distortion [12].

This evaluation method assumes that damage varies linearly between yield and ultimate. A point is found on the line connecting P_y and P_u such that the horizontal distance from P_y is to the horizontal distance to P_u inversely as the horizontal distances from the point to intersections with the demand spectra for elastic damping and damping at ultimate. "Percent of damage" is defined as the horizontal distance from yield to the point divided by the distance from yield to ultimate.

NCEL has developed computer programs to apply the method. It requires an engineer experienced in evaluation of structures for seismic loading. The method cannot be applied to some complex structures which should be considered for further investigation on the basis of judgment. The damage percentage is a fictitious number, and the method tends to overestimate damage for steel and timber structures. It is used to identify structures which may be very vulnerable and warrant further investigation. It also provides an indication of the relative vulnerability of the various structures. It has been satisfactory for these purposes.

ECONOMICS OF SEISMIC DESIGN

The establishing of criteria for seismic design levels for new and existing construction is a complex task involving areas of dynamic nonlinear structural response and damage prediction as well as economics of costs of strengthening and expected damage over the life of the facility. The Navy, as shown above, has numerous facilities requiring upgrade and limited resources. An area of interest was the relationship of the cost of seismic strengthening in new construction or the cost of the level of upgrade in existing construction to damage which is expected to occur. The objective was to produce a facility which will function within prescribed structural limits over its lifetime to insure the mission tasks performed in the building can be accomplished safely without unacceptable interruption at the lowest cost [1, 4, and 9].

Detailed investigation of the cost of seismic strengthening for various lateral restraint systems was investigated. Damage to structural and non-structural components was developed as functions of drift and story acceleration. Costs of seismic strengthening and damage expected from an earthquake

were combined with the probability distribution of site ground motion in an economic analysis.

Stiffening by itself limits drift damage but increases acceleration damage. Damage to a structure is complex, and is influenced by damping level, degree of inelastic behavior, acceleration and drift levels, and the spectral region of response. The most cost-effective design acceleration is a function of construction type and site seismic exposure. Since acceleration produces a significant amount of damage, special care should be taken to design ceilings and lights that will withstand it. Acceleration causes equipment to overturn, a factor accounting for most mechanical and electrical losses. Since stiffening produces increased acceleration, consideration should be given to the development and utilization of isolation techniques. A value of design acceleration with a 60- to 100-year return time appears reasonable. A 100-year return-time acceleration would represent a design level of about 70 percent of the 225-year return time level, and a 60-year return-time acceleration would be about 50 percent of the 225-year return-time level.

An examination of the computed results of the probabilistic expected damage analysis over the life of the structure (probability times damage in increments) shows that most damage comes from the exposure to low-level acceleration. Structures which respond elastically in this range, having been designed for high acceleration, exhibit high floor accelerations causing much of the damage. As noted, strengthening produces little or no reduction in damage at low acceleration levels, which are most probable since floor acceleration increase with the resulting stiffening of the structure.

An analysis of a three-story structure with five seismic concepts of lateral resistance was conducted and evaluated under a range of loadings. Results show a design acceleration with a return time of 60- to 100-year to be cost-effective. These results are limited to the type of low-rise (three-story) structure studied and are shown to be a function of the type of seismic strengthening concept used. The results support continued use of seismic design levels as defined in NAVFAC P355.

LIQUEFACTION

Present engineering practice does not consider the presence of the structure in determining the potential for liquefaction. Further, once liquefaction is evaluated, the resulting deformation state is not determined. Clearly this is inadequate to determine the likelihood of liquefaction and the consequences for a major structure. The finite element technique offers the potential for detailed analysis of soil structure problems and has been used extensively. Development of a constitutive model to predict pore pressure and characterize soils in terms of effective stresses (stress on soil grains causing deformation) rather than simple total stress offers the potential capability to analyze complex structures where liquefaction is possible. The finite element method has in recent years been a useful tool in analyzing structures, structures on soil, and soil-structures. An emphasis has been placed on use of nonlinear plasticity soil models to more accurately capture the soil response. Navy research work has focused on effective stress analysis, the ability to not only calculate the soil stress but also to calculate the pore fluid pressure.

NCEL undertook a review of available material models. Comparison was made between test data and model predictions [3]. Based on this study, it was concluded that the material model under development at Princeton University by Professor J.H. Prevost was able to predict the behavior of cohesive and cohesionless materials, and that its development was well advanced into the implementation stage in a finite element code [6].

The Princeton University Soil Model represents soil as a two-component system: the soil skeleton and the pore fluid. The hysteretic stress-strain behavior of the soil skeleton is modeled by using the effective-stress elastic-plastic model reported in Reference 7. The model is an extension of the simple multi-surface J_2 -plasticity theory and uses conical yield surfaces. The model has been developed to retain the extreme versatility and accuracy of the multi-surface J_2 -theory in describing observed shear nonlinear hysteretic behavior, shear stress-induced anisotropy effects, and to reflect the strong dependency of the shear dilatancy on the effective stress ratio in granular cohesionless soils. The model is applicable to general three-dimensional stress-strain conditions, but its parameters can be derived entirely from the results of conventional triaxial soil tests.

Soil test data required to develop model material parameters were obtained from a series of drained and undrained triaxial compression and extension tests conducted on a number of sands by universities and private geotechnical firms under NCEL sponsorship [5 and 8]. Cyclic undrained tests and proportional loading tests were also conducted. With detailed soil test data available, NCEL began the task of validating the soil model by using the undrained monotonic and cyclic behavior with actual test data. In general the material model was capable of giving excellent representation of drained test data under a variety of loading conditions using parameters based on drained triaxial compression and extension. The model is capable of giving good agreement with undrained monotonic test data and can track the occurrence of liquefaction in cyclic tests in approximately the same number of cycles. Centrifuge model studies were conducted and analyzed as a further validation of the effective stress soils models accuracy when applied to boundary value problems.

Work is underway validating the Princeton University Effective Stress Soil Model. When the effective stress soil model is validated, it will be an important tool in the analysis of waterfront construction. To obtain basic soil behavior, the Navy is participating with the Natural Science Foundation in sponsoring a test program to measure in-situ pore water pressures during an earthquake. A site in Taiwan has been instrumented with piezometers and pore water pressures measured during two significant earthquakes.

BASE ISOLATION OF STRUCTURES

Base isolation of the columns of structures using rubber bearing systems was found to offer potential for reduction of damage to essential structures, particularly where the contents are costly and need to survive. Various concepts for base isolation are being reviewed. A detailed study is underway on evaluating isolator parameters and their effects in analysis of the response of the structure.

SUMMARY

For more than a decade, the Navy has had an active program in evaluation and mitigation of seismic risk to its facilities. This program has involved research to develop procedures for analysis, engineering investigations to evaluate the existing vulnerability and mitigation to upgrade the inventory of facilities. Over 14,000 buildings have been screened and 750 have been identified as requiring upgrade. Two hundred buildings have been upgraded and an additional 200 projects are being programmed for.

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SECTION 13080

SEISMIC BASE ISOLATION SYSTEM REQUIREMENTS

1.0 SCOPE

The Los Angeles County Fire Department Fire Command and Control Facility (FCCF) will be an indispensable link in Los Angeles County lifelines, dispatching and directing county-wide fire and medical emergency relief services whenever needed. This facility function must be maintained during and following major earthquakes and subsequent aftershocks. In order to provide the highest level of seismic protection to the FCCF function, the building will be supported on a seismic base isolation system.

The isolation system requirements contained in this specification are based upon the application of the most stringent requirements taken from the Tentative Seismic Isolation Design Requirements of the Structural Engineers Association of Northern California (reference 2.1), the Los Angeles County Building Code seismic provisions (reference 2.2), and project-specific criteria. The latter encompasses a contingency-level event representing a 1000 year return period earthquake for which the building and equipment will be designed for elastic response. This criteria exceeds code requirements and governs the design.

Any proposed base isolation system shall meet the requirements listed in this specification. All engineering documentation and calculations shall be provided by a California registered structural engineer.

The seismic base isolation system will consist of 36 (32 support bearings plus 4 spare) laminated steel plate and natural rubber bearing assemblies, their attachment to the building and foundation, and an ultimate restraint system, all of which has been designed, manufactured and tested to meet the following requirements. Sliding type isolation systems shall not be used.

2.0 REFERENCES

All work included in this specification shall conform to the applicable provisions of the following codes and standards:

- 2.1 "Tentative Seismic Isolation Design Requirements," Base Isolation Subcommittee of the Seismology Committee, Structural Engineers Association of Northern California, September 1986.
- 2.2 Building Code, 1987 Edition, County of Los Angeles, Department of Public Works, Building and Safety Division, Los Angeles County Code, Title 26.

- 2.3 "Recommended Lateral Force Requirements and Commentary," Seismology Committee, Structural Engineers Association of California, 1980 Edition.
- 2.4 "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings," American Institute of Steel Construction, Eighth Edition.
- 2.5 ASTM A36, Specification for Structural Steel (top and bottom cover plates).
- 2.6 ASTM A570, Specification for Hot-Rolled Carbon Steel Sheet and Strip, Structural Quality (shim plates).
- 2.7 ASTM D395, Test Methods for Rubber Property - Compression Set.
- 2.8 ASTM D412, Test Methods for Rubber Properties in Tension.
- 2.9 ASTM D624, Test Methods for Rubber Property - Tear Resistance.
- 2.10 ASTM D429, Test Methods for Rubber Property - Adhesion to Rigid Substrates.
- 2.11 ASTM D573, Test Methods for Rubber - Deterioration in an Air Oven.
- 2.12 ASTM D1149, Test Methods for Rubber Deterioration - Surface Ozone Cracking in a Chamber (Flat Specimens).
- 2.13 ASTM D2137-83, Test Methods for Rubber and Rubber-Coated Fabrics - Brittleness Temperature by Impact.
- 2.14 ASTM D2240, Test Methods for Rubber Property - Durometer Hardness.
- 2.15 ASTM D4014, Annex A1, Standard Specification for Plain and Steel-Laminated Elastomeric Bearings for Bridges (A1. Determination of Shear Modulus).
- 2.16 ASTM E4, Load Verification of Testing Machines.

3.0 REQUIREMENTS

3.1 Supported Building and Foundation

Isolator bearings will support a two story, steel braced frame building at each of its 32 columns. The structure is doubly symmetrical in plan, measuring 3 bays by 7 bays (see attached plans and elevations). The mass eccentricity is zero for the building, and the fixed base period of the building is approximately 0.2 seconds.

The isolator units will be supported on spread footings founded on bedrock. The site period, T_s , is 0.5 seconds.

3.2 Vertical Loads

<u>Building Location</u>	<u>No. of Bearings</u>	<u>DL (Kips)</u>	<u>LL (Kips)</u>	<u>E (Earthquake) (Kips)</u>
Corner	4	60	53	±55
Exterior	16	105	105	±55
Interior	12	190	196	0
Spare	4	= 2 Interior & 2 Exterior		
Total	36			

3.3 Vertical Deflection

The seismic isolation system shall have a calculated vertical natural frequency no less than 10 Hz.

The vertical long term (25 years) creep shall not exceed the initial elastic deflection under the design load (DL).

3.4 Displacement

The design displacement, D (displacement across the isolator system at the building c.g.), and the maximum total displacement, D_T (includes allowance for bidirectional horizontal ground motions and real and accidental torsion), is given in Table 1.0 corresponding to the equivalent viscous modal damping of the isolation system.

TABLE 1.0 DESIGN AND TOTAL DISPLACEMENTS

<u>Modal Damping (% of critical)</u>	<u>D (in)</u>	<u>D_T (in)</u>
10	7.8	9.6
15	7.0	8.6
20	6.3	7.8
25	5.9	7.3

All isolator bearings shall be capable of accommodating a displacement of $1.5 D_T$ under full design vertical load (DL+LL+E) without loss of stability.

3.5 Isolation Period

The isolated structure period (reference 2.1 definition) shall be in the range of 1.5 to 2.5 seconds at the design displacement and under the total building weight of 4230 kips.

3.6 Isolation System Damping

The equivalent viscous modal damping of the isolation system shall be at least 10 percent of critical but not greater than 25 percent of critical at the design displacement.

3.7 Base Shear

The maximum base shear transmitted through the isolation system to the building (assuming a planar analysis) shall not exceed $0.15W$, where W is the total building weight equal to 4230 kips. The building will be designed to respond elastically at this force level. The actual base shear and actual displacement (at the building c.g.) shall be calculated (and reported for the NS and EW directions) using modal response spectrum analysis methods (or equivalent) and incorporating the design response spectrum given in Table 3.0 (see Footnote 1.). For purposes of this simplified analysis, the planar analytical model of the building shown in Figure 1.0 shall be used.

The maximum horizontal seismic reactions (NS and EW) for each bearing type shall be calculated and reported using the computed base shear results from above and assuming a rigid first floor diaphragm.

3.8 Wind Drift

The isolation system shall exhibit a lateral drift of no more than 0.4 inches under a design wind load of $0.033W$ ($W=4230$ kips). Fusible links to resist wind loads shall not be used.

1. The design response spectrum given in Table 3.0 is for five percent of critical damping. For modal damping other than five percent of critical, adjust the spectrum in accordance with the deamplification factors given in Reference 2.1, Table 1-K.

3.9 Ultimate Restraint System

The seismic isolation system shall be supplied with an ultimate restraint system which will arrest building displacements and thereby provide additional base shear resistance. The ultimate restraint system shall begin engagement at a displacement of $1.4 D_T$ and the ultimate restraint system shall be designed so as to provide an ultimate total resistance of 125 percent of the computed actual base shear (calculated in Section 3.7). This resistance is in addition to that provided by the isolator bearings in shear at a displacement of $1.4 D_T$. Thus, the seismic isolation system shall have a total ultimate base shear capacity of at least 225 percent of the computed actual base shear at a displacement of 15.5 in. or less.

The ultimate restraint system shall exhibit a smooth increase in stiffness as the restraints are engaged. The transition of system resistance over the range from zero deflection to a force resistance of at least 200 percent of the base shear calculated in Section 3.7 shall have been demonstrated by sinusoidal loading test data for a similar design. In addition, the higher mode response of the ultimate restraint system shall have been demonstrated by shake table test of a similar system. The absence of large collision-induced accelerations shall have been demonstrated by such tests.

Any supplier of a base isolation system proposed for use in the Fire Command and Control Facility shall submit, as a part of their response to this RFQ, 1) the sinusoidal loading test and shake table test results identified in the foregoing paragraph, and 2) the proposed design details of the ultimate restraint system (and its connection to the foundation and building frame), including an explanation of its operation and how it relates to the test data supplied in item 1).

3.10 Self-Restoring Capability

The isolation system shall be capable of restoring the plan location of the building, measured relative to the foundation, to within 33 percent of the design displacement, D , from its original position following a design earthquake. The recentering capability of the bearings shall be determined in accordance with Reference 2.1, Section E.1.4.

3.11 Safety Factors

Each isolator bearing shall be capable of supporting the combination of dead, live and earthquake induced vertical reactions at various levels of lateral bearing displacement and

with minimum factors of safety against failure (overstress or buckling) as shown in the following table. See Section 3.2 for dead, live and earthquake loads applicable to each bearing location.

TABLE 2.0 VERTICAL LOAD CAPACITY

<u>Vertical Load</u>	<u>Bearing</u>	
	<u>Lateral Displacement</u>	<u>Safety Factor</u>
D+L	Zero	3.0
D+L+E	D_T	2.0
D+L+E	$1.5 D_T$	1.0
D+0.2L+E	Note No. 1	1.0

Note No. 1: Displacement of 15.5 in. or less corresponding to full engagement of the ultimate restraint system at its maximum design reaction.

3.12 Bearing Height

All isolator bearings shall have the same height.

3.13 Bearing Attachment

Each bearing will be supported on a one inch thick steel plate anchored and embedded in top of the foundation. Each building column will be supported on a one inch thick column base plate which in turn will be supported by an isolator bearing. These embed and column base plates will be supplied by others. Any supplier of a base isolation system proposed for use in the FCCF shall submit, as a part of their response to this RFQ, the proposed connection details (for joining the bearing to the bottom embed plate and top column base plate). The connection details shall allow for shimming of the columns and shall provide for easy removal and replacement of isolator bearings.

3.14 Maintenance Program

Any supplier of a base isolation system proposed for use in the Fire Command and Control Facility (FCCF) shall submit, as a part of their response to this RFQ, a proposed maintenance program report. The maintenance program report shall define the following:

3.14.1 Necessary regular inspection of isolation system and ultimate restraint system.

3.14.2 Installed isolator testing schedule.

3.14.3 Conditions which may be hazardous to isolation bearings.

3.14.4 Recommended replacement time interval.

3.14.5 Procedure for replacement of bearings.

3.15 Steel Plates

Shim plates and top and bottom cover plates shall be made from rolled carbon steel conforming to ASTM A570 or A36 or equivalent. All plates shall be sandblasted and cleaned of all surface rust and scale before bonding. Plates shall be uniform thickness, flat, true and free from blemishes. Mating surfaces between cover plates and mating anchorage plates shall be machined and coated to resist corrosion.

All plate edges to be bonded to the elastomer shall be rounded by grinding to remove sharpness. All corners in plan shall have a minimum radius of 0.20 in.

3.16 Fabrication

Bearings must be cast as a unit in a mold and bonded and vulcanized under heat and pressure.

3.17 Precedence

Where inconsistencies occur between this specification and the requirements set forth in the references cited in Section 2.0, REFERENCES, this specification shall govern.

4.0 QUALITY ASSURANCE PROVISIONS

4.1 Dynamic Shake Table Test

Any supplier of a base isolation system proposed for use in the Fire Command and Control Facility shall submit, as a part of their response to this RFQ, documentation of at least one dynamic shake table test of the isolation system (including an ultimate restraint system of similar design) using a scaled structural model. The test model need not represent the Fire Command and Control Facility. The scale of the model shall have linear dimensions which are not less than one-fifth of life-size dimensions.

4.2 Validation Testing

Prior to fabrication of the production isolation bearings, the following validation tests shall be conducted to confirm the design properties to be used in the design and analysis of the seismic isolation system and the building. Note, these tests are

not required as a part of the submittal to this RFQ, but shall be conducted by the base isolation system supplier after contract award.

4.2.1 Sequence of Tests

The following sequence of tests shall be performed on at least two full size assemblies of each type and size of isolation bearing used in the system. The test data shall include the ultimate restraint system where noted. Specimens tested shall not be used for construction.

For each test the cyclic force-deflection and hysteretic behavior of the test specimen shall be recorded.

- a. Twenty fully reversed cycles of loading at a force corresponding to the design wind load of 3.3 percent of the building dead load.
- b. Three fully reversed cycles of loading at the following increments of the maximum design displacement, D_T : 0.25, 0.50, 0.75, 1.0 and 1.25.

If variations in effective stiffness greater than ± 15 percent occur in these three cycles of loading at a given displacement, then three additional cycles of loading at the given amplitude shall be performed.

- c. Ten fully reversed cycles of loading at the maximum design displacement, D_T , and under a vertical load of DL (see Section 3.2). The ten cycle test shall be repeated for two other vertical loads as follows:
 1. $1.2 \text{ DL} + 0.5 \text{ E}$
 2. $0.8 \text{ DL} - 0.5 \text{ E}$
- d. Three fully reversed cycles of loading at the displacement which mobilizes the 225 percent of the computed actual base shear, including the 125 percent base shear contribution of the ultimate restraint system. This test shall be made for a vertical load of $\text{DL} + 2\text{E}$.

4.2.2 Determination of Dynamic Properties

The following deformation and damping characteristics shall be calculated (and reported) for each of the foregoing tests.

a. Stiffness

The effective stiffness of the system at each displacement shall be calculated for each cycle of loading in accordance with Section D.3 of Reference 2.1.

- b. The equivalent viscous damping ratio of each cycle of loading shall be calculated in accordance with Section D.4 of Reference 2.1.

4.2.3 System Adequacy

The seismic base isolation system test performance shall be assessed as adequate if:

- a. The test force deflection plots for all tests specified in Section 4.2.1 have a positive incremental stiffness.
- b. There is less than a 20 percent deterioration in the equivalent stiffness for the six cycles of test (if six are performed as specified in Section 4.2.1.b).
- c. There is less than a 25 percent deterioration in the equivalent stiffness and damping values for the ten cycles of a test at a given vertical load specified in Section 4.2.1.c.

4.2.4 Testing Requirements

The validation testing specified in the foregoing shall be carried out by a testing facility approved by the Engineer. All records of such tests shall be submitted to the Engineer for his approval. The cost of all testing shall be paid for by the bearing supplier.

4.3 Quality Control and Material Tests

4.3.1 Batch Requirements

The elastomer shall be natural rubber compounded with other ingredients, specified by the bearing designer. The bearing manufacturer may have to make some adjustments in the compound in order to meet the target shear modulus specified by the bearing designer.

Each batch of material shall have its components weighted to an accuracy of + or - 0.5% and identified by code number. Blended batches are not permitted.

4.3.2 Compound Tests

Each batch of elastomer compound shall conform to the following:

- a. Tensile strength of 2500 psi minimum in accordance with ASTM D412.
- b. Elongation at break of 500 percent minimum in accordance with ASTM D412.
- c. Compression set of 25 percent maximum after 22 hours at 158 degrees F in accordance with ASTM D395 (Method B).
- d. Tear strength of 180 pounds per inch minimum in accordance with ASTM D624 (die C).
- e. Hardness (Type A Durometer) of (value and + or - range to be specified by bearing designer) in accordance with ASTM D2240.
- f. Ozone resistance of no cracks for 20 percent strain, 100 hours exposure at 104 + or - 3.6 degrees F in accordance with ASTM D1149 (Specimen A), except 50 + or - 10 parts per 100,000,000.
- g. Low temperature brittleness of no failures at -40 degrees F in accordance with ASTM D2137 (Specimen A).
- h. After accelerated aging in accordance with ASTM D573 for 70 hours at 212 degrees F, the elastomer shall not show deterioration changes in excess of the following limits:
 1. Tensile strength -25 percent
 2. Elongation at break -40 percent
 3. Hardness +10 points
- i. After the accelerated aging (specified in Section 4.3.2.h) the bond strength (using the shear test sample) shall be 40 lb/in. minimum in accordance with ASTM 429 (Method B).
- j. The shear modulus of the elastomer at 50 percent shear strain shall be determined by the use of a double shear tuning fork test (in accordance with ASTM 4014, Annex A1). The virgin elastomer shall be subjected to three cycles of strain to ± 50 percent shear at a frequency of 0.5 Hz. The cord modulus

from the third cycle shall be recorded. It shall be within ± 10 percent of the target shear modulus (as initially specified by the bearing designer) or a revised value as determined by the bearing designer. The area of the third hysteresis loop shall be measured. Three samples from each batch of elastomer shall be tested and the average of these three tests recorded and supplied to the bearing designer. The initial target shear modulus is (to be supplied by bearing designer) psi at 50 percent shear strain.

- k. The bearing supplier may employ additional quality control tests, e.g., vulcanization characteristics, in accordance with ASTM D2084, on each batch of elastomer compound in order to establish base line values and/or limits on prototype production.

4.3.3 Completed Bearing Tests

a. Tolerance on Dimensions

The tolerance on finished bearing dimensions shall be as follows:

<u>Dimension</u>	<u>Tolerance</u>
Thickness of individual rubber layers	$\pm 20\%$ of design value, but not more than ± 0.12 in.
Top and bottom rubber cover layer thickness (if used)	± 0.04 in.
External plan dimensions	± 0.20 in.
Shim plan dimensions	± 0.12 in.
Variation from plane parallel to the theoretical surface	
Top	Slope relative to the bottom of no more than 0.005 radians
Sides	± 0.20 in.
Overall bearing height	$+0.25$ in., -0.125 in.

b. Sustained Compression

Each bearing shall be loaded in compression to 1.5 times its rated load (DL+LL) for a period of 15 hours. During this test, the compression load shall be maintained within a tolerance of ± 10 percent throughout the 15 hour period.

The bearing shall be inspected for faults during this test as follows:

1. If lack of rubber-to-steel bond is detected, the bearing shall be rejected.
2. If laminate placement faults are present which result in layer thickness that exceeds the tolerances given in Section 4.3.3.a, the bearing shall be rejected.
3. If there are any surface cracks which are greater than 0.08 in. wide or 0.08 in. deep, the bearing shall be rejected.

c. Compression Stiffness

Each bearing shall be loaded to 1.5 times its rated load (DL+LL) at zero shear strain. The applied load shall then be reduced to 0.3 times the rated load.

Each bearing shall again be loaded to 1.5 times its rated load and the applied loading then reduced to 0.3 times the rated load and maintained at this value for one minute when deflection indicator gauges shall be set to zero. The bearing shall then be loaded at a uniform rate over a minimum period of one minute to 1.1 times the rated load and maintained at this value for one minute. Gauge readings of compressive deflection shall then be taken to evaluate the compressive stiffness of the bearing.

1. If lack of rubber-to-steel bond is indicated, the bearing shall be rejected.
2. If laminate placement faults are present which result in layer thickness that exceeds the tolerances in Section 4.3.3.a, the bearing shall be rejected.
3. If there are any surface cracks which are greater than 0.08 in. wide or 0.08 in. deep, the bearing shall be rejected.

4. If the compressive stiffness so determined for a particular bearing varies by more than $\pm 10\%$ from the median value of all bearings (of the same design), the bearing shall be rejected.

d. Combined Compression and Shear Tests

After completion of the test for compression, 20% of each bearing type shall be tested by pairs in combined shear and compression in a test rig that permits a shear deflection to be applied to the two bearings, each pair shall be loaded in compression to the maximum applied dead load plus live load. The compression load shall be maintained while the bearings are deflected in shear at a uniform rate of approximately one inch per minute. The bearings shall be subjected to five complete reversed cycles of loading from 0 to $+D_T$ to 0 to $-D_T$. A continuous plot of the shear load and shear deflection shall be recorded to evaluate the combined shear stiffness of the two bearings.

The force in each bearing $F_{\max}$ at displacement D_T shall be within $\pm 10\%$ of the mean value for all bearings of the same type.

4.3.4 Testing Requirements

The quality control and material tests specified in the foregoing shall be carried out by a testing facility approved by the Engineer. The cost of all testing shall be paid by the bearing supplier.

5.0 MARKING AND CERTIFICATION

Each bearing shall be marked on all four sides with a code for order number, batch number, bearing number, location and type. The location numbers shall correspond to the column identification numbers shown on the foundation location plan, sheet S-2.

The marking shall be made by a permanent method. The proposed method for marking the bearings shall be submitted as a part of the response to this RFQ.

The manufacturer shall certify that each bearing satisfies the performance specification and shall provide a certified copy of all test results to the Engineer. The bearing test certificates shall identify the bearings by their marking.

6.0 TECHNICAL INFORMATION REQUIRED

Proposals submitted in response to this RFQ shall include the following technical information:

- 6.1 Ultimate restraint system data including sinusoidal loading and shake table test results, and design information (Section 3.9).
- 6.2 Bearing attachment connection details (Section 3.13).
- 6.3 Maintenance program report (Section 3.14).
- 6.4 Dynamic shake table test documentation (Section 4.1).
- 6.5 Proposed method for permanent marking of bearings (Section 5.0).

END OF SECTION

Figure 1.0 ANALYTICAL MODEL OF FCCF BUILDING ON FIXED BASE

TABLE 3.0

DESIGN RESPONSE SPECTRUM (5% DAMPING)

S P E C T R A L O R D I N A T E S

<u>No.</u>	<u>T-Sec</u>	<u>Sa-g</u>	<u>Sv-cm/sec</u>	<u>Sv-in/sec</u>	<u>Sd-cm</u>	<u>Sd-inch</u>
1	0.030	0.4200	1.97	0.77	0.009	0.004
2	0.050	0.5397	4.22	1.66	0.034	0.013
3	0.075	0.7140	8.36	3.29	0.100	0.039
4	0.100	0.8526	13.32	5.24	0.212	0.083
5	0.150	1.0395	24.36	9.59	0.581	0.229
6	0.200	1.1088	34.64	13.64	1.103	0.434
7	0.250	1.1067	43.22	17.01	1.720	0.677
8	0.300	1.0689	50.09	19.72	2.392	0.942
9	0.350	1.0080	55.11	21.70	3.070	1.209
10	0.400	0.9156	57.21	22.52	3.642	1.434
11	0.500	0.7665	59.87	23.57	4.764	1.876
12	0.600	0.6489	60.82	23.94	5.808	2.286
13	0.700	0.5544	60.62	23.87	6.754	2.659
14	0.800	0.4788	59.83	23.56	7.618	2.999
15	0.900	0.4200	59.04	23.25	8.458	3.330
16	1.000	0.3738	58.39	22.99	9.293	3.659
17	1.500	0.2310	54.12	21.31	12.921	5.087
18	2.000	0.1630	50.92	20.05	16.207	6.381
19	2.500	0.1227	47.92	18.87	19.069	7.507
20	3.000	0.0968	45.35	17.85	21.651	8.524
21	3.500	0.0791	43.24	17.02	24.085	9.482
22	4.000	0.0662	41.36	16.28	26.329	10.366
23	4.500	0.0566	39.75	15.65	28.470	11.209
24	5.000	0.0489	38.22	15.05	30.411	11.973
25	6.000	0.0383	35.92	14.14	34.300	13.504
26	8.000	0.0260	32.44	12.77	41.298	16.259
27	10.000	0.0191	29.78	11.73	47.404	18.663

Spectral Ordinates averaged for the Following 2 Site Conditions:

ROCK Sites; and STIFF SOIL sites.

Zero-Period Acceleration = 0.420 g

Spectral Damping = 5.0%

LEGEND:

T = Period in seconds.

Sa-g = Pseudo-Absolute Spectral Acceleration in g's.

Sv- = Pseudo-Relative Spectral Velocity in cm/sec and in inches/sec.

Sd- = Relative Spectral Displacement in cm and in inches.

No-downtime seismic retrofit

Contractors install seismic isolators on columns, strengthen frame, and isolate elevator core while building stays in round-the-clock operation

By D. L. Frohmader
Fort Collins, Colorado

When Rockwell International decided to seismically retrofit its eight-story Seal Beach, California, information systems center, it planned to ensure the uninterrupted operation of the center as well as the safety of its employees. To that end, Rockwell and the structural engineering firm, Englekirk and Hart, decided to reinforce the structure of the building and simultaneously mount it on a base isolation system. To those at Rockwell, a company accustomed to innovations and precise engineering, the job looked straightforward. To the contractors, it conjured scenes from their worst nightmares.

The job was loaded with unusual specifications and performance requirements. And while these difficulties were being tackled the building had to remain fully functional. Only six 24-hour holidays could halt Rockwell's operations in the building, allowing critical utilities to be revamped. Many proposed construction techniques also were untried and, worse, at the time of the bidding process some were not yet developed. When Rockwell required a lump-sum bid, the job looked ominous.

Morley Construction, a California-based contractor for 43 years, was awarded the job and, despite the inherent difficulties, is successfully completing the work with innovative construction techniques. The project is scheduled for completion in the first quarter of 1991.

The isolators

Base isolating an existing building is, in the most general sense, like cutting it from its foundation

and remounting it on a series of shock absorbers. Because they are flexible in the horizontal direction, isolators prevent most horizontal movement from being transmitted to the building during an earthquake. This reduces floor accelera-

The seismic retrofit for this eight-story building included strengthening the structural frame. Old C-shaped columns seen above the scaffolding were filled in and enlarged to a rectangular cross-section.

Morley Construction Company

Figure 1. Isolator units that are inserted in columns contain alternating layers of steel plates and rubber pads. They are stiff in the vertical direction but flexible in the horizontal direction. The flat jack preloads the isolator after it's installed.

tions and relative movements between floors.

The isolator units were designed by the isolation consultant to suit the building and predicted possible earthquake motions with a maximum lateral displacement of 14 inches. The units are essentially alternating layers of steel plates and rubber pads bonded together (Figure 1). They displace laterally like a deck of cards—each layer slips to the side a bit further than the last when pushed to the side. The 26 interior columns use one isolator per column as do the four exterior corners. The remaining 24 exterior columns use two isolators each. Lead-cored units are being used in the exterior columns to limit movement caused by winds or minor tremors.

Installing the interior isolators

Early on, the owner and design engineers decided to install the isolators between the first and second floor rather than below the first floor as normally done. The

equipment housed in the basement level is essential to continued operations in the building and could not be removed during construction. The first floor is relatively free of vital equipment and provides easy access to most of the columns where the isolators will be located.

Installing an isolator requires cutting out a section of column so the isolator can be inserted. Only four nonadjacent columns (two interior and two exterior) can be cut at any time without dangerously weakening the building. The load on the columns (an average 950 kips on the interior columns and 1,200 kips on the exterior columns in this case) has to be supported as the installation proceeds.

The isolator installation subcontractor designed a mechanical yoke to support the loads (Figure 2). The yoke grips the column above and below the area of the cut and four 500-ton hydraulic jacks are connected between the yoke sections. Jacks are preloaded

Figure 2. A mechanical yoke carries column loads while the isolator is installed. Four 500-ton hydraulic jacks transferred column load to the yoke system.

to calculated levels, taking the entire load off the area between.

To cut a section out of a column for the isolator, a 10-inch hole is bored horizontally through the reinforced concrete. An unusual Swedish plunge saw is then passed through the hole with successively larger-diameter horizontal blades until the column is completely cut.

The lower cut is made first to eliminate binding the saw when the section is cut free. The cuts are sized to allow enough room for assembly and loading of the isolator while preventing the upper and lower isolator plates from moving thereafter. This accurate fit and the weight of the structure is what keeps the isolator in place as no mechanical attachment is made.

Once the column section is removed and the area cleaned, the upper and lower isolator bearing plates are slipped over the column ends and the isolator is inserted between them. The isolator and plates are held in position by a se-

ries of perimeter bolts and the plates are grouted to the columns. The locator bolts are long enough so the isolator can still move vertically, allowing a large, hollow, steel pancake called a flat jack to be inserted between the upper plate and the isolator.

Once the locator bolts are in place, the flat jack is pressurized with liquid epoxy and expands to support the full column load. When the epoxy sets, the jacks are released at the yokes and the yokes removed.

To avoid rotation damage if a quake should occur before all the isolators are in place, locking plates are secured to the isolators on exterior columns. These plates will be removed only when all the isolators are in place.

Exterior isolators add more problems

Although conceptually the same as the interior units, the exterior isolators presented additional challenges because the exterior columns are being strengthened. The old columns had a C-shaped cross-section. New heavily reinforced concrete is being used to fill in and expand the column cross-section (see photo on page 843). Additional caissons (36 inches in diameter and 50 feet deep) were required to carry the load of the new, heavier, exterior frames.

Drilling piles. Drilling most of the piles was conventional and presented little difficulty, but the others caused some tense moments. Overhead clearance for the drilling gear at some locations was only 13 feet. A conventional drilling rig was too tall so the drilling subcontractor designed and built a telescoping kelly bar mounted on an industrial tractor. The unit allowed drilling to a depth of 54 feet without demolition of any overhead structure. Eight piles were drilled with it.

Old dewatering holes were discovered during construction. When the building was originally constructed, the area was dewatered with a series of 70-foot-deep wells

that were later filled with gravel and abandoned. Unfortunately, some of these wells were right where the new caissons needed to be. Where possible, the new piles were drilled the full 70 feet and locations were adjusted slightly to accommodate the old wells and still provide adequate bearing. One well presented difficulty, though.

"One was inside where we only had the ability to drill 54 feet," says Cynthia Kennec, Morley's project engineer. "We drilled a little and found it (the dewatering well). Then we called in a pressure-grouting company and pressure-grouted the gravel to solidify the mass down to 70 feet. We slowly injected a very liquid slurry of concrete and let it harden a few days, then located the new pile as far away as we could and still get the bearing the engineer wanted on the pile."

Placing the caissons. The water table under the building is at 25 feet, and since drilling went down beyond that, a bentonite slurry was used instead of a steel casing. The well bottoms had to be thoroughly clean and the concrete placed on undisturbed soil as the caissons were designed to be 80% end-bearing. The slurry was recycled through a de-silting unit until it reached a 4% silt level before any concrete could be placed. The pour of 4000-psi concrete was done by inserting a delivery hose to the bottom of the bentonite-filled well and injecting from the bottom up. The bentonite was pumped from the top as the pour progressed.

Reinforcing exterior columns and spandrel beams

Columns and spandrel beams in the first six stories of the exterior frame were strengthened to reduce drift—relative movement between adjacent floors in the building. Too much drift can cause significant damage to a building's structural framing system. It also can damage the building facade, interior ceilings or wall partitions, and piping.

One objective of the strengthening was to limit structural frame damage to minor cracking without noticeable concrete spalling. This reduces the likelihood of any downtime for the building after a major quake. During postquake inspections by government officials, even minor spalling could be viewed as cause for temporarily closing the building while safety was assessed. To minimize even the *appearance* of difficulty, Rockwell authorized additional reinforcement to make spalling less likely.

Strengthening was achieved by building a moment-resisting ductile concrete frame that was tied to the existing frame by 16,000 epoxy-anchored rebar dowels. This will limit interstory drift to about ¼ inch for the design worst-case earthquake.

Anchoring the new concrete. After the area that would bond to the new concrete was sandblasted, drilling the 16,000 anchor holes began. Each hole had to be precisely located to allow reinforcement mesh cages to be located properly, and all holes were drilled before adding the reinforcement.

Both sandblasting and hole drilling were done on swing stages and at night to reduce noise interference with Rockwell employees. Sixteen-inch dowels, drilled 8 inches into the original reinforced concrete, were set in epoxy. Due to their number and the multiple angles they are set at, dowels in column areas could only be set after column reinforcement units were completed and in place. Reaching through the cage ribs to insert and epoxy anchors was a tricky job: they were spaced as little as 3½ inches apart and some anchors were installed 4½ feet in from the face of the cage.

The reinforcing cages. The reinforcement for the exterior columns consists of a series of factory-built 5-foot structural mesh subunits assembled into column cages onsite. The steel mesh units were factory-built to improve quality control and speed operations. The horizontal bars in the mesh are spaced

Figure 3. At the junction of spandrel and column reinforcing, spandrel beam bars are threaded individually through the column steel. The column cage was assembled on the ground and hoisted into position by crane.

at 2½ inches and 3½ inches, and site assembly of individual tie bars would have been cumbersome.

While the spandrel beam reinforcement was assembled in place (Figure 3), the columns were assembled as a complete unit, flat on the ground including vertical steel rebar, and lifted into place.

Pouring the concrete. With the reinforcement in place, the forming could begin. Wood forms were used because they could most easily be adjusted to compensate for obstructions and varying dimensions in the strengthened sections. Superplasticized 4000-psi concrete was poured with a full liquid head in each 10-foot lift and a relatively small aggregate used because of closely spaced reinforcing steel and to reduce mechanical vibration.

Isolating the elevators

Unlike the rest of the structure, the elevator core had to be isolated at the foundation to keep elevator guide-rails straight and aligned and the elevators functioning. Cutting the core at the first-floor level would create a soft area where the rails could be dis-

torted during a quake, so the Teflon bearing plates were located at the lowest possible point.

To begin the installation, a moat was dug around the elevator core to provide a 14-inch clearance in any horizontal direction. The core was isolated a section at a time. Slots were cut in the core wall at the foundation and Teflon plates inserted. The plates were hand grouted and allowed to set and harden before the next set of slots were cut and the procedure repeated.

Rather than try to set forms in a 14-inch space, reinforced concrete block was used to construct the perimeter retaining wall. The block was laid in arm-length lifts working from above.

The first-floor diaphragm was detached from the elevator core and expansion joints installed to allow movement between the core and the nonisolated floor slab.

Making utilities flexible

From the start, the utilities presented complex problems. Electrical, telephone, computer connection, water, sewer, fire sprinkler, equipment cooling, heating,

and air conditioning services all had to be retrofitted to allow movement with the rest of the base-isolated structure. Flexible connections at the level of the isolators were required.

Pressurized lines were provided with specially designed universal joints allowing movement in all directions. Waste lines were cut and flexible braided stainless steel connections used. Ducting required special joints. Wiring had to be given room to move and had to be properly secured.

In one 24-hour period, building equipment was shut down, the large cooling lines drained, the lines cut, the new joint assemblies welded into place, the lines refilled and checked, and building functions resumed. Welders worked in shifts and backup workers and equipment were onsite and ready for any failure. One crew worked while another slept.

One of a kind

When all these separate phases of construction draw to a close, the Rockwell project will represent a unique application of base isolation. Although base isolation is being used in new structures and in historic structures like the City and County Building in Salt Lake City, this is the first time a building of this size will have been base-isolated while still fully occupied and operational.

Credits

Owner: Rockwell International Corp.

Structural engineer: Englekirk & Hart Consulting Engineers, Inc., Irvine, CA

Consulting engineer: Dames & Moore, Los Angeles

Isolation system: Dynamic Isolation Systems, Inc., Berkeley, CA

General contractor: Morley Construction Co., Santa Monica, CA

Subcontractor, isolator installation: Adams & Smith, Pleasant Hill, CA

Subcontractor, column cutting: Craftsman Concrete Cutting Co., El Monte, CA

Subcontractor, pile drilling: CMZ Drilling Co., Irwindale, CA

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Appendix IL

APPENDIX IL
TENTATIVE GENERAL REQUIREMENTS
FOR THE DESIGN AND CONSTRUCTION OF
SEISMIC-ISOLATED STRUCTURES

Ad Hoc Base Isolation Subcommittee
of the Seismology Committee
Structural Engineers Association
of California

Gary C. Hart (SEAOSC)

Charles A. Kircher (SEAONC)

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1990

Appendix 1L

**TENTATIVE GENERAL REQUIREMENTS
FOR THE DESIGN AND CONSTRUCTION OF
SEISMIC-ISOLATED STRUCTURES**

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**TENTATIVE GENERAL REQUIREMENTS
FOR THE DESIGN AND CONSTRUCTION OF
SEISMIC-ISOLATED STRUCTURES**

INTRODUCTION

Seismic isolation, commonly referred to as base isolation, is a design concept based on the premise that a structure can be substantially decoupled from potentially damaging earthquake motions. By decoupling the structure from ground vibration, seismic isolation reduces the level of response in the structure from the level which would otherwise occur in a conventional fixed-base building or, conversely, offers the advantage of designing with a reduced level of earthquake load to achieve the same degree of seismic protection and reliability as a conventional fixed-base building. In these provisions, it is assumed that the decoupling will be accomplished using an isolation scheme which makes the fundamental period of the seismic-isolated structure several times greater than the period of the elastic fixed-base structure above the isolation interface.

The potential advantages of seismic isolation and the recent advancements in isolation system products have already led to the design and construction of seismic-isolated buildings and bridges in California. This activity has, in turn, identified a need to supplement existing provisions with design requirements developed specifically for such buildings. This need is shared by the public which requires assurance that seismic-isolated buildings are "safe" and by the engineering profession which requires a minimum standard upon which design and construction can be based. Accordingly, the Ad Hoc Base Isolation Subcommittee of the SEAOC Seismology Committee has developed this appendix to supplement Chapter 1 of the 1990 SEAOC Recommended Lateral Force Requirements (hereinafter referred to as these Recommendations), with additional design requirements for seismic-isolated buildings.

This appendix was synthesized from two source documents; *Tentative Seismic Isolation Design Requirements, September 1986*, published by the Northern Section of SEAOC and "Draft Guidelines for the Design of Buildings with Base Isolators," prepared by the Ad Hoc Base Isolation Committee of the Southern Section of SEAOC and published in the Applied Technology Council's *Proceedings of a Seminar and Workshop on Base Isolation and Passive Energy Dissipation*, (ATC-17). In addition, this appendix reflects review comments provided by members of base isolation subcommittees active in the Northern and Southern Sections of SEAOC.

This appendix was prepared in keeping with the most current information and the present state-of-the-practice of seismic isolation. However, seismic isolation is a relatively new technology and there are many design-related issues

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that require additional research. In establishing the design requirements of this appendix, the Ad Hoc Base Isolation Subcommittee recognized these limitations and chose the most conservative approach in resolving conflicting points of view. It is anticipated that as experience with these types of structures increases and as the results of related research become available, the design requirements for seismic-isolated structures can be refined accordingly.

Rather than addressing a specific method of base isolation this appendix provides general design requirements applicable to a wide range of possible seismic isolation systems. In remaining general, the design requirements rely on mandatory testing of isolation system hardware to confirm the engineering parameters used in the design and to verify the overall adequacy of the isolation system. Some systems may not be capable of demonstrating acceptability by test and, consequently, would not be permitted. In general, acceptable systems will:

- remain stable for required design displacements,
- provide increasing resistance with increasing displacement,
- not degrade under repeated cyclic load, and
- have quantifiable engineering parameters (e.g., force-deflection characteristics and damping).

The requirements prescribe the use of either one of two different procedures for determining design-basis earthquake response. The first procedure is intended for use on stiff buildings of regular configuration located on stiff soil or rock sites, away from active faults. This procedure uses a formula (similar to the seismic-coefficient formula now used in conventional building design) to prescribe peak lateral displacement and force as a function of seismic zone, soil profile, proximity to active faults, and isolated-building period and damping.

The second approach, which is required for all other situations, relies on dynamic analysis procedures to determine maximum force and displacement response of the isolated building. Dynamic analysis procedures include both response spectrum (linear) analysis and time history (nonlinear) analysis. The latter is required for design of buildings with significantly nonlinear isolation systems and is an option for design of superstructures with inelastic elements. Initially, it is anticipated that a linear dynamic analysis will be used to compute member forces and drift of the superstructure. If this analysis indicates that one or more members are stressed beyond allowable values then either those members would be redesigned or inelastic response of the superstructure would be evaluated using nonlinear dynamic analysis. Marginally overstressed elements would be found acceptable provided story drift ratios calculated by nonlinear dynamic analysis were found to satisfy the drift limits in the criteria.

Both the static and dynamic analysis procedures are based on the same level of seismic input and require the same level of performance from the building. Design-basis earthquake load corresponds to a level of ground motion that has a 10 percent probability of being exceeded in a 50-year time period. For buildings not requiring a site-specific hazard analysis, the design-basis spectra are defined by the ground motion spectra recommended by ATC-3-06 and are essentially the same as those specified by these Recommendations for dynamic analysis of conventional fixed-based buildings. As an additional requirement, isolation system stability must be verified by test for the maximum level of earthquake ground motion that may ever be expected at the site. This earthquake intensity is defined as not less than 1.25 times design-basis earthquake load and may be taken as the level of ground motion that has a 10 percent probability of being exceeded in a 250 year time period when a site-specific hazard analysis is performed.

A. GENERAL

1. Every seismic-isolated structure and every portion thereof shall be designed and constructed in accordance with the requirements of this document and the applicable sections of Chapter 1 of these Recommendations.

a. The Lateral Force Resisting System and the Isolation System shall be designed to resist the deformations and stresses produced by the effects of seismic ground motions as provided in this document.

b. Where wind forces prescribed by these Recommendations produce greater deformations or stresses, such loads shall be used for design in lieu of the deformations and stresses resulting from earthquake forces.

B. DEFINITIONS

The definitions of Section 1B of these Recommendations and the following apply to the provisions of this division:

1. **DESIGN-BASIS EARTHQUAKE** is the level of earthquake ground shaking which has a 10 percent probability of being exceeded in a 50 year time period.

2. **DESIGN DISPLACEMENT** is the Design-Basis Earthquake lateral displacement, **excluding** additional displacement due to actual and accidental torsion, required for design of the Isolation System.

3. **EFFECTIVE DAMPING** is the value of equivalent viscous damping corresponding to energy dissipated during cyclic response of the Isolation System.

4. **EFFECTIVE STIFFNESS** is the value of the lateral force in the Isolation System, or an element thereof, divided by the corresponding lateral displacement.

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5. **ISOLATION INTERFACE** is the boundary between the upper portion of the structure, which is isolated, and the lower portion of the structure which moves rigidly with the ground.

6. **ISOLATION SYSTEM** is the collection of structural elements which includes all individual Isolator Units, all structural elements which transfer force between elements of the Isolation System, and all connections to other structural elements. The Isolation System also includes the Wind-Restraint System if such a system is used to meet the design requirements of this section.

7. **ISOLATOR UNIT** is a horizontally-flexible and vertically-rigid structural element of the Isolation System which permits large lateral deformations under design seismic load. An Isolator Unit may be used either as part of or in addition to the weight supporting system of the building.

8. **MAXIMUM CREDIBLE EARTHQUAKE** is the maximum level of earthquake ground shaking which may be expected at the building site within the known geological framework. This intensity may be taken as the level of earthquake ground motion that has a 10 percent probability of being exceeded in a 250-year time period.

9. **TOTAL DESIGN DISPLACEMENT** is the Design-Basis Earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for design of the Isolation System, or an element thereof.

10. **TOTAL MAXIMUM DISPLACEMENT** is the Maximum Credible Earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for verification of the stability of the Isolation System, or elements thereof, design of building separations, and vertical load testing of Isolator Unit prototypes.

11. **WIND-RESTRAINT SYSTEM** is the collection of structural elements which provide restraint of the seismic-isolated structure for wind loads. The Wind-Restraint System may be either an integral part of Isolator Units or may be a separate device.

C. SYMBOLS AND NOTATIONS

The symbols and notations of Section 1C of these Recommendations and the following provisions apply to the provisions of this document:

- B = Numerical coefficient related to the Effective Damping of the Isolation System as set forth in Table C.
- b = The shortest plan dimension of the structure, in feet, measured perpendicular to d.
- D = Design Displacement, in inches, at the center of rigidity of the Isolation System in the direction under consideration, as prescribed by Formula (E-1).
- D_T = Total Design Displacement, in inches, of an element of the Isolation System including both translational displacement at the center of rigidity, D, and the component of torsional displacement in the direction under consideration, as specified in Section E3c.
- D_{TM} = Total Maximum Displacement, in inches, of an element of the Isolation System including both translational displacement at the center of rigidity and the component of torsional displacement in the direction under consideration, as prescribed by Formula (E-4).
- d = The longest plan dimension of the structure, in feet.
- e = The actual eccentricity, in feet, measured in plan between the center of mass of the structure above the Isolation Interface and the center of rigidity of the Isolation System, plus accidental eccentricity, in feet, taken as 5 percent of d.
- F^- = Maximum negative force in an Isolator Unit during a single cycle of prototype testing at a displacement amplitude of Δ^- .
- $F_{\max}^-$ = Maximum negative force in an Isolator Unit for all cycles of prototype testing at a common displacement amplitude of Δ^- .
- $F_{\min}^-$ = Minimum negative force in an Isolator Unit for all cycles of prototype testing at a common displacement amplitude of Δ^- .
- F^+ = Maximum positive force in an Isolator Unit during a single cycle of prototype testing at a displacement amplitude of Δ^+ .
- $F_{\max}^+$ = Maximum positive force in an Isolator Unit for all cycles of prototype testing at a common displacement amplitude of Δ^+ .
- $F_{\min}^+$ = Minimum positive force in an Isolator Unit for all cycles of prototype testing at a common displacement amplitude of Δ^+ .
- k_{eff} = Effective Stiffness of an Isolator Unit, as prescribed by Formula (L-1).

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- $k_{\max}$ = Maximum Effective Stiffness of the Isolation System at the Design Displacement in the horizontal direction under consideration.
- $k_{\min}$ = Minimum Effective Stiffness of the Isolation System at the Design Displacement in the horizontal direction under consideration.
- N = Numerical coefficient related to the proximity of the building or structure to active faults as set forth in Table B.
- R_{wI} = Numerical coefficient related to the type of Lateral Force Resisting System above the Isolation Interface as set forth in Table D for seismic-isolated structures.
- S_I = Numerical coefficient for site-soil profile as set forth in Table A for seismic-isolated structures.
- T_I = Period of seismic-isolated structure, in seconds, in the direction under consideration, as prescribed by Formula (E-2).
- V_b = The total lateral seismic design force or shear on elements at or below the Isolation Interface, as prescribed by Formula (E-5).
- V_s = The total lateral seismic design force or shear on elements above the Isolation Interface, as prescribed by Formula (E-6).
- W = The total seismic dead load defined in Section 1E1c of these Recommendations. For design of the Isolation System, W is the total seismic dead load weight of the structure above the Isolation Interface.
- y = The distance, in feet, between the center of rigidity of the Isolation System rigidity and the element of interest, measured perpendicular to the direction of seismic loading under consideration.
- Δ^+ = Maximum positive displacement of an Isolator Unit during each cycle of prototype testing.
- Δ^- = Maximum negative displacement of an Isolator Unit during each cycle of prototype testing.
- β = Effective Damping of the Isolation System, as prescribed by Formula (L-2).

D. CRITERIA SELECTION

1. **Basis for Design.** The procedures and limitations for the design of seismic-isolated structures shall be determined considering zoning, site characteristics, occupancy, configuration, structural system, and height in accordance with Section 1D of these Recommendations, except as noted below.
2. **Stability of the Isolation System.** The stability of the vertical load carrying elements of the Isolation System shall be verified by analysis and test, as required, for lateral seismic displacement equal to the Total Maximum Displacement.

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3. Occupancy Categories. The Importance Factor, I , for a seismic-isolated building shall be taken as 1.0, regardless of occupancy category.

4. Configuration Requirements. Each structure shall be designated as being regular or irregular on the basis of the structural configuration above the Isolation Interface.

5. Selection of Lateral Response Procedure. Any seismic-isolated structure may be, and certain seismic-isolated structures defined below shall be, designed using the dynamic lateral response procedure of Section F.

a. Static Analysis. The static lateral response procedure of Section E may be used for design of a seismic-isolated structure, provided:

- (1) The structure is located at least 15 km from all active faults.
- (2) The structure is located on a soil profile with a site factor of S_1 or S_2 .
- (3) The structure is located in Seismic Zone 3 or 4.
- (4) The structure above the Isolation Interface is equal to or less than four stories, or 65 feet, in height.
- (5) The isolated period of the structure, T_1 , is equal to or less than 3.0 seconds.
- (6) The isolated period of the structure, T_1 , is greater than 3 times the elastic, fixed-base period of the structure above the Isolation Interface, as determined by Formula (1-5) of these Recommendations.
- (7) The structure above the Isolation Interface is of regular configuration.
- (8) The Isolation System does not limit the Total Maximum Displacement to less than 1.5 times the Total Design Displacement.
- (9) The Isolation System is defined by all of the following attributes:

- (a) The Effective Stiffness of the Isolation System at the Design Displacement is greater than one-third of the Effective Stiffness at 20% of the Design Displacement.
- (b) The Isolation System is capable of producing a restoring force, as specified in Section H2d.
- (c) The Isolation System has force deflection properties which are independent of the rate of loading.
- (d) The Isolation System has force deflection properties which are independent of vertical load and bilateral load.

b. Dynamic Analysis. The dynamic lateral response procedure of Section F shall be used for design of seismic-isolated structures as specified below:

- (1) **Response Spectrum Analysis.** Response spectrum analysis may be used for design of a seismic-isolated structure, provided:

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- (a) The structure is located on a soil profile with a site factor of S_1 , S_2 or S_3 .
- (b) The Isolation System is defined by all of the attributes specified in Sections D5a(8) and D5a(9).
- (2) **Time History Analysis.** Time history analysis may be used for design of any seismic-isolated structure and shall be used for design of all seismic-isolated structures not meeting the criteria of Section D5b(1).
- (3) **Site-Specific Design Spectra.** Site-specific ground motion spectra of the Design-Basis Earthquake and the Maximum Credible Earthquake, developed in accordance with Section 1F2 of these Recommendations, shall be used for design and analysis of all seismic-isolated structures as specified below.
 - (a) The structure is located on a soil profile with a site factor of S_3 or S_4 .
 - (b) The structure is located within 15 km of an active fault.
 - (c) The structure is located in Seismic Zone 1, 2A or 2B.
 - (d) The isolated period of the structure, T_1 , is greater than 3.0 seconds.

E. STATIC LATERAL RESPONSE PROCEDURE

1. **General.** Except as provided in Section F, Dynamic Lateral Force Procedure, every seismic-isolated structure, or portion thereof, shall be designed and constructed to resist minimum earthquake displacements and forces as specified by this section and the applicable requirements of Section 1E of these Recommendations.
2. **Deformation Characteristics of the Isolation System.**
 - a. Minimum lateral earthquake design displacements and forces on seismic-isolated structures shall be based on the deformation characteristics of the Isolation System.
 - b. The deformation characteristics of the Isolation System shall explicitly include the effects of the Wind-Restraint System, if such a system is used to meet the design requirements of this document.
 - c. The deformation characteristics of the Isolation System shall be based on properly substantiated tests performed in accordance with Section L of this document.
3. **Minimum Lateral Displacements.**
 - a. **Design Displacement.** The Isolation System shall be designed and constructed to withstand minimum lateral earthquake displacements which act in the direction of each of the main horizontal axes of the structure in accordance with the formula:

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$$D = 10ZNS_I T_I / B \quad (E-1)$$

b. **Isolated-Structure Period.** The isolated-structure period, T_I , shall be determined using the deformational characteristics of the Isolation System in accordance with the formula:

$$T_I = 2\pi \sqrt{\frac{W}{k_{\min} g}} \quad (E-2)$$

c. **Total Design Displacement.** The Total Design Displacement, D_T , of elements of the Isolation System shall include additional displacement due to actual and accidental torsion calculated considering the spatial distribution of the lateral stiffness of the Isolation System and the most disadvantageous location of mass eccentricity.

(1) The Total Design Displacement, D_T , of elements of an Isolation System with uniform spatial distribution of lateral stiffness shall not be taken as less than that prescribed by the formula:

$$D_T = D [1 + \gamma(12e/(b^2 + d^2))] \quad (E-3)$$

(2) The Total Design Displacement, D_T , may be taken as less than the value prescribed by Formula (E-3), but not less than 1.1 times D , provided the Isolation System is shown by calculation to be configured to resist torsion accordingly.

d. **Total Maximum Displacement.** The Total Maximum Displacement, D_{TM} , required for verification of Isolation System stability in the most critical direction of horizontal response shall be calculated in accordance with the formula:

$$D_{TM} = 1.5 D_T \quad (E-4)$$

4. Minimum Lateral Forces.

a. **Structural Elements At or Below the Isolation Interface.** The Isolation System, the foundation, and all structural elements below the Isolation Interface shall be designed and constructed to withstand a minimum lateral seismic force, V_b , using all of the appropriate provisions of these Recommendations for a non-isolated structure where:

$$V_b = \frac{k_{\max} D}{1.5} \quad (E-5)$$

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b. **Structural Elements Above the Isolation Interface.** The structure above the Isolation Interface shall be designed and constructed to withstand a minimum shear force, V_s , using all of the appropriate provisions for a non-isolated structure where:

$$V_s = \frac{k_{\max} D}{R_{wI}} \quad (E-6)$$

The R_{wI} factor shall be based on the type of Lateral Force Resisting System used for the structure above the Isolation System.

c. **Limits on V_s .** The value of V_s shall not be taken as less than the following:

- (1) The lateral seismic force required by these Recommendations for a fixed-base structure of the same weight, W , and a period equal to the isolated period, T_I .
- (2) The base shear corresponding to the design wind load, and
- (3) The lateral seismic force required to fully activate the Isolation System (e.g., the yield level of a softening system, the ultimate capacity of a sacrificial Wind-Restraint System or the static friction level of a sliding system).

5. **Vertical Distribution of Force.** The total force shall be distributed over the height of the structure above the Isolation Interface in accordance with the following formula:

$$F_x = \frac{V_s w_x}{\sum_{i=1}^n w_i} \quad (E-7)$$

At each level designated as x , the force F_x shall be applied over the area of the building in accordance with the mass distribution at the level. Stresses in each structural element shall be calculated as the effect of force, F_x , applied at the appropriate levels above the base.

6. **Drift Limits.** The maximum interstory drift ratio of the structure above the Isolation System shall not exceed $0.010/R_{wI}$.

F. DYNAMIC LATERAL RESPONSE PROCEDURE

1. **General.** As required by Section D5, every seismic-isolated structure, or portion thereof, shall be designed and constructed to resist earthquake displacements and forces as specified in this section and the applicable requirements of Section 1F of these Recommendations.

a. Isolation System and Structural Elements Below the Isolation Interface.

- (1) The Total Design Displacement of the Isolation System shall not be taken as less than 90 percent of D_T , specified by Section E3c.
- (2) The Total Maximum Displacement of the Isolation System shall not be taken as less than 80 percent of D_{TM} , prescribed by Formula (E-4).
- (3) The design lateral shear force on the Isolation System and structural elements below the Isolation Interface, shall not be taken as less than 90 percent of V_b , prescribed by Formula (E-5).

b. Structural Elements Above The Isolation Interface.

- (1) The design lateral shear force on the structure above the Isolation Interface, if regular in configuration, shall not be taken as less than 80 percent of $k_{max}D/R_{wI}$, nor less than the limits specified by Section E4c.

EXCEPTION: The design lateral shear force on the structure above the Isolation Interface, if regular in configuration, may be taken as less than 80 percent of $k_{max}D/R_{wI}$, but not less than 60 percent of $k_{max}D/R_{wI}$, provided time history analysis is used for design of the structure.

- (2) The design lateral shear force on the structure above the Isolation Interface, if irregular in configuration, shall not be taken as less than $k_{max}D/R_{wI}$, nor less than the limits specified by Section E4c.

EXCEPTION: The design lateral shear force on the structure above the Isolation Interface, if irregular in configuration, may be taken as less than $k_{max}D/R_{wI}$, but not less than 80 percent of $k_{max}D/R_{wI}$, provided time history analysis is used for design of the structure.

2. Ground Motion.

a. **Design Spectra.** Properly substantiated, site-specific spectra are required for design of all structures with an isolated period, T_1 , greater than 3.0 seconds, or located on a soil type profile of S_3 , or S_4 , or located within 15 km of an active fault or located in Seismic Zones 1, 2A or 2B. Structures not requiring site-specific spectra shall be designed using spectra based on Figure 1B of these Recommendations.

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(1) A design spectrum shall be constructed for the Design-Basis Earthquake. This design spectrum shall not be taken as less than the normalized response spectrum given in Figure 1B of these Recommendations for the appropriate soil type, scaled by the Seismic Zone coefficient.

EXCEPTION: If a site-specific spectrum is calculated for the Design-Basis Earthquake, then the design spectrum may be taken as less than 100 percent, but not less than 80 percent of the normalized response spectrum given in Figure 1B of these Recommendations for the appropriate soil type, scaled by the Seismic Zone coefficient.

(2) A design spectrum shall be constructed for the Maximum Credible Earthquake. This design spectrum shall not be taken as less than 1.25 times the Design Basis Earthquake spectrum. This design spectrum shall be used to determine the Total Maximum Displacement for testing of the stability of the base-isolation system.

b. **Time Histories.** Pairs of horizontal ground motion time history components shall be selected from not less than three recorded events. These motions shall be scaled such that the square root sum of the squares (SRSS) of the 5%-damped spectrum of the scaled horizontal components does not fall below 1.3 times the 5%-damped spectrum of the Design-Basis Earthquake (or Maximum Credible Earthquake) by more than 10% in the period range of T_1 , as determined by Formula (E-2), for periods from T_1 minus 1.0 seconds to T_1 plus 2.0 seconds.

(1) The duration of the time histories shall be consistent with the magnitude and source characteristics of the Design-Basis Earthquake (or Maximum Credible Earthquake).

(2) Time histories developed for sites within 15 km of a major active fault shall incorporate near-fault phenomena.

3. **Mathematical Model.**

a. **General.** The mathematical model(s) of the isolated structure, including the Isolation System, the Lateral Force Resisting System and other structural elements, shall conform to Section 1F3 of these Recommendations and to the following requirements of Sections F3b and F3c.

b. **Isolation System.**

(1) The Isolation System shall be modeled using deformational characteristics developed and verified by test in accordance with the requirements of Section E2.

(2) The Isolation System shall be modeled with sufficient detail to:

(a) account for the spatial distribution of Isolator Units,

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(b) calculate translation, in both horizontal directions, and torsion of the structure above the Isolation Interface, considering the most disadvantageous location of mass eccentricity,

(c) assess overturning/uplift forces on individual Isolator Units.

(d) account for the effects of vertical load, bilateral load and/or the rate of loading if the force deflection properties of the Isolation System are dependent on one or more of these attributes.

c. **Isolated Structure.**

(1) The maximum displacement of each floor and the Total Design Displacement and Total Maximum Displacement across the Isolation System shall be calculated using a model of the isolated structure which incorporates the force-deflection characteristics of nonlinear elements of the Isolation System and the Lateral Force Resisting System.

(a) Isolation Systems with nonlinear elements include, but are not limited to, systems which do not meet the criteria of Section D5a(9).

(b) Lateral Force Resisting Systems with nonlinear elements include, but are not limited to, irregular structural systems designed for a lateral force less than $k_{\max}D/R_{wI}$ and regular structural systems designed for a lateral force less than 80 percent of $k_{\max}D/R_{wI}$.

(2) Design forces and displacements in key elements of the Lateral Force Resisting System may be calculated using a linear elastic model of the isolated structure, provided:

(a) pseudo-elastic properties assumed for nonlinear Isolation System components are based on the maximum Effective Stiffness of the Isolation System.

(b) all key elements of the Lateral Force Resisting System are linear.

4. **Description of Analysis Procedures**

a. **General.** A response spectrum analysis and/or a time history analysis shall be performed in accordance with Sections 1F4 and 1F5 of these Recommendations and the following requirements.

b. **Input Earthquake.** The Design-Basis Earthquake shall be used to calculate the Total Design Displacement of the Isolation System and the lateral forces and displacements of the isolated structure. The maximum Credible Earthquake shall be used to calculate the Total Maximum Displacement of the Isolation System.

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c. **Response Spectrum Analysis.**

(1) Response spectrum analysis shall be performed using a damping value equal to the Effective Damping of the Isolation System or 30 percent of critical, whichever is less.

(2) Response spectrum analysis used to determine the Total Design Displacement and the Total Maximum Displacement shall include simultaneous excitation of the model by 100 percent of the most critical direction of ground motion and 30 percent of the ground motion on the orthogonal axis.

d. **Time History Analysis.**

(1) Time history analysis shall be performed with at least three appropriate pairs of horizontal time history components, as defined in Section F2b.

(2) Each pair of time histories shall be applied simultaneously to the model, considering the most disadvantageous location of mass eccentricity.

(3) The maximum response of the parameter of interest calculated by the three time history analyses shall be used for design.

5. **Design Lateral Force.**

a. **Structural Elements At or Below the Isolation Interface.** The Isolation System, the foundation and all structural elements below the Isolation Interface shall be designed using all of the appropriate provisions for a non-isolated structure and the forces obtained from the dynamic analysis reduced by a factor of 1.5

b. **Structural Elements Above the Isolation Interface.** Structural elements above the Isolation Interface shall be designed using all of the appropriate provisions for a non-isolated structure and the forces obtained from the dynamic analysis reduced by a factor of R_{wI} . The R_{wI} factor shall be based on the type of Lateral Force Resisting System used for the structure above the Isolation System.

c. **Scaling of Results.** When the factored lateral shear force on structural elements, determined using either response spectrum or time history analysis, is less than the minimum level prescribed by Section F1, then all response parameters, including member forces and moments shall be adjusted upward proportionally.

6. **Drift Limits.** Maximum interstory drift corresponding to the design lateral force shall not exceed the following limits.

a. **Response Spectrum Analysis.** The maximum interstory drift ratio of the structure above the Isolation System, calculated by response spectrum analysis, shall not exceed $0.015/R_{wI}$.

b. **Time History Analysis.** The maximum interstory drift ratio of the structure above the Isolation System, calculated by time history analysis considering the force-deformation characteristics of nonlinear elements of the Lateral Force Resisting System, shall not exceed $0.020/R_{WI}$.

c. **P-Delta.** The secondary effects of the Maximum Credible Earthquake lateral displacement (δ) of the structure above the Isolation System combined with gravity forces shall be investigated if the interstory drift ratio exceeds $0.010/R_{WI}$.

G. LATERAL LOAD ON ELEMENTS OF STRUCTURES AND NON-STRUCTURAL COMPONENTS SUPPORTED BY STRUCTURES.

1. **General.** Parts or portions of an isolated structure, permanent non-structural components and the attachments to them, and the attachments for permanent equipment supported by a structure shall be designed to resist seismic forces and displacements as prescribed by this section and the applicable requirements of Section 1G of these Recommendations.

2. Forces and Displacements

a. **Components At or Above The Isolation Interface.** Elements of seismic-isolated structures and non-structural components, or portions thereof, which are at or above the Isolation Interface, shall be designed to resist a total lateral seismic force equal to the maximum dynamic response of the element or component under consideration divided by a factor of 1.5.

EXCEPTION: Elements of seismic-isolated structures and non-structural components, or portions thereof, may be designed to resist total lateral seismic force as prescribed by Formula (1-10) of these Recommendations.

b. **Components Which Cross the Isolation Interface.** Elements of seismic-isolated structures and non-structural components, or portions thereof, which cross the Isolation Interface shall be designed to withstand the Total Maximum Displacement.

c. **Components Below The Isolation Interface.** Elements of seismic-isolated structures and non-structural components, or portions thereof, which are below the Isolation Interface shall be designed and constructed in accordance with the requirements of Section 1G of these Recommendations.

H. DETAILED SYSTEMS REQUIREMENTS.

1. **General.** The Isolation System and the structural system shall comply with the requirements of Section D of this document and the material requirements of Chapters 2 through 6 of these Recommendations. In addition, the Isolation System shall comply with the detailed system requirements of this section and the structural system shall comply with the detailed system requirements of this section and the applicable portions of Section IH of these Recommendations.

2. Isolation System.

a. **Environmental Conditions.** In addition to the requirements for vertical and lateral loads induced by wind and earthquake, the Isolation System shall be designed with consideration given to other environmental conditions including aging effects, creep, fatigue, operating temperature and exposure to moisture or damaging substances.

b. **Wind Forces.** Isolated structures shall resist design wind loads at all levels above the Isolation Interface in accordance with the general wind design provisions. At the Isolation Interface a Wind Restraint System shall be provided to limit lateral displacement in the Isolation System to a value equal to that required between floors of the structure above the Isolation Interface.

c. **Fire Resistance.** Fire resistance for the Isolation System shall meet that required for the building's columns, walls, or other structural elements.

d. Lateral Restoring Force.

(1) The Isolation System shall be configured to produce a restoring force such that the lateral force at the Total Design Displacement is at least $0.025W$ greater than the lateral force at 50 percent of the Total Design Displacement.

EXCEPTION: The Isolation System need not be configured to produce a restoring force, as required above, provided the Isolation System is capable of remaining stable under full vertical load and accommodating a Total Maximum Displacement equal to the greater of either 3.0 times the Total Design Displacement or $36ZNS_I$ inches.

e. **Displacement Restraint.** If the Isolation System is configured to limit the Total Maximum Displacement to less than 1.5 times the Total Design Displacement, then the Isolation System shall be designed for lateral forces corresponding to the Maximum Credible Earthquake, and the structure above the Isolation System shall be checked for stability and ductility demand of the Maximum Credible Earthquake. The Total Maximum Displacement required for vertical load testing of Isolater Unit prototypes, building separations and verification of Isolation System stability shall be calculated using time history analysis, but shall not be taken as less than 1.2 times the Total Design Displacement.

f. **Vertical Load Stability.** The Isolation System shall provide a factor of safety of three (3) for vertical loads (dead load plus live load) in its laterally undeformed state. It shall also be designed to be stable under the full design vertical loads at a horizontal displacement equal to the Total Maximum Displacement.

g. **Overturning.**

(1) The factor of safety against global structural overturning at the Isolation Interface shall be not less than 1.0 for the load combinations required by these Recommendations. All gravity and seismic loading conditions shall be investigated, except that seismic forces for overturning calculations shall be based on the maximum base shear for the superstructure; and W shall be used for the vertical restoring force.

(2) Local uplift of individual elements is permitted provided the resulting deflections do not cause overstress or instability of the Isolator Units or other building elements.

h. **Inspection and Replacement.** Access for inspection and replacement of all components of the Isolation system shall be provided.

i. **Quality Control.** A quality control testing program for Isolation Units shall be established by the engineer responsible for the structural design.

3. **Structural System.**

a. **Horizontal Distribution of Force.** A horizontal diaphragm or other structural elements shall provide continuity above the Isolation Interface and shall have adequate strength and ductility to transmit forces (due to non-uniform ground motion) from one part of the building to another.

b. **Building Separations.** Minimum separations between the isolated building and surrounding retaining walls or other fixed obstructions shall be not less than the Total Maximum Displacement.

I. **NON-BUILDING STRUCTURES**

a. **General.** Non-building structures shall be designed in accordance with the requirements of Section II of these Recommendations using design displacements and forces calculated in accordance with Section E or Section F of this document.

J. **FOUNDATIONS**

a. **General.** Foundations shall be designed and constructed in accordance with the requirements of Section 1J of these Recommendations using design forces calculated in accordance with Section E or Section F of this document.

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K. DESIGN AND CONSTRUCTION REVIEW

1. **General.** Design and construction review of all seismic-isolated structures shall be performed in accordance with Section 1K of these Recommendations and the following requirements.

a. Design review of the Isolation System and related test programs shall be performed by an independent engineering team including persons licensed in the appropriate disciplines, experienced in seismic analysis methods and the theory and application of seismic isolation.

2. **Isolation System.** Isolation System design review shall include, but not be limited to, the following:

a. Review of site-specific seismic criteria, including the development of site-specific spectra and ground motion time histories, and all other design criteria developed specifically for the project.

b. Review of the preliminary design, including the determination of the Total Design Displacement of the Isolation System design displacement and lateral force design level.

c. Overview and observation of prototype testing (Section L).

d. Review of the final design of the entire structural resisting system and all supporting analyses.

e. Review of the Isolation System quality control testing program (Section H2i).

L. REQUIRED TESTS OF ISOLATION SYSTEM

1. **General.**

a. The deformation characteristics and damping values of the Isolation System used in the design and analysis of seismic-isolated structures shall be based on the following tests of a selected sample of the components prior to construction.

b. The Isolation System components to be tested shall include the Wind Restraint System if such systems are used in the design.

c. The tests specified in this section are for establishing and validating the design properties of the Isolation System, and shall not be considered as satisfying the manufacturing quality control tests of Section H2i.

2. **Prototype Tests**

a. Prototype tests shall be performed separately on two full-size specimens of each type and size of Isolator Unit of the Isolation System. The test specimens shall include the Wind Restraint System, as well as individual Isolator Units, if such systems are used in the design. Specimens tested shall not be used for construction.

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- b. For each cycle of tests the force-deflection and hysteretic behavior of the test specimen shall be recorded.
- c. The following sequence of tests shall be performed for the prescribed number of cycles at a vertical load equal to the typical or average DL on all Isolator Units of a common type and size.
- (1) Twenty full reversed cycles of loading at a lateral force corresponding to the wind design force.
 - (2) Three fully reversed cycles of loading at each of the following increments of the Total Design Displacement; 0.25, 0.50, 0.75 and 1.0.
 - (3) $15S_1/B$, but not less than ten, fully reversed cycles of loading at 1.0 times the Total Design Displacement and a vertical load equal to DL.
- d. If an Isolator Unit is also a vertical load carrying element, then Item (2) of the sequence of cyclic tests specified in Section L2c shall be performed for two additional vertical load cases:
- (1) $1.2DL + 0.5LL + |E|$
 - (2) $0.8DL - |E|$

where DL, LL and E are as defined in these Recommendations. In these tests, the combined vertical load shall be taken as the typical or average downward force on all Isolator Units of a common type and size.

e. If the force deflection properties of the Isolator Units are dependent on the rate of loading, then each set of tests specified in Sections L2c and L2d shall be performed dynamically at three different rates of loading. The frequency of loading shall correspond to 1/2, 1, and 2 times the inverse of the isolated structure period, defined as a cycle of response at the Total Design Displacement.

EXCEPTION: Properly documented dynamic tests of scaled specimens may be used to quantify the properties of rate dependent systems, in lieu of the above specified dynamic tests. The scaled specimens shall be representative of full-scale prototype specimens.

The force deflection properties of an Isolator Unit shall be considered to be dependent on the rate of loading if there is greater than a plus or minus 10 percent change in the Effective Stiffness at the Design Displacement for either a factor of two increase or a factor of two decrease in the rate of loading, where the frequency of loading is taken as the inverse of the isolated structure period.

f. If the force deflection properties of the Isolator Units are dependent on bilateral load, then the tests specified in Sections L2c, L2d and L2e shall be augmented to include bilateral load at increments of the Total Design Displacement; 0.25 and 1.0, 0.50 and 1.0, 0.75 and 1.0, and 1.0 and 1.0.

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EXCEPTION: Properly documented dynamic tests of scaled specimens may be used to quantify the properties of direction dependent systems, in lieu of the above specified tests. The scaled specimens shall be representative of full-scale prototype specimens.

The force deflection properties of an Isolator Unit shall be considered to be dependent on bilateral load, if the bilateral and unilateral force deflection properties have greater than a 10 percent difference in Effective Stiffness at the Design Displacement.

g. The vertical load carrying elements of the Isolation System shall be statically tested for maximum and minimum downward vertical load, at the Total Maximum displacement. In these tests, the combined vertical load of $1.2DL + 1.0LL + |E|$ shall be taken as the maximum downward force, and the combined vertical load of $0.8DL - |E|$ shall be taken as the minimum downward force, on any one Isolator Unit of a common type and size.

h. If a sacrificial Wind Restraint System is to be utilized, then the ultimate capacity shall be established by test.

i. The prototype tests are not required if an Isolator Unit of similar size and of the same type and material as the prototype Isolator Unit has been previously tested using the specified sequence of tests.

3. Determination Of Force-Deflection Characteristics.

a. The force deflection characteristics of the Isolation System shall be based on the cyclic load test results for each fully reversed cycle of loading.

b. The Effective Stiffness of an Isolator Unit shall be calculated for each cycle of loading as follows:

$$k_{eff} = \frac{F^+ - F^-}{\Delta^+ - \Delta^-} \quad (L-1)$$

where F^+ , and F^- are the maximum positive and maximum negative forces, respectively, and Δ^+ and Δ^- are the maximum positive and maximum negative test displacements, respectively.

(1) If the minimum Effective Stiffness is to be determined then F_{min}^+ and F_{min}^- shall be used in the equation.

(2) If the maximum Effective Stiffness is to be determined then F_{max}^+ and F_{max}^- shall be used in the equation.

4. System Adequacy.

a. The performance of the test specimens shall be assessed as adequate if the following conditions are satisfied:

(1) The force deflection plots of all tests specified in Section L2 have a positive incremental force-carrying capacity.

(2) For each increment of test displacement specified in Section L2c(2) and for each vertical load case specified in Section L2d:

(a) For each test specimen there is no greater than a 10 percent difference between the Effective Stiffness at each of the three cycles of test and the average value of Effective Stiffness.

(b) There is no greater than a 10 percent difference in the average value of Effective Stiffness of the two test specimens of a common type and size of the Isolator Unit over the required three cycles of test.

(3) For each specimen there is no greater than a plus or minus 20 percent change in the initial Effective Stiffness of each test specimen over the $15S_I/B$, but not less than ten, cycles of test specified in Section L2c(3).

(4) For each specimen there is no greater than a 20 percent decrease in the initial Effective Damping over for the $15S_I/B$, but not less than ten, cycles of test specified in Section L2c(3).

(5) All specimens of vertical load carrying elements of the Isolation System remain stable at the Total Maximum Displacement for static load as prescribed in Section L2g.

5. Design Properties of the Isolation System.

a. **Effective Stiffness.** The minimum and maximum Effective Stiffnesses of the Isolation System shall be determined as follows:

(1) The value of k_{min} shall be based on the minimum Effective Stiffnesses of individual Isolator Units as established by the cyclic tests of Section L2c(2) at a displacement amplitude equal to the Design Displacement.

(2) The value of k_{max} shall be based on the maximum Effective Stiffnesses of individual Isolator Units as established by the cyclic tests of Section L2c(2) at a displacement amplitude equal to the Design Displacement.

(3) For Isolator Units that are found by the tests of Sections L2d, L2e and L2f to have force deflection characteristics which vary with vertical load, rate of loading or bilateral load, respectively, the value of k_{max} shall be increased and the value of k_{min} shall be decreased, as necessary, to bound the effects of measured variation in Effective Stiffness.

b. **Effective Damping.** The Effective Damping β of the Isolation System shall be calculated as:

$$\beta = \frac{1}{2\pi} \left[\frac{\text{Total Area}}{k_{max}D^2} \right] \quad (L-2)$$

where the Total Area shall be taken as the sum of the areas of the hysteresis loops of all Isolator Units and the hysteresis loop area of each Isolator Unit shall be taken as the minimum area of the three hysteresis loops established by the cyclic tests of Section L2c(2) at a displacement amplitude equal to the Design Displacement.

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APPENDIX 1L

TABLE A
SITE COEFFICIENTS
FOR SEISMIC-ISOLATED STRUCTURES

Soil Profile Types ⁽¹⁾	S_I Factor
S_1	1.0
S_2	1.5
S_3	2.0
S_4	2.7

⁽¹⁾ See Table 1-B of these Recommendations for a description of soil profile types and related requirements.

TABLE B
NEAR-FIELD RESPONSE COEFFICIENT, N

Response Coefficient	Closest Distance, d_F , to an Active Fault ^{(1),(2)}		
	$d_F > 15\text{km}$	$d_F = 10\text{km}$	$d_F < 5\text{km}$
N	1.0	1.2	1.5

⁽¹⁾ Active faults shall be established from properly substantiated geotechnical data (e.g., most recent mapping of active faults by the United States Geological Survey or the California Division of Mines and Geology).

⁽²⁾ The near-fault response coefficient shall be based on linear interpolation for values of the closest distance, d_F , to an active fault other than those given.

TABLE C
DAMPING COEFFICIENT, β

Damping Coefficient	Effective Damping (Percent of Critical) ^{(1),(2)}						
	$< 2\%$	5%	10%	20%	30%	40%	$> 50\%$
β	0.8	1.0	1.2	1.5	1.7	1.9	2.0

⁽¹⁾ The damping coefficient shall be based on the Effective Damping of the Isolation System determined in accordance with the requirements of Section L5 of this Appendix.

⁽²⁾ The damping coefficient shall be based on linear interpolation for Effective Damping values other than those given.

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TABLE D
STRUCTURAL SYSTEMS ABOVE THE ISOLATION INTERFACE

BASIC STRUCTURAL SYSTEM ⁽¹⁾	LATERAL LOAD-RESISTING SYSTEM—DESCRIPTION	R _{wt} ⁽⁵⁾	H ⁽²⁾
A. Bearing Wall System	1. Light-framed Walls with Shear Panels		
	a. Plywood walls for structures three stories or less	2.6	65
	b. All other light-framed walls	2.2	65
	2. Shear Walls		
	a. Concrete	2.6	160
	b. Masonry	2.6	160
	3. Light-steel-framed Bearing Walls with Tension-only Bracing	1.5	65
	4. Braced Frames Where Bracing Carries Gravity Loads		
	a. Steel	1.8	160
	b. Concrete ⁽³⁾	1.5	—
	c. Heavy timber	1.5	65
B. Building Frame System	1. Steel Eccentric Braced Frame (EBF)	3.0	240
	2. Light-framed Walls with Shear Panels		
	a. Plywood walls for structures three stories or less	2.6	65
	b. All other light-framed walls	2.2	65
	3. Shear Walls		
	a. Concrete	3.0	240
	b. Masonry	3.0	160
	4. Concentric Braced Frames		
	a. Steel	2.2	160
	b. Concrete ⁽³⁾	2.6	—
	c. Heavy timber	2.6	65
C. Moment Resisting Frame System	1. Special Moment-resisting Frames (SMRF)		
	a. Steel	3.0	N.L. ⁽⁴⁾
	b. Concrete	3.0	N.L.
	2. Concrete Intermediate Moment-resisting Frames (IMRF) ⁽⁶⁾	2.2	—
	3. Ordinary Moment-resisting Frames (OMRF)		
	a. Steel	1.8	160
D. Dual System	1. Shear Walls		
	a. Concrete with SMRF	3.0	N.L.
	b. Concrete with concrete IMRF ⁽⁶⁾	2.6	—
	c. Masonry with SMRF	3.0	160
	d. Masonry with concrete IMRF ⁽³⁾	2.6	—
	2. Steel EBF with Steel SMRF	2.6	N.L.
	3. Concentric Braced Frames	—	—
	a. Steel with steel SMRF	2.2	N.L.
	b. Concrete with concrete SMRF ⁽³⁾	2.2	—
	c. Concrete with concrete IMRF ⁽³⁾	1.8	—
E. Undefined Systems	See Section 1D9b	—	—

NOTES

(1) Basic structural systems are defined in Section 1D6.

(2) H=height limit applicable to Seismic Zones 3 and 4. See Section 1D7 for exceptions.

(3) Prohibited in Seismic Zones Nos. 3 and 4.

(4) N.L.=No limit.

(5) See Section 1E3 for combination of structural system.

(6) Prohibited in Seismic Zones 3 and 4, except as permitted in Section 1I2.

CR 89.006

January 1989

NCEL

Contract Report

Investigation Conducted by
University of California, Los Angeles

Sponsored by Naval Facilities
Engineering Command

Issues Related to the Selection of U.S. Navy Buildings for Base Isolation

ABSTRACT This report presents research conducted at UCLA. It recommends the base isolation systems that are fully satisfactory for Navy buildings. The report contains a discussion of the building selection process, quality control and design issues. A technical discussion of ground motion, torsion and uncertainty is also presented.

NAVAL CIVIL ENGINEERING LABORATORY PORT HUENEME, CALIFORNIA 93043

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures				Approximate Conversions from Metric Measures			
Symbol	When You Know	Multiply by	To Find	Symbol	When You Know	Multiply by	To Find
In ft yd mi	Inches feet yards miles	LENGTH		mm cm m km	millimeters centimeters meters kilometers	LENGTH	
		2.5	centimeters			0.04	Inches
		30	centimeters			0.4	Inches
		1.6	meters			3.3	feet
In ² ft ² yd ² mi ²	square inches square feet square yards square miles acres	AREA		cm ² m ² km ² ha	square centimeters square meters square kilometers hectares (10,000 m ²)	AREA	
		6.5	square centimeters			0.16	square inches
		0.09	square meters			1.2	square yards
		0.8	square meters			0.4	square miles
oz lb	ounces pounds short tons (2,000 lb)	MASS (weight)		g kg t	grams kilograms tonnes (1,000 kg)	MASS (weight)	
		28	grams			0.035	ounces
		0.45	kilograms			2.2	pounds
		0.9	tonnes			1.1	short tons
tsp Tbsp fl oz c pt qt gal ft ³ yd ³	teaspoons tablespoons fluid ounces cups pints quarts gallons cubic feet cubic yards	VOLUME		ml l m ³	milliliters liters cubic meters	VOLUME	
		5	milliliters			0.03	fluid ounces
		15	milliliters			2.1	pints
		30	milliliters			1.06	quarts
°F	Fahrenheit temperature	TEMPERATURE (exact)		°C	Celsius temperature	TEMPERATURE (exact)	
		5/9 (after subtracting 32)	Celsius temperature			9/5 (then add 32)	Fahrenheit temperature

*1 In = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price \$2.26, SD Catalog No. G17-101286.

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FORWORD

This report describes research performed for the U.S. Navy, Naval Civil Engineering Laboratory, Port Hueneme, under the Seismic Hazard Mitigation Program (YM33F60001) . The research was performed under the direction of Mr. John Ferritto. The author wishes to acknowledge the assistance of Dr. Tom Sabol, and UCLA graduate students Won-Kee Hong and George T. Zorapapel.

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CHAPTER 1

EXECUTIVE SUMMARY

Major structural engineering advances in the area of earthquake engineering have taken place with increased frequency since the 1971 San Fernando earthquake. Advances include (1) steel detailing for reinforced concrete buildings, (2) ductile design criteria for reinforced masonry shear walls and, (3) eccentric bracing of steel frames. One major advance has been the introduction of shock isolation concepts into building design and this advance is commonly referred to as Base Isolation. As with any new development, there are many proposed variations which more or less seek to satisfy the same objective. Some base isolation schemes are simple, well considered, and supported by both theoretical and dynamic experimental research. Others are not developed sufficiently to trust their use where U.S. Navy lives and operations are at risk.

This report presents the results of research conducted by the author and UCLA graduate students. It recommends the base isolation systems that are fully satisfactory for U.S. Navy buildings. It also addresses several key issues. Some of these issues are really "non-issues" and, at the present time, sufficient data exists to state that further research is not justified. Other issues do require further research to either greatly increase the quality of the final structure or to provide significant cost savings through uncertainty reduction.

The issues that are most often raised and that are really "non-issues" because they are sufficiently developed to provide acceptable structural engineering design confidence are:

- (1) Description of design earthquake ground motion at the building site.
- (2) Quality assurance testing of the base isolator components.
- (3) Aging or replacement, if needed, of the base isolator units.
- (4) Field construction and inspection.
- (5) Architectural, mechanical and electrical interface.

The issues that are presently of concern have been divided into two categories. One category includes those concerns that must be resolved prior to U.S. Navy adoption of base isolation as a widespread and available option for Navy buildings. These issues are:

- (1) Development of design criteria and commentary.
- (2) Development of a position on second party independent review and field inspection.

The second category consists of those issues, in order of importance, that should be addressed in order to increase cost effectiveness and confidence in base isolation design.

- (1) Research should be done to better understand the role vertical ground acceleration plays in base isolated building response.
- (2) The real need for a back-up system needs to be rationally addressed to quantify the cost/risk reduction.
- (3) The development of an expert system to facilitate its use by non-engineers such

as planners and architects.

- (4) The development of an expert system to enable the U.S. Navy structural engineering staff to understand, review, and manage a base isolation project.

Research reports of this type always foster more issues that are worthy of discussion, and thus, further research. In this respect, base isolation is no different from steel, concrete or masonry building design. Issues such as torsional response and plan or vertical irregularity are examples. Although, we see the merit of these issues, they are not critical to total design confidence and no attempt is made herein to develop such a research agenda.

This report benefits from results published in a 1986 NSF workshop on Base Isolation that was presented by the Applied Technology Council [1.1]. Every effort will be made herein to present these research findings to our broader audience. To this end, Chapters 2 and 3 are intended to provide a foundation and also an identification of the factors that must be considered in the building selection process. Chapters 4 and 5 present the most commonly cited quality and technical issues. The author seeks to be specific while at the same time not overly detailed so as to not go beyond the interests of most readers of this report. Appendix A contains documentation of an expert system and is intended to illustrate the concepts of expert systems and not to address issues 3 and 4 above.

If the development of a design criteria and commentary and a position on second party independent review and field inspection are resolved then base isolation will provide a viable economic alternative for many U.S. Navy buildings. Base isolation is a design advance whose time has arrived. It will be preferable for most hospitals and essential facilities.

CHAPTER 2

BASE ISOLATION

2.1 General

This section presents a brief introduction to the basic fundamentals of base isolation. The methods by which base isolation can be utilized are discussed as are the advantages and disadvantages of base isolation.

Figure 2.1 shows a schematic of the soil/isolator/building system. The isolator system provides an interface between the soil (foundation) and the building. Its function is to filter the input ground motion.

In the soil system of the base isolation model the relevant characteristics are the site soil properties, the potential for seismically induced settlement, instability, and liquefaction, and the nature of the ground motion input. In general, when base isolation is used, sites founded on moderately stiff to stiff soil will experience greater relative reductions to the input motion than will sites founded on softer, long period soils. Base isolation cannot mitigate the effects of settlement, slope instability, and liquefaction. The nature and magnitude of the input motion has an influence on the effectiveness of the base isolation system. The short period ground motion will be reduced to a greater degree by base isolation than will the long period ground motion.

Figure 2.1 The Soil/Isolator/Building/System

The base isolation system consists of the isolators, foundation, and the interface between the site and building utilities. The bearing or other isolation elements act to modify the input motion transmitted through the foundation from the earthquake. Because of the motion generated by the base isolation system, an interface between the site and building utilities is required. These building utilities might be thought of as including the water, power, telephone, communication, sewer, and gas lines as well as the stairs and the elevator systems. Knowledge of the magnitude of the anticipated motion is required to accommodate this displacement and still permit the effected systems to function.

2.2 Concepts and Design Philosophy

Structural systems for U.S. Navy buildings are designed to resist seismic ground motion without collapse. Their Design Criteria for "typical" buildings provide for strength and ductility and prescribe seismic design forces which assume that the building's structural system will experience inelastic deformation and structural damage for major earthquakes that can typically be expected to occur approximately once every 75 to 100 years. The structural system is designed with structural details that provide sufficient ductility for the system to survive a major earthquake without collapse. This type of earthquake corresponds to an approximately once in 500 year event.

The inelastic deformations of a building's lateral force resisting structural system for the design earthquake will produce significant permanent structural damage to the members of the structural system. The associated interstory displacements will induce nonstructural damage to building components such as partitions, curtain walls, and glazing. Additionally, the

dynamic response characteristics of the structural system will cause the earthquake ground motion to be amplified and the upper levels of the building will experience accelerations far greater in amplitude and duration than the earthquake ground motion imported to the structure at its base. These accelerations can cause significant nonstructural damage by inducing sliding or overturning of the contents of the structure, e.g., bookshelves, computers, electronic equipment, filing cabinets, and utilities.

2.3 Base Isolation System Response Characteristics

A base isolation structural system offers an alternative method of resisting seismic earthquake ground motions. The basic intent of most base isolation systems is to elongate the fundamental period of vibration of the total soil/isolator building system, and thus, reduce the earthquake induced accelerations at the floor levels. Figure 2.2 shows an idealized force response curve. By introducing flexibility at the base, for horizontal motion, the fundamental period of a building with a base isolation system is longer than the same structural system without isolators and is often beyond the range of periods that comprise strong earthquake ground motions. As a result, the magnitude of the accelerations transmitted to the superstructure above the base isolator system is greatly reduced. Stated differently, the base isolation system modifies the intense and damaging short period components of the ground motion and reduces the forces to the structural members. The system functions in much the same way as the shock absorber in a car. It filters out the short period or high frequency variations in the road surface.

Figure 2.2 Idealized Force Response Curve

When a base isolation system is used the building floor accelerations are reduced but there is increased displacement in the system as a whole. The relative displacement between the floors of the building will decrease because the seismic forces imparted decrease but the total lateral floor displacements are large because the isolator displacement is large. The base isolator controls the response such that the isolators experience a large response (6" to 12") but the floor to floor displacements are small. The base isolation system must be designed to accommodate this deformation.

2.4 Base Isolation Systems

A wide variety of base isolation systems has been proposed by researchers. References 2.1 and 2.2 discuss these alternative systems. Table 2.1, which will be discussed later, provides an evaluation of these systems as they relate to U.S. Navy buildings. Currently, in the author's opinion, the only feasible base isolation structural system for a U.S. Navy building is one where the superstructure is supported on a set of elastomeric bearings that are located between the base of the superstructure and the building's foundation system. A typical schematic layout for such a base isolation structural system is shown in Figure 2.3. The bearings are located just below the basement level. Other locations are possible, for example at the bottom or top of a column. However, these locations require extra special care in both design and construction. Therefore, the following discussion applies to the situation shown in Figure 2.3

When developing the configuration of the base isolation system several factors having an impact on both structural and nonstructural elements must be considered. First, a floor

Table 2.1
COMPARISON OF BASE ISOLATION
AND ENERGY DISSIPATION SYSTEMS

Trade Name	Restoring Force Capacity	Reserve Capacity and Structural Stability	Test Data	Resistance Aging	Tolerance to Imperfections
Lead-Rubber Bearings	Yes. Energy dissipation sensitive to vertical load.	Can incorporate back-up safety system. Low displacements ensure good reserve capacity of 2 to 3 for P-Δ effects.	Large number of bearing tests. Three series of shake table tests.	Good for natural rubber with 20, 30 and 40 year data available.	Good
High Damping Rubber Bearings	Yes	Can incorporate back-up safety system. Larger displacements lower the reserve capacity for P-Δ effects.	Small number of bearing tests. One series of shake table tests in progress.	Compound recently developed. No long term data available.	Good
Damped Brace Frames	Yes	No backup system. Lower displacements.	Component and shake table tests.	Reasonable but limited long term data available.	Fair
Rubber Bearings with Sliding Friction Plates	Partial but with permanent offset	Can incorporate back-up safety system. Reserve capacity ensured by sliding plate.	Many bearing tests. No shake table tests.	Neoprene can stiffen with age. OK if natural rubber is used.	Require very precise installation.
Earthquake Barrier System	Not in Current Design	Reserve capacity is function of limits on energy dissipating mechanisms.	No component tests. Shake table tests in progress.	Longevity of friction and ball bearing mechanism is unknown.	Require precise installation.

Table 2.1 (Continued)
COMPARISON OF BASE ISOLATION
AND ENERGY DISSIPATION SYSTEMS

Trade Name	Maturity and Number of Applications	Technical Support	Flexible Element	System Components Damping	Wind Restraint
Lead-Rubber Bearing	Installed in 2 Calif. bridges and over 30 structures in lower seismic zones.	DIS provides excellent support in U.S.	Elastomeric Bearings	Provided by Lead Plug Approx. 20 - 30% Viscous Damping.	Provided by Lead Plug
High Damping Rubber Bearings	Installed in 1 California building.	Limited availability in U.S. Malaysian Rubber Producers Assn. in England.	Elastomeric Bearings	Provided by rubber. Approx. 10% Viscous Damping.	Provided by Non-Linearity in Rubber
Damped Brace Frames	No current applications.	Available through private consultant arrangement and 3M.	None	Provided by friction or viscous device. Over 30% Viscous Damping.	Conventional System
Rubber Bearings with Sliding Friction Plate	No California installations. 4 Nuclear Plants in low seismic zones.	Available through U.S. agents for Electricite De France.	Elastomeric Bearings	Provided by friction of Sliding Plate and Rubber.	Sacrificial
Earthquake Barrier System	No Applications.	Available through M.S. Caspe.	Friction Surface or Ball Bearings	Provided by Control Beams, Friction Damper or other Mechanical Systems.	Provided by Control Beams or Friction Damper.

Figure 2.3 Typical Schematic Layout for Base Isolation

diaphragm at the base of the superstructure must exist to distribute the lateral forces from the superstructure to the bearings. Second, the bearings must be situated so as to be accessible for post-seismic inspection and, if necessary, replacement. Third, adequate horizontal clearance must be provided at the base level to accommodate, without serious expense, the large displacements in the base isolation bearings.

For the typical system shown in Figure 2.3, a separate perimeter retaining wall between the superstructure and the ground is required to provide the necessary seismic gap. Also, special consideration of such nonstructural elements as elevators and stairs as well as electrical, mechanical and utility lines is required. Flexible connections and special detailing are necessary to ensure continuity from the superstructure through the base isolator level to the exterior of the building.

The three types of bearings currently available for base isolation structural systems are:

- (1) Reinforced elastomeric bearings.
- (2) Reinforced elastomeric bearings with a lead plug damper.
- (3) Reinforced elastomeric bearings with a friction slip surface.

All types of base isolators are a lamination of rubber and steel shim plates. The two materials are bonded together through a vulcanization process thereby developing a continuity such that the bearing will act as an integral unit.

The first base isolation bearing type is shown in Figure 2.4. It is a sandwich constructed of alternating layers of rubber and steel plates. Each bearing has a relatively low horizontal shear stiffness with the actual stiffness characteristics dependent on the dimensions

Figure 2.4 Elastomeric Bearing

of the bearing and the material properties of the rubber. It is desirable that the vertical stiffness of the bearing be as large as practical. By its nature, the bearing's rubber wants to bulge or "barrel out" (the Poisson effect) under any appreciable compressive load. Rigid steel plates are placed between layers of rubber and through their bond to the rubber the bulging is restrained. This provides vertical stiffness far greater than the rubber acting alone.

The bearing shown in Figure 2.4 has some inherent damping. To further increase the damping of the bearing, and thus, provide additional energy dissipation for the base isolation system two additional types of bearings have been developed. One type has a cylindrical lead plug placed in a preformed hole in the center of the bearing, see Figure 2.5. This bearing is essentially the same as the first type but the lead plug now provides increased damping through hysteretic energy dissipation. The rigid steel plates force the lead plug to experience plastic shear deformations that produce the desired hysteresis. In order for the lead plug to function properly a tight fit is imperative. The lead plug also functions as a stiffener under low level loads. The lead, having high elastic stiffness, provides rigidity against moderate wind or seismic loads.

The third type of bearing, shown in Figure 2.6, utilizes sliding friction to enhance its damping properties. The bearing is not rigidly connected to the superstructure and the interface between the superstructure and the bearing forms a friction surface. Hysteresis, and hence effective damping, is produced by the friction force developed between two plates sliding against each other. A design consideration for the system is the selection of an interface coefficient of friction that will restrain the superstructure under wind loads and moderate seismic excitations. At extreme seismic loads both the deformation in the bearing and the

Figure 2.5 Elastomeric Bearing with Lead Plug

Figure 2.6 Elastomeric Bearing with Friction Slip Surface

frictional slip at the interface provide damping.

Other base isolation or other energy dissipation techniques to assist buildings in resisting earthquake shaking are currently receiving considerable structural engineering attention. Consider now some of the candidates this report has rejected.

Base isolation systems, as they are commonly understood in the United States, are primarily associated with elastomeric or rubber bearings being placed under each column of the building at the foundation level. However, a general definition of "base isolation" can also include other systems which include energy dissipation devices. These state-of-the-art devices provide earthquake energy dissipation through friction or viscous dampers typically added to the cross bracing members of the structure. Figure 2.7 shows an illustrative example of one type of energy dissipation system. This system uses a steel cross bracing energy dissipation system and was proposed by Pall. One advantage of the supplemental energy dissipation system of the type shown in Figure 2.7 is that it is typically added to the superstructure, and thus, no special consideration is required at the ground level such as building separation or flexible utility connections. Therefore, to call it base isolation is really stretching the definition. One usual drawback is that cross bracing may be architecturally restrictive. However, this system has undergone significant analytical scrutiny. A series of shaking table tests conducted by an internationally recognized and impartial expert in earthquake engineering at the University of British Columbia [2.3] has documented the system's reliability and effectiveness.

Figure 2.7 Building with Energy Dissipation Devices

Another system similar in concept utilizes viscoelastic damping devices in parallel with the cross braces. This device has been tested at 3M Laboratories and also by an impartial authority in earthquake engineering at the University of Michigan [2.2]. Good energy dissipation characteristics exist. These devices have been used successfully to damp out wind vibrations in the World Trade Center in New York City. Unfortunately, no shaking table tests have yet been conducted.

The Earthquake Barrier System has been developed by Mark S. Caspe [2.4]. The design is constantly being refined and consists of sliding teflon coated plates coupled with energy dissipating devices. The friction between the sliding surfaces is selected such that they slip after the earthquake force reaches a certain threshold level. However, at the present time and when compared with the other systems, we find little well documented analytical or experimental research has been conducted by impartial researchers.

The following three major factors were used in the evaluation process:

(1) **Technical**

Professional confidence must exist that the system selected will meet the technical requirements necessary to ensure proper function, predictability of behavior, and demonstrated long-term reliability.

(2) **Maturity**

There must be significant maturity. Systems range from unused, untested concepts to systems that have been extensively tested and used for a significant

number of structures. A mature data base of good quality experimental and analytical studies is essential.

(3) **Technical/Professional Support**

The construction industry relies on a network of engineers providing professional support to both promote their individual product and ensure its correct usage so as to enhance quality control. Therefore, the availability of technical/professional support for the system selected is very important.

Table 2.1 provides a summary of the systems reviewed in this research and comments on the evaluation factors.

A base isolation system which is of special note has recently been tested at UC Berkeley. Figures 2.8 through 2.10 show schematics of the system called the FPS Seismic Isolation System. The FPS (Friction Pendulum System) and the FPS connections are patented products developed by Earthquake Protection Systems, Inc., San Francisco, California that dissipate energy through pendulum action. Indications are that it is a promising system.

CENTERED POSITION

DISPLACED POSITION

Figure 2.8 FPS Connection Operation

PENDULUM MOTION

SLIDING PENDULUM MOTION

PERIOD $T = 2\pi\sqrt{r/g}$

BASIC PRINCIPLES

Figure 2.9 Friction Pendulum Concept

Figure 2.10 Section View of the FPS Connection

CHAPTER 3

BUILDING SELECTION PROCESS

3.1 General

Base isolation can offer many advantages over a conventionally designed building. The first step in the design process must be to see if such a system is a rational alternative for the specific building under consideration. A quantitative feasibility study can be performed at the very early stages of the design process that makes it possible to quickly determine whether or not base isolation is a feasible option. If such a study shows that a base isolation system is feasible, the 35% design phase should investigate relative costs and benefits between the isolated and non-isolated systems.

A feasibility study on a specific building must consider the following:

- (1) The potential need for a base isolation structural system.
- (2) The suitability of the site and the building.
- (3) The cost effectiveness from a construction and a life cycle perspective.

3.2 Excellent Potential Applications of Base Isolation

Base isolation is most applicable for buildings located in areas of high seismicity where moderate earthquake ground motions (e.g. peak accelerations 0.2g or higher) are expected to occur every 15 to 20 years and where high design earthquake accelerations (e.g.

0.4g or higher) are expected to occur during the design life of the building. For conventional fixed base buildings, the U.S. Navy criteria assumes that minor structural damage will occur during a moderate earthquake. Such damage may not be deemed acceptable.

The intended use of the building is very important. Often, increased safety is required for mission essential buildings. For example, a U.S. Navy hospital is required to be operational both during and after an earthquake. Also, in many buildings, floor accelerations must be controlled to limit the overturning of computers or sensitive contents. Often, intrastory displacements must be limited to small amplitudes to minimize damage to nonstructural elements. These two factors, lowering accelerations and reducing strong drifts, work against each other in conventional structural systems.

3.3 Suitability of the Site

Short buildings having short fundamental periods of vibration without base isolation benefit the most, on a comparative basis, from base isolation. As a basic guideline, base isolation is best suited to relatively stiff, squat, low-to-medium-rise buildings in the range of 2 to 5 stories. These buildings have fixed-base fundamental periods of vibration that are typically less than 0.7-1.0 second. Because the isolators lengthen the period to 2.5 to 3.0 seconds this period shift typically reduces floor accelerations and forces by 60 to 80 percent.

The site soil conditions and geology are very important considerations in evaluating the suitability of a building for base isolation. In general, sites having moderately stiff to stiff subsurface soil are best suited for a base isolation system. The ground motion input applied at the structure's base is affected by the underlying solid deposit. Soft soils tend to filter out

the high frequency motion of the seismic waves as well as provide some attenuation. As a result, soft soil provides a form of isolation itself. Therefore, we do not want to tune the base isolation system to the soil system. Sites with a site period greater than 1.0 second are not desirable. Stiff soils, on the other hand, transmit the high frequency motion. Therefore, by comparison, a base isolation system will be very effective for a building underlain by a stiff soil.

Finally, the candidate site must have adequate space to accommodate the predicted maximum horizontal displacements at the base. A base isolation system transmits lower forces but deflections across the isolators require clearances on the order of at least 6 to 12 inches.

3.4 Initial Construction Costs and Life Cycle Costs

The cost effectiveness of a base isolation system must consider the initial cost of the isolation system and the potential savings due to reduced damage to the building's structural system and contents over its design life.

The initial costs of implementing a base isolation system into a building design include the cost of the bearings and the associated structural and nonstructural factors required to accommodate the bearing system. Initial costs depend on the building and the extent of early planning. The costs usually include a rigid concrete slab above the isolators, bearings, a back-up system and the cost of providing a sufficient seismic gap around the building. Non-structural costs would correspond to the special detailing required for stairs, elevators and mechanical shafts at the isolator level. In addition, one needs special flexible utility

connections.

The initial benefits are related to material quantity savings in the lateral force resisting system, reduction or elimination of sensitive equipment tie down costs, and reduced foundation costs. However, the most substantial benefits of base isolation are associated with life cycle costs. Structural damage arising from inelastic deformation in the superstructure can be expected to occur during the building's design life. However, nonstructural damage to a building's contents is related to floor accelerations which are significantly reduced with base isolation. Furthermore, base isolation decreases interstory drifts and associated damage to such nonstructural elements as cladding, glass and partitions without stiffening the building and increasing floor acceleration.

The initial construction costs for a conventional building are comprised of the following items.

- (1) Structural costs such as structural steel, concrete or masonry.
- (2) Non-structural costs such cladding, windows, partitions and fixtures.
- (3) Machine, electrical, plumbing and vertical transportation system costs.
- (4) General conditions (insurance, permits, inspections).

A base isolated building has additional costs associated with the cost of base isolators, or energy dissipation devices, the special detailing required at the isolation level to provide structural separation, and flexible utility connections.

Earthquake induced financial loss to a building can take many forms. The most direct loss is due to structural damage to the building's lateral force resisting system. Other forms of loss that can often equal or exceed structural damage are: (1) loss due to damage to non-structural contents such as computers and other equipment; (2) loss due to operational interruption; and (3) loss due to potential liability to occupants. Base isolation will have a positive impact on these costs.

Typical earthquake induced financial loss for commercial buildings is not relevant to or easily quantifiable for U.S. Navy buildings. For example, there are no insurance or financing costs for government buildings although there may be costs associated with alternative leasing arrangements used in lieu of constructing new facilities. The loss of "business" due to interruptions is not easily quantifiable because no direct revenue generating business is conducted such as is the case for commercial buildings. The loss of operational capabilities is certainly a clear consequence of significant earthquake damage. However, the costs associated with these problems cannot be clearly quantified.

Because of the uncertainties associated with predicting the maximum earthquake ground motion and the ensuing damage, an economic and performance evaluation of a base isolated building design versus a conventional building design must be addressed in a probabilistic manner. The potential methodology for this review is summarized in Figure 3.1. The components of this methodology are: (a) a probabilistic description of the earthquake damage, both structural and nonstructural, for a conventional and a base isolated building, (b) a probabilistic description of the anticipated earthquake ground motion, (c) an estimation of the initial cost of the building, (d) an estimation of the average annual earthquake loss based on (a), (b)

Figure 3.1 Schematic of Evaluation Methodology

and (c) above, (e) an estimation of the present value of future earthquake loss and, (f) the present value of total costs. The present value of the total costs so obtained for different structural systems can then provide the decision makers enough information to evaluate each particular structural system.

The damage matrix approach is appropriate for the feasibility evaluation of a base isolation system. This method of damage estimation was presented in a rudimentary form just after the 1971 San Fernando Valley Earthquake in a Massachusetts Institute of Technology study funded by the National Science Foundation. This approach has been extended and improved over the years using decision theory techniques plus our knowledge of structural engineering.

3.5 Role of Expert Systems

A base isolation expert system is very desirable for effective technology transfer in the early planning stages of essential buildings. An expert system is a computer based system that can be constructed to play the role of human experts. These systems use specific questioning that enables users who are not structural engineers to describe their problems or goals. Eventually, solutions are provided through a process of inference.

A base isolation expert system will communicate with the user by providing information or definitions for the user who is not familiar with base isolation. The system explains why a question is relevant and it can explain or justify any solution or advice.

Six main components comprise a base isolation expert system. The components are: the input, output, user interface, inference engine, parser, and knowledge base. These components will now be briefly discussed.

- (1) **The input** is provided by the user. It may be answers to the questions asked by the expert system or commands to the expert system.
- (2) **The output** of the expert system takes the form of questions to the user in order to: obtain information, definitions or explanations when the user is confused; respond to commands from the user; or to provide useful information for the system user.
- (3) **The user interface** transforms the contents of the base isolation/structural engineering knowledge base into a form that the non-structural engineer can comprehend. The most typical product of the user interface is the "menu" which prompts the user to obtain information for each variable contained in the knowledge base.
- (4) **The inference engine** combines the user's answers to the questions posed by the user interface with the rules in the knowledge base in order to produce further questioning and, finally, the advice that the user needs. The inference engine also provides explanations that pertain to the advice given or the conclusions drawn.
- (5) **The parser** is the interface between the author of the knowledge base and the

system. The author writes the components of the knowledge base in English like the rules used by a text editor. Then, the parser checks for mistakes and displays warnings to the author. If there are no mistakes, the parser translates the text into the format necessary for the knowledge base to be processed by the computer.

- (6) **The knowledge base is developed using published technical literature and existing U.S. Navy design information.**

The users of a base isolation expert system will have a very limited knowledge of the problem. It is the role of the expert system to extract from them all the information concerning the environmental conditions of the problem and to explain the meaning or the purpose of the questions. There are 6 steps to the development of a meaningful base isolated expert system. These steps are:

STEP 1. Identify Criteria: The initial step is to clearly define the criteria used to evaluate the building for base isolation.

STEP 2. Select an Expert System "Shell": After identifying all the criteria, the size of the expert system must be estimated.

STEP 3. Identify and Write the Attributes: The attributes that receive their values from the responses of the user are called "input attributes". There are also "inferred attributes". Their values are computed by the expert system based on the rules in the knowledge base.

STEP 4. Write the Logical Rules: The rules show the expert system how to take the building description and derive the value of a certain inferred attribute from the values of the lower hierarchy's attributes. The complete set of rules is written after a flowchart of the problem is made that describes each move along the flowchart.

STEP 5. Define and Write the Actions: The base isolation expert system must be given directions to follow in order to reach the final conclusion. These directions are given in the form of "actions".

STEP 6. Test the Expert System: Like any other computer program, the base isolation expert system must be tested for different combinations of the values of its attributes in order to detect possible errors of judgment as well as to determine the program's usefulness.

For illustrative purposes, the draft entitled "Guidelines for Selection and Use of Base Isolation Systems" has been used to develop an expert system. The system was built to enable a nonstructural user (e.g. a design civil engineer) to perform a preliminary feasibility study for a building located in a certain seismic zone. Appendix A contains an illustrative example of a base isolation expert system.

CHAPTER 4

QUALITY CONTROL ISSUES

4.1 General

Quality control is an issue whose critical importance is obvious. Analytical model assumptions are only as good as the experimental data and professional insight on which they are based and the care with which the design is constructed. Quality control, for the purpose of this report, has been divided into three parts. First, it is essential to have good design criteria. Second, a quality insurance program for the isolators must exist. Third, field construction quality control must be addressed.

4.2 Design Criteria

The development of design criteria for base isolated buildings is not only possible but it can be done consistent with existing U.S. Navy Essential Building Criteria (TMS-810). The author took a very active role as co-chairman of a Structural Engineers Association of California (SEAOC) professional committee that developed base isolation design criteria for California hospitals. It is clear that many areas of common agreement exist with the U.S. Navy Essential Building Design Criteria such as in quality assurance testing, the importance of torsion and near field displacements, etc. The SEAOC hospital criteria is contained in Appendix B.

Base isolated structures differ in one major structural way from conventional buildings as they relate to EQ-I and EQ-II U.S. Navy criteria. The EQ-I performance criteria (typically a 50% chance of being exceeded in 50 years) for conventional mission essential buildings requires that the structural system remains essentially elastic with no steel member or reinforcing steel experiencing yielding. It is clear that base isolated buildings can have the same performance criteria for the structures above the isolators. However, the isolators themselves will experience inelastic deformation at earthquake excitement well below the EQ-I excitation. Therefore, the analytical model used to calculate the EQ-I response of a base isolated building must be inelastic whereas a model typically used for a conventional building is elastic. This difference creates a problem for structural engineers with little or no real understanding of structural dynamics. However, these individuals should not be designing base isolated buildings. The current problem is that a discussion does not exist as to an acceptable modeling approach followed by several illustrative examples. Technical manual TM 5-810 for the seismic design of essential buildings had to address just such a technical issue. It accomplished this beautifully in both its discussion and examples for Post-Yield Analysis Provisions (Section 4-4, pg 4-7). The same approach can and should be used for base isolation.

The EQ-II performance criteria (typically a 10% chance of being exceeded in 50 years) for a base isolated mission essential building should be the same as for a conventional mission essential building. No reason exists to modify the concept that a post-yielded analysis is necessary and that inelastic demands should be less than pre-established values. What is not appropriate is to use the capacity spectrum method at the exclusion of multiple strong motion time history analyses. Inelastic strong motion time history computer codes for building design

are today variable and affordable (e.g. DRAIN-2D, NOODY).

A U.S. Navy design criteria that is consistent with the philosophy of the U.S. Navy Mission Essential Building Criteria and the SEAOC hospital criteria is needed for base isolated buildings. The existence of such a criteria will improve the quality of design. This improvement will be most evident in less designer and structural review confusion.

4.3 Quality Assurance Testing

A detailed study of the SEAOC hospital criteria in Appendix B will clearly indicate that quality assurance testing was a major SEAOC concern. It was also a task that could be addressed by committee and it has, in the author's opinion, been solved by SEAOC. What is needed is a review and then some possible modification to address any special needs of the U.S. Navy. For illustration, some issues that have been addressed are:

- (1) The nature of cyclic time history loading on isolation.
- (2) The minimum acceptable base isolation displacement capacity.
- (3) The number of isolators tested.

Performance specifications have been developed by SEAOC, and thus, any propriety issues do not exist.

4.4 Field Construction Quality Control

Existing U.S. Navy procedures are essentially acceptable for base isolated buildings. Experience indicates that base isolated designs can be less complicated to construct and easier to inspect than conventional buildings. By virtue of the reduced seismic loads we can expect to have less ductile moment connections and less reinforcing steel congesting small areas.

One item of special note is that the structural engineer of record should be contracted to provide regular job site visits for the purpose of observation. This is currently done with some conventional building projects but it should be required on all base isolated building projects. Placement of the isolators is critical and the load paths for the building must be inspected.

EQ-1 : M=8+, 79 KILOMETERS
 EQ-2 : 50 % IN 50 YEARS (72 YEARS)
 EQ-3 : 10 % IN 50 YEARS (475 YEARS)
 EQ-4 : M=7.0, 1 KILOMETER (15000 YEARS)

—○—
 —△—
 - - -◇- - -
 —+—

Figure 5.1 Response Spectra

CHAPTER 5

TECHNICAL ISSUES

5.1 General

This chapter provides insight into several technical issues. The approach taken is to identify those technical issues that should be discussed because of concerns expressed by others or a desire to highlight the need for a more in-depth study of particular issues.

5.2 Local Fault Ground Motion Excitation

The design criteria for base isolation must require the development of site specific ground motion histories and response spectra. It is clear from Figure 5.1 that a design spectrum can be exceeded by smaller local fault generated earthquakes. With a geotechnical study of the site the topic of local faults and near field ground displacements will have been addressed and the issue becomes a non-issue.

A real concern and perhaps a secondary issue is that the A/E selection process must provide a detailed review of the geotechnical consultant. This is not typically done when the leader of the U.S. Navy interview team is an architect rather than a structural engineer. However, this is an administrative issue which can be resolved by others.

A pool of qualified geotechnical consultants exists who can rationally estimate field as well as far field design earthquake ground motions. Therefore, the selection of a qualified consultant is not an issue.

5.3 Dynamic Analysis of Buildings

Prior to 1970 most structural engineers were not educated in even the basics of structural dynamics. Today, most structural engineers know at least the basics and understand response spectra analyses. Therefore, it's reasonable to take the position that for a "high tech" system such as base isolation structural dynamics must be used.

The SEAOC hospital criteria requires a dynamic analysis. The U.S. Navy should have a similar requirement.

The U.S. Navy Essential Buildings Criteria has two levels of performance which are addressed as EQ-I and EQ-II design. It is the position of this research that:

- (1) The EQ-I earthquake motion be prescribed in terms of either a response spectra or a set of time histories. This choice can be made by the structural engineer of record. The resulting analysis can be a dynamic analysis with an equivalent linear elastic spring representing the isolator.
- (2) The EQ-II earthquake motion shall be a set of time histories. A step-by-step dynamic analysis is required.

5.4 Torsion

Torsion is not really an issue but it is a topic raised over and over again. The current design criteria for ordinary U.S. Navy buildings does not require a three-dimensional dynamic analysis. However, a base isolated building should have such an analysis. If this is

done, torsion is accounted for in the design.

Accidental torsion, however, is a topic requiring comment. No technical basis exists for the current criteria for accidental eccentricity. If no criteria exists the data available to technically support the 5% number does not exist. However, like many items in the design criteria, it is a requirement that has served its purpose. If the issue is how do we model structural system torsion then it is not an issue. If the issue is how do we model construction errors, the three dimensional twisting of the ground, etc, then research needs to be funded by the U.S. Navy on this topic.

5.5 Back-Up System

The research conducted as part of this project involved talking with many, many structural engineers about back-up systems. Items discussed were the need for the system and the type of system to be used. This is a real issue and a very expensive one.

A back-up system can cost, at minimum, several hundred thousand dollars. It is intended to provide a secondary line of safety if the isolation components fail. The intent is good but the benefit and cost/benefit is unclear. The arguments, at this time, are emotional.

Research does not exist to clearly document the technical benefits of a base isolation back-up system. If the isolators do fail and if the back-up system comes into action then the theory of structural dynamics must be used to quantify the resultant response, not speculation.

Such studies have not been done to the author's satisfaction, and therefore, this issue is

viewed as a top priority that can be technically solved with a reasonable amount of research effort.

5.6 Vertical Ground Motion

This issue transcends base isolated buildings and really applies to all buildings. Sufficient data exists to address the impact of vertical ground motion on the dynamic response of a base isolated building on a building by building basis. However, the level of effort is large and the cost is probably best not handled on a project by project basis.

A detailed analysis of one or more case study projects is recommended using site specific time histories of ground motion input. This type of a research effort would provide insight and probably an answer to the question - Is it time to let our structural dynamic models include vertical ground motion input.

5.7 Soil Uncertainty Input

This is a very interesting issue because early returns are that the inclusion of isolators between the soil and the structure may significantly reduce the uncertainty in the soil/building system. The isolators act to filter out some of the uncertainty associated with soil modeling and parameter values.

Models of the soil/isolator/building system have been done using elastic half space models and, as part of this research, simple spring mass models. The results are inconclusive because the scope of study has been very limited. Finite element models could similarly be developed for the system. Research should be carried out on this topic but in order to

develop reliable design guidelines considerable effort will be required. Therefore, the research is probably best accomplished over a two to three year span of time. The research must include the nonlinear response characteristics of the soil.

CHAPTER 6

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APPENDIX A

ILLUSTRATIVE BASE ISOLATION EXPERT SYSTEM

A.1 What is an Expert System?

Expert systems are computer based systems that can be constructed to play the role of human experts. These systems use specific questioning that enables users to describe their problems or goals completely. The user input is processed according to the rules provided within the knowledge basis of the expert system. For each specific problem further questioning is directed. Eventually, solutions or advice are provided through a process of inference.

Expert systems seek to communicate clearly with the user. They are able to provide additional information or definitions in cases where the user is not familiar with the items related to the content of a particular question. The systems can also explain why a particular question is relevant in case the user is confused. Eventually, the systems can explain or justify any solution or advice.

Expert systems can solve problems in any possible domain, ranging from business to education to government. They can find specific information and give advice on how to use this information. They can be utilized to provide the expertise of an employee who is absent, leaving or retiring. They can save the time of a highly qualified employee, transferring complex problems to a regular employee who usually performs routine work. The ability of expert systems to explain their conclusions is valuable for customer relations as well as for staff training.

A more sophisticated and rewarding use of expert systems is related to their feedback. Suppose that criteria and the rules for practical application have been set for a new product or activity. Instead of costly and sometimes painful experimental application, a simulation process can be used to explore the possible consequences of the new criteria. The most appropriate tool to perform the simulation would be expert system whose knowledge base contains the new criteria and the rules. This expert system can be given to several users who will try different possible situations. Eventually, the solutions and advice provided by the expert system will be reviewed, possibly along with their justification, from the viewpoint of feasibility and possible implications. If all the results look reasonable, the set of rules can be validated. If unacceptable consequences are found, the justification logical path will point to rules for their consideration.

Three categories of individuals must be involved in creating an expert system:

- (1) The domain expert is the source of information for the knowledge base. Sometimes, his, or her expertise is already organized in books, articles or other types of documents.
- (2) The knowledge base author organizes the information from the domain expert and writes the "English-like" knowledge base.
- (3) The knowledge engineer is a computer scientist who writes a program that generates the expert system starting from the "English-like" knowledge base written by the knowledge base author.

The users of the expert system may have only a very limited knowledge of the problem or the goal to be reached. It is the role of the expert system to extract from them all the information concerning the environmental conditions of the problem and to explain the meaning or the purpose of the questions.

Presently, there are several programs on the market that are able to generate expert systems. They differ mostly by the size of the knowledge base. There are several measures of the capacity of a knowledge base. In our opinion, the most significant feature is the number of rules the knowledge base can support because this is the limitation that is most often encountered in practical applications. Programs for illustrative or educational purposes usually have twenty or thirty rules as their capacity limit. However, programs that can support a knowledge base that contains up to 2000 rules and yet can be run on microcomputers are already on the market. structures. The "field of experience" becomes an attribute.

A.2 Illustrative Sample

For illustrative purposes, the draft entitled "Guidelines for Selection and Use of Base Isolation Systems" has been used as the expert document. An expert system has been built in order to enable a common user (ex. a civil engineer) to explore the possibility of using the base isolation solution for a building located in a certain seismic zone.

STEP 1. Identification of Criteria

1. Seismic zone criterion: base isolation solution can be considered for seismic zones 3 and 4. In case of special structures the solution can be considered also for zone 2.
2. Soil type criterion: the structure must be located on firm soil or rock.
3. Variability in soil conditions: variations in conditions across the site are undesirable.
4. Liquefaction potential criterion: the areas of potential liquefaction should be avoided.
5. Near field criterion: base isolation should not be used for structures located within 2 miles of an active fault (probable considering the danger of the long period pulse due to the Doppler effect).

6. Configuration criterion: for existing structures presenting irregular in plan configurations like "H", "L", "E", and others, the base isolation rehabilitation solution is not allowed. For new structures, the solution becomes possible only when providing appropriately located seismic joints. This additional requirement must be considered in the cost analysis.
7. Natural period criterion: an upper limit of 0.7 seconds (for the structure considered without isolators) will be established.
8. Cost of the base isolation solution: this criterion has been added to the initial set in order to provide a final trial step before reaching the conclusion. For illustrative purposes, three ranges of cost per square foot have been considered so far. If the base isolation solution has "passed" the preceding criteria, this last comparison will complete the screening process. If the cost is in the "ball park" a more detailed analysis is recommended; for lower or higher costs, the recommendation of adopting or discarding the base isolation solution is given.

STEP 2. Selection of an Expert System Shell

The program used for this example is MICRO PS. It is available from Ashton-Tate as part of an expert system book/computer program package entitled Building Your First Expert System by Nagy, Gault and Nagy.

STEP 3. Identify and Write the Attributes

Attribute	Values
1. Seismic zone	1 2 3 4
2. Special structure	yes no
3. Soil type	soft firm rock
4. Liquefaction potential	low moderate high
5. Variable soil conditions across the site	yes no
6. Smallest distance to an active fault	less than 2 miles more than two miles
7. Building status	existing new
8. Shape in plan	square/rectangular/circular other
9. Structural system	steel frame reinforced concrete frame shear wall system

10. Building height range	less than 35 feet
35 feet to 67 feet
67 feet to 100 feet
more than 100 feet
11. Building depth range	less than 50 feet
more than 50 feet
12. Cost of base isolation solution	less than 15 \$/sf.
15 \$/sf. - 30 \$/sf.
more than 30 \$/sf.

STEP 4. Write the Logical Rules (See the following pages for the rules.)

NOTE: The data file for this example of the MICRO PS computer program is available from the author upon presentation of the purchase receipt for MICRO PS.

attributes:

seismic zone

(int): from 1 to 4.

special structure

(smlt): yes,no.

soil type

(smlt): soft soil,firm soil,rock.

liquefaction potential

(smlt): low,moderate,high.

variable soil conditions across the site

(smlt): yes,no.

smallest distance to an active fault

(smlt): less than two miles,more than two miles.

building status

(smlt): existing,new.

shape in plan

(smlt): square or rectangular or circular,other.

structural system

(smlt): steel frame,reinforced concrete frame,shear wall system.

building height range

(smlt): less than thirty five feet,between thirty five and sixty seven feet,between sixty seven and hundred feet, more than hundred feet.

building depth range

(smlt):less than fifty feet,more than fifty feet.

cost of base isolation solution

(smlt): less than fifteen dollars per square foot,between fifteen and thirty dollars

per square foot,more than thirty dollars per square foot.

path1

(smlt):ok,ng.

path2

(smlt):ok,ng.

path3

(smlt):ok,ng.

decision

(smlt): the base isolation solution is recommended,more detailed analysis is requested,

the base isolation solution is not recommended

%

rules:

r1

if seismic zone = 1,

/soil type = soft soil,

/liquefaction potential = high,

/variable soil conditions across the site = yes,

/smallest distance to an active fault = less than two miles,

then path1 = ng.

r2 if seismic zone = 2,
 &special structure = no,
 then path1 = ng.

r3 if seismic zone gt 2
 &soil type#soft soil,
 &liquefaction potential#high,
 &variable soil conditions across the site = no,
 &smallest distance tu an active fault = more thann two
 miles,
 then path1 = ok.

r4 if seismic zone = 2
 &special structure = yes,
 &soil type#soft soil,
 &liquefaction potential#high,
 &variable soil conditions across the site = no,
 &smallest distance tu an active fault = more thann two
 miles,
 then path1 = ok.

r5 if path1 = ok,
 &building status = existing,
 &shape in plan = other,
 then path2 = ng.

r6 if path1 = ok,
 &building status = new,
 then path2 = ok.

r7 if path1 = ok,
 &building status = existing,
 &shape in plan = square or rectangular or circular,
 then path2 = ok.

r8 if path2 = ok,
 &structural system = steel frame,
 &building height range#less thann thirty five feet,
 then path3 = ng.

r9 if path2 = ok,
 &structural system = reinforced concrete frame,
 &building height range = between sixty seven and
 hundred feet,
 /building height range = more thann hundred feet,
 then path3 = ng.

r10 if path2 = ok,
 &structural system = shear wall system,
 &building height range = more thann hundred feet,
 then path3 = ng.

r11 if path2 = ok,
 &structural system = shear wall system,
 &building height range = between sixty seven and
 hundred feet,
 &building depth range = less thann fifty feet,
 then path3 = ng.

r12 if path2 = ok,
 &structural system = shear wall system,
 &building height range = between thirty five and sixty
 seven feet,
 /building height range = less thann thirty five feet,
 then path3 = ok.

r13 if path2 = ok,
 &structural system = shear wall system,
 &building height range = between sixty seven and
 hundred feet,
 &building depth range = more thann fifty feet,
 then path3 = ok.

r14 if path2 = ok,
 &structural system = reinforced concrete frame,
 &building height range = between thirty five and sixty
 seven feet,
 /building height range = less thann thirty five feet,
 then path3 = ok.

r15 if path2 = ok,
 &structural system = steel frame,
 &building height range = less thann thirty five feet,
 then path3 = ok.

r16 if path3 = ok,
 &cost of base isolation solution = more thann thirty
 dollars per square foot,
 then decision = the base isolation solution is not recommended.

r17 if path3 = ok,
 &cost of base isolation solution = between fifteen and
 thirty dollars per square foot,
 then decision = more detailed analysis is requested.

r18 if path3 = ok,
 &cost of base isolation solution = less thann fifteen
 dollars per square foot,
 then decision = the base isolation solution is recommended

```

%
actions:
obtain path1.
if path1 = ng,
then message "the base isolation solution is not recommended"
endif.
if path1 = ng, then pause
endif.
obtain path2.
if path2 = ng,
then message "the base isolation solution is not recommended"
endif.
if path2 = ng,
then pause
endif.
obtain path3.
if path3 = ng,
then message "the base isolation solution is not recommended"
endif.
if path3 = ng,
then pause
endif.
if shape in plan = other,
then message "please estimate the cost taking into account seismic
joints for "
"the design solution"
endif.
obtain decision.
message "after the screening procedure I consider that".
display value(decision).
pause
%
```

APPENDIX B

SEAOC DESIGN CRITERIA FOR HOSPITALS

DRAFT

SEISMIC ISOLATION DESIGN GUIDELINES FOR HOSPITALS

1. INTRODUCTION

This document provides guidelines for the design of hospital structures which use seismic (base) isolation systems as an alternative structural system permitted by Section 2-2312(a) of Title 24, California Administrative Code (CAC).

Complete project description and design criteria should be provided to the Office of Statewide Health, Planning and Development (OSHDP) for review by the Office of the State Architect (OSA) prior to submission of construction documents for checking.

2. DETERMINATION OF DISPLACEMENT AND FORCES

Seismic isolation design forces and displacements should be obtained from dynamic analyses using seismic input corresponding to the maximum probable and maximum credible events.

2.1 Analysis Procedures - General

Analytical models of the building should consider the three-dimensional aspects of the structural system and should accurately represent both the deformational characteristics of the isolation system and the deformational characteristics of the superstructure. Analysis of lateral response should be performed in both orthogonal directions of the building. The isolation system and the superstructure may be analyzed as being linear as long as response is nearly-elastic. Suggested

criteria for determining nearly-elastic structural behavior are given in Appendix A.

2.2 Analysis Procedures - Isolation System

The deformational characteristics of the isolation system should be explicitly modeled and substantiated by test, as specified in Section 6. The deformational characteristics of the isolation system should account for the spatial distribution and variation in isolator stiffness.

It is not necessary for the building model used for analysis of the isolation system to be a member-by-member representation of the superstructure. Provided the essential dynamic characteristics are accurately represented, the building model may utilize a "stick" model representation of the superstructure.

If the deformational characteristics of the isolation system are not stable or vary appreciably with the nature of the load (e.g., rate, amplitude, or time dependent), then design displacements should be based on the deformational characteristics of the isolation system which give the largest possible deflection (e.g., minimum effective stiffness of isolators), and the design forces should be based on the deformational characteristics of the isolation system which give the largest possible force (e.g., maximum effective stiffness of isolators).

If the deformational characteristics of the isolation system are not stable or vary appreciably with the nature of the load (e.g., rate, amplitude, or time dependent), then the hysteretic behavior (i.e., damping) used to determine design displacements and forces should be based on the deformational characteristics of the isolation system which represent the minimum amount of energy dissipated during cyclic response.

2.3 Horizontal Torsional Moments

Provisions should be made for the increase in response resulting from horizontal torsion due to an eccentricity between the center of mass and the center of rigidity. Accidental torsion should be accounted for in analysis by placing floor mass a distance from the calculated location equal to $\pm 5\%$ of the building dimension perpendicular to the direction under consideration.

2.4 Time History Analysis

Time history is required for buildings which have either a nonlinear isolation system or a nonlinear superstructure. If time history analysis is performed, at least three appropriate time histories, as defined in Section 3, should be used for each level of seismic input. Explicit modeling of deformational characteristics should consider both minimum and maximum effective stiffness of the isolation system. This may require two separate analyses for each time history used. The maximum response of all required analyses should be used for design.

If minimum and maximum effective stiffness of the isolation system do not differ by more than 10%, then only one analysis per time history would be sufficient using deformational characteristics which represent an average value of effective stiffness.

2.5 Response Spectrum Analysis

If a response spectrum analysis is used (i.e., for linear systems), two separate analyses should be performed for each level of seismic input, one using maximum effective stiffness and the other using minimum effective stiffness of the isolation system. In both cases, the minimum effective damping value at the design displacement should be used. Guidance for evaluation of minimum and maximum effective stiffness and minimum effective damping are given in Appendix B. The maximum response of all required analyses should be used for design.

If minimum and maximum effective stiffness of the isolation system do not differ by more than 10%, then only one response spectrum analysis is required using an average value of effective stiffness.

3. SEISMIC INPUT

The site-specific ground motion should be based on appropriate geologic, tectonic, seismic, and foundation material information. The two levels of ground motion which should be considered are as defined in Section 2-2312(d) 1A of Part 2, Title 24, CAC.

If time history analysis is used, the input time histories should be selected from different recorded events and based on similarity to source magnitude, foundation material and tectonic conditions. They should be scaled such that their 5%-damped response spectrum essentially envelopes the site-specific spectrum and does not fall below the site-specific spectrum by more than 10% at any period. Time histories developed for sites within 10 km of a major active fault must incorporate near-fault phenomena. Duration of time histories should be consistent with the magnitude and source characteristics of an event.

4. LATERAL DESIGN FORCE AND DESIGN DISPLACEMENT

4.1 Isolation System

Displacements and forces used for design of the isolation system and connections to structural elements immediately above and below the isolation system should be 1.5 times the maximum displacements and forces determined from dynamic analysis using the maximum probable earthquake, or 1.1 times the maximum displacements and forces determined from dynamic analysis using the maximum credible earthquake, whichever is greater.

4.2 Structural Elements Below the Isolation Interface

The strength of the elements below the isolation interface should not be less than that required to sustain the lateral forces, as determined by dynamic analysis for the maximum credible earthquake,

together with either a factored load of 1.2 times the dead load plus 0.5 times the reduced live load, or 0.8 times the dead load, whichever is greater.

4.3 Structural Elements Above the Isolation Interface

4.3.1 Maximum Probable Earthquake

Using the dynamic analysis for the maximum probable earthquake, structural members should be proportioned on the basis of their maximum capacity, otherwise known as yield or limit strength design in accordance with Section 2-2312(d) 1D, or on the basis of the deflection or drift limitations set forth in Section 2-2307, whichever governs.

The base shear forces resulting from this analysis should be compared with:

- a. the base shear corresponding to the design wind load and
- b. the yield level of a softening system, the ultimate capacity of a sacrificial wind-restraint system, and the static friction level of a sliding system.

If the dynamic analysis base shear force for the superstructure is less than these limits, the design forces should be increased proportionately so that the greater of the limits is satisfied.

4.3.2 Maximum Credible Earthquake

Using the dynamic analysis for the maximum credible earthquake, ductile moment resisting frame structures should be designed so that Section 2-2312(d) 1F is satisfied.

For other types of framing, or for highly irregular or unusual buildings, other criteria as determined by the project architect or structural engineer and approved by the Office of the State Architect

will be required to demonstrate safety against collapse from the maximum credible earthquake.

5 ADDITIONAL REQUIREMENTS

5.1 Isolation System

5.1.1 Back-up System

An alternate vertical load-carrying system should be provided in case of isolation system failure.

5.1.2 Environmental Conditions

In addition to the requirements for vertical and lateral loads induced by wind and earthquake, the isolation system should be designed with consideration given to other environmental conditions including aging effects, creep, fatigue, operating temperature and exposure to moisture or damaging substances.

5.1.3 Wind Loads

Isolated structures should resist design wind loads at all levels above the isolator level in accordance with Title 24 wind design provisions. At the isolator level, a wind restraint system should be provided as necessary to avoid human discomfort within the building and as necessary to limit lateral displacement in the isolation system to a value equal to that required between floors.

5.1.4 Fire Resistance

Fire resistance for the isolation system should meet that required for the building's columns, walls, or other structural elements.

5.1.5 Lateral Restoring Force

The isolation system should be configured to produce a restoring force sufficient to ensure that the maximum offset of any isolator unit does not exceed 33% of the design displacement. This requirement ensures that the isolation system will not have significant residual displacement following an earthquake, such that the isolated structure will be: 1) stable; and 2) in a condition to survive aftershocks and future earthquakes.

The isolation system need not be configured to produce a restoring force, as required above, provided the isolation system is capable of remaining stable under full vertical load and accommodating lateral displacements equal to four times the maximum offset. Isolation systems which are not configured to produce a lateral restoring should be capable of accommodating displacements significantly greater than the design displacement. Such displacements could occur in these systems as a result of directional biases in vibratory response, earthquakes with multiple segments of strong motion, or as a result of aftershocks.

5.1.6 Vertical Load Stability

The isolation system should provide a factor of safety of three (3) for vertical loads (dead load plus live load) in its laterally undeformed state. It should also be designed to be stable under the full design vertical loads at a horizontal displacement which is the greater of either 1.25 times the design displacement or four times the maximum offset for softening systems and sliding systems, or 1.25 times the design force for hardening systems.

The factor of safety of three for vertical loads was obtained from the NBS Special Publication 577 entitled "Development of a Probability Based Load Criterion for American National Standard A58."

The application of the factor of safety of three will be dependent on the type of isolation system, but should be applied as a well-

defined limiting stress or strain value. For example, systems with roller bearings may be governed by contact pressure, and systems with elastomeric bearings, may be governed by the tensile strain in the rubber.

An additional consideration for systems that incorporate an element that can overturn (e.g., elastomeric bearing with a dowel shear transfer mechanism) is the stability of the element at a horizontal displacement which is the greater of either 1.25 times the design displacement or four times the maximum offset. The required check of stability will ensure that there is sufficient margin prior to any loss of vertical load support capacity.

5.1.7 Overturning

The factor of safety against global structural overturning at the isolation level should not be less than 1.0 for the maximum credible event. Local uplift of individual elements is permitted provided the resulting deflections do not cause overstress or instability of building elements.

The intent of this requirement is to prevent global structural overturning and overstress of elements due to local uplift for the maximum credible event. Uplift in a braced frame or shear wall is acceptable provided the isolation system does not disengage from its horizontal resisting connection detail. The connection details used in some isolation systems are such that tension is not permitted on the system. If the tension capacity of an isolation system is to be utilized on resisting uplift forces, then component tests should be performed to demonstrate the adequacy of the system on resisting tension forces at the design displacement.

If an isolation system is designed to resist tensile forces, then this vertical load case should be included in the sequence of tests specified in Section 6.2(2).

5.1.8 Inspection and Replacement

Access for inspection and replacement of the isolation system should be provided.

The isolation system may sustain damage and the building may develop offsets as a result of extreme ground motion. After an earthquake, the building should be inspected for offset and possible damage to the isolation system. Damaged elements should be replaced or repaired. If offset in the isolation system is appreciable, then consideration should be given to jacking the building back to its original position, replacing damaged isolators, or altering the isolation system.

5.1.9 Quality Control

A quality control testing program for the isolation system should be established by the design structural engineer.

For systems based in whole or in part on an elastomeric bearing, this should include:

- a) All bearings should be tested in compression for 1.5 (DL + LL) in accordance with ASTM D-4014-81.
- b) Twenty percent (20%) of all bearings should be tested in combined compression and shear with the actual dead load to 50% shear strain in the elastomer.
- c) Destructive testing to determine fatigue and bond strengths should be performed on samples of at least one bearing on each project. Tests should comply with Part I of Caltrans Test Method 663-1978.

A test and inspection program is necessary for both fabrication and installation of the isolation system. Because seismic isolation is a developing technology, it may be difficult to reference standards for

testing and inspection. Reference can be made to standards for some materials such as elastomeric bearings (ASTM D4014-81 and British standards). Similar standards are required for other isolation systems. Special inspection procedures and load testing to verify manufacturing quality should be developed for each project. The requirements will vary with the type of isolation system used.

5.2 Structural System

5.2.1 Lateral Drift

The structure above the isolation system should conform to drift criteria of Title 24 for the maximum probable earthquake.

5.2.2 Horizontal Distribution of Force

A horizontal diaphragm or other structural elements located immediately above the isolation system should provide continuity between individual isolators, should have adequate rigidity to ensure that the building structure moves as a rigid body on top of the isolators, and should be strong enough to transmit forces (due to non-uniform ground motion) from one part of the building to another.

5.2.3 Separations

Minimum separations between the isolated building and surrounding retaining walls or other fixed obstructions should be not less than 1.1 times the design displacement, four times the maximum offset, or the minimum distance required for conventional structures.

5.3 Nonstructural Components

5.3.1 Components Above the Isolation Interface

Design of nonstructural component anchorage or bracing should be consistent with the response of the particular structure under consideration, as substantiated by analysis and/or tests for the maximum credible event. Alternatively, anchorage or bracing may be designed

using the force requirements of Tile 24 for non-base isolated structures.

5.3.2 Components Which Cross the Isolation Interface

All architectural, equipment and utility components which cross the seismic interface should be designed to accommodate 1.1 times the design displacement.

To accommodate the differential movement between the isolated building and the ground, provisions for flexible utility connections should be made. In addition, rigid structures crossing the interface, such as stairs, elevator shafts, and walls, should have details to accommodate differential motion at the isolator level without sustaining damage sufficient to threaten life safety.

6. REQUIRED TESTS OF THE ISOLATION SYSTEM

6.1 General

The deformation characteristics and damping values used in the design and analysis should be based on existing test data of the system and confirmed by the following tests on a selected sample of the components prior to construction. The isolation system tested should include the ultimate restraint system and the wind restraint system if such systems are used in the design. They should not be considered as manufacturing quality control requirements.

The design displacement and forces developed from these provisions are predicated on the basis that the deformational characteristics of the base isolation system have been previously defined by a comprehensive set of tests. If a comprehensive amount of test data are not available on a system, then major design alterations in the building may be necessary after the tests are complete. This would result from variations in the isolation system properties assumed for design and those obtained by test. Therefore, it is advisable that prototype systems be tested during the early phases of design if insufficient test data are not available on an isolation system.

6.2 Sequence of Tests

The following sequence of tests should be performed on at least two components of the full-size isolation system. The test specimens should include the ultimate restraint system, the wind restraint system, as well as the individual isolators, if such systems are used in the design. Specimens tested should not be used for construction.

Each set of tests should be performed at three different rates of loading. The rate of loading should correspond to 1/2, 1, and 2 times the inverse of the isolated period, defined as a cycle of maximum response for the maximum credible event.

1. Twenty fully reversed cycles of loading at a force corresponding to the design wind force. If a sacrificial wind restraint system is to be utilized, its ultimate capacity should be established by test.
2. Three fully reversed cycles of loading at each of the following increments of the design displacement or design force of Section 4: 0.25, 0.50, 0.75, 1.0, and 1.25.
 - a. For softening systems, a displacement-controlled test should be performed with the specified increments based on the design displacement of Section 4.1.
 - b. For hardening systems, a force-controlled test should be performed with the specified increments based on the maximum force of Section 4.1.

Exception: If variations in effective stiffness greater than $\pm 15\%$ occur in these three cycles of loading at a given amplitude, then three additional cycles of loading at the given amplitude should be performed.

3. Ten fully reversed cycles of loading at 1.0 times the design displacement.

If an isolator is also a vertical load carrying element the above sequence of tests should be performed for each of three different vertical loads as follows:

- i) DL
- ii) DL + 20% DL + 50% Overturning Force
- iii) DL - 20% DL - 50% Overturning Force

For each cyclic test the force deflection and hysteretic behavior of the test specimen should be recorded. The vertical load carrying elements of the isolation system should also be statically load tested to demonstrate stability under a vertical load of 1.5 times dead load plus reduced live load plus seismic overturning at 1.0 times the design displacement.

The required sequence of tests will experimentally verify:

1. The assumed stiffness and capacity of the wind restraining mechanism.
2. The variation in the isolator's deformational characteristics with rate of loading, amplitude, and with vertical load, if it is a vertical load-carrying member.
3. The variation in the isolator's deformational characteristics for a realistic number of cycles of loading at the design displacement.

Force-deflection tests are not required if similar-sized components have been previously tested using the specified sequence of tests.

Variations in effective stiffness greater than + 15% would require an additional 3 cycles of loading at a given amplitude to determine if

deterioration continues and variations greater than $\pm 20\%$ would be cause for concern. The variations in the vertical loads required for tests of isolators which carry vertical, as well as lateral loads, are necessary to determine possible variation in the system properties due to vertical ground acceleration and overturning force. Test set-ups may not be capable of incorporating very low vertical loads because of static instability in the test assembly. Consequently, a compromise may be required to set a limit on the lower vertical load. Clearly, the engineer will have to use judgement in selecting the appropriate dead loads and overturning forces for the test system because these will vary throughout the structure. As noted in Section 6.3, the design values of the isolation system are based on the full DL tests provided the average results of the other two vertical load tests do not vary by more than 10% from the DL tests. This requirement is based on the premise that the overturning forces are equal and opposite.

6.3 System Adequacy

The seismic isolation system test performance may be assessed as adequate if:

1. The test force-deflection plots for all tests specified in Section 6.2 have a positive incremental stiffness.
2. There is less than a 20% deterioration in the equivalent stiffness for the six cycles of test (if six are performed at a given vertical load specified in Section 6.2 (2)).
3. There is a less than 25% deterioration in the equivalent stiffness and damping values for the 10 cycles of tests at a given vertical load specified in Section 6.2(3).

An incremental reduction in load resistance over a displacement range greater than 1/2 inch is indicative of deterioration or instability, neither of which can be tolerated for the specified tests. If this occurs, then the tests should be extended so that at least

another 3 cycles of loading are required with a 25% increase in the test amplitude or force at which this phenomena was observed. If reduction in load resistance continues, then the system is inadequate.

If the variation in stiffness and damping values for a given vertical load are less than 25%, then the system is adequate. If the variations are greater than 40%, then the seismic isolation system should not be used. Variations between 25% and 40% would be cause for concern and should probably require tests on additional isolators.

7. MONITORING AND INSTRUMENTATION

The isolation system should be monitored for the life of the building, and access for inspection and replacement of the system should be provided. A program for monitoring should be established by the person responsible for the structural design and would be submitted for approval with the plans and specifications. The monitoring program would become a part of the approval. The implementation and maintenance of the program should then be the responsibility of the owner of the building. Approval and enforcement of this program would be delegated to the Office of the State Architect by the Office of State Health Planning and Development.

As a minimum, the program should include the following:

1. Approved instrumentation would be provided and maintained to record structural motion at appropriate locations within the building and at the levels of the bottom and the top of the isolators. Verified reports confirming adequate maintenance and monitoring of the instruments should be submitted to the Office of the State Architect semi-annually. Hard copies of accelerograms should be submitted to OSA within one month of a recorded event. Records for significant events as described in 2. should be accompanied by appropriate response spectra.
2. Visual inspections should be made by a Structural Engineer after every significant earthquake (defined as an earthquake

large enough to produce a ground motion peak acceleration of 0.2g or larger or a displacement record of two inches or greater at the base of the isolators). The inspection should consist of viewing the structural performance of the building, the records produced by the building instrumentation, and a visual examination of the isolators and their connections for deterioration, offset, or physical damage. A report of such inspection, including an analysis for the recorded ground motion and conclusions on the continuing adequacy of the structure, should be submitted to OSA for review within three months of such an event.

3. Selected isolators (approximately two percent of the total, but not less than two) should be removed temporarily at intervals not to exceed 10 years for physical testing. Tests should determine lateral stiffness under the design dead load and design displacement. The results of these test should be compared to the data obtained by the qualification testing. If the mean value of the material properties determined by the testing vary by more than one standard deviation of the qualification test data from that data, the analysis of the structure-base isolation model should be repeated to verify that the design is still valid. A report of the results of testing, including conclusions on the continuing adequacy of the structure and isolators, should be submitted to OSA for review within three months of the testing.

APPENDIX A

Suggested Criteria for Determining Nearly Elastic Structural Behavior

The following concepts have been extracted from "Seismic Design Guidelines for Essential Buildings", Department of the Army, TM5-809-10-1. If additional clarification or examples are required, then reference to that document is recommended. These concepts are incorporated solely for the purpose of defining when linear model of the superstructure is appropriate for dynamic analysis.

Nearly elastic behavior is interpreted as allowing some structural elements to exceed their strength level within specific limits. If the structural response is within the limits required for nearly elastic behavior, then the structural deformations may be assumed to be equal to those found by the use of a linear model of the structure. For a structure that has a multiplicity of structural elements that form the lateral-force-resisting system, the yielding of a small number of elements will generally not affect the overall elastic behavior of the structure if the excess load can be distributed to other structural elements that have not exceeded their yield strength.

A linear model of the superstructure is acceptable if the procedure that evaluates overstresses of individual elements, outlined below, is followed, and the limits given in Table A-1 are not exceeded.

1. Perform analysis of the structure using the appropriate maximum credible earthquake response spectrum or time histories.
2. Calculate the forces on all of the structural elements. Load combinations are given below and are in Section 2-2312(d) 1D. These forces will be defined as the demand forces and denoted with subscript D (e.g., M_D , V_D , F_D).

$$\text{Demand Force} = 1.2D + 0.5L + E$$

$$\text{Demand Force} = 0.8D + E$$

where:

L = reduced live load

E = earthquake force from (1)

3. Calculate the elastic capacities of all the structural elements in the same force units used in paragraph (2) above. These forces will be defined as the capacity forces and denoted with the subscript C (e.g., M_C , V_C , F_C).

The elastic capacity is set to equal the strength capacities of the structural components.

- a. Reinforced Concrete: The strength capacity for reinforced concrete elements will be given by the ACI Building Code 318.
- b. Structural Steel: In lieu of strength design criteria for structural steel, the working stresses for ordinary or nonseismic construction may be increased by 70 percent to provide the strength capacity; for example,

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.7$$

- c. Reinforced Masonry Design: In lieu of a strength design criteria for reinforced masonry, working stresses for ordinary or nonseismic construction may be increased by 70 percent to provide the strength capacity; for example, $f_a \leq 1.7 F_a$.
- d. Connections: All connections that do not develop the strength of the connecting members will have a strength reduction factor of 0.75. This reduction factor will be applied to the strength capacity of the connection material.

4. Calculate the ratio of the demand forces to the capacity forces of all the structural elements. These ratios will be defined as inelastic demand ratios.
5. Review the inelastic demand forces for uniformity, symmetry, mechanisms, and relative values. Compare values to limits set forth in Table A-1. If any of the following conditions exist, a linear model representation of the structure is not acceptable and the structure must be analyzed by developing a simplified inelastic representation of the building which incorporates the progressive inelastic deformation of the structural members.
 - a. Exceeding the inelastic demand ratios of Table A1.
 - b. Unsymmetrical yielding, on a horizontal plane in the structure, that will decrease the torsional resistance.
 - c. Hinging of columns at a single story level that will cause a mechanism.
 - d. Discontinuity in vertical elements that can cause instability or fracture.
 - e. Unusual distributions of inelastic demand ratios.
6. Engineering judgment is required for the structural evaluation of the post-yield analysis. If the review of the inelastic demand ratios satisfies the requirements of paragraph (5) above, it may be assumed that the inelastic deformations can be adequately approximated by elastic analysis.

A-1 - Limits on Inelastic Demand Ratios

Building System	Element	Demand Ratios
Steel DMRSF	Beams	2.0
	Columns	1.25
Braced Frames	Beams	1.5
	Columns	1.25
	Diag. Braces	1.25
	K-Braces	1.0
	Connections	1.0
Concrete DMRSF	Beams	2.0
	Columns	1.25
Concrete Walls	Shear	1.25
	Flexure	2.0
Masonry Walls	Shear	1.1
	Flexure	1.5

APPENDIX B

Determination of Effective Stiffness and Damping

The intent of these requirements is to ensure that the deformational properties used in design result in the maximum design forces and displacements. For determining design displacement, this means using the lowest damping value and minimum effective stiffness of the isolation system. For determining design forces, this means using the lowest damping value and maximum effective stiffness of the isolation system.

B.1 Stiffness and Damping Properties

The effective stiffness, and equivalent viscous damping to be used to determine the design base shear forces and displacements by response spectrum analysis may be determined as follows:

1. The minimum stiffness, k_{min} , to be used in the determination of the design displacement should be based on the average of the lowest three effective stiffness values determined from each of the 10 cycles of loading in the test sequence specified in 6.2(3) for the dead load test, provided these lowest three values do not differ by more than 15%. If they do, then the lowest value should be used:
2. The maximum stiffness, k_{max} , to be used in the determination of the design base shear force should be based on the average of the highest three effective stiffness values determined from each of the 10 cycles of loading in the test sequence specified in 6.2(3) for the dead load test provided these highest three values do not differ by more than 15%. If they do, then the highest value should be used.
3. The equivalent viscous damping, if it is to be used in a linear system in the determination of the design displacement, should be based on the average of the lowest three equivalent viscous

damping values determined from each of the 10 cycles of loading in the test sequence specified in 6.2(3) for the dead load test, provided the lowest three values do not differ by more than 15%. If they do, then the lowest value should be used.

4. For isolation systems that act as vertical load-carrying members, the test sequence specified in 6.2(3) is required for 3 different vertical loads. Items (1), (2), and (3) above, are based on the vertical dead load test results provided the average equivalent stiffness and damping values for each of the 10 test cycles for all greater and lesser vertical load tests are within 15% of the corresponding average values of the dead load vertical test. If they are not, then Items (1), (2), and (3) above should be based on the appropriate highest or lowest values from the three sets of vertical load tests.

B.2 Determination of Stiffness Characteristics

The effective stiffness of the system at each test displacement should be calculated for each cycle of loading as follows:

$$k_{eff} = \frac{F_p - F_n}{\Delta_p - \Delta_n}$$

where, F_p , Δ_p , and F_n , Δ_n are the maximum positive and negative forces and displacements, respectively. If the minimum effective stiffness is to be determined then $F_{p,min}$ and $F_{n,min}$ should be used in the equation. If the maximum effective stiffness is to be determined, then $F_{p,max}$ and $F_{n,max}$ should be used in the equation.

The effective stiffness is determined from the hysteresis loops shown in Figure B1. Stiffness may vary considerably as the test amplitude increases, but should be reasonably stable ($\pm 10\%$) for more than one cycle at a given amplitude.

B.3 Determination of Damping Characteristics

The equivalent viscous damping ratio (β) for systems with velocity-related damping for each cycle of loading shall be calculated as:

$$\beta = \frac{1}{2\pi} \frac{\text{Area of Hysteresis Loop}}{k_{\text{eff}} \Delta_{\text{max}}^2}$$

The determination of equivalent viscous damping is reasonably reliable for systems whose damping characteristics are velocity-dependent. For systems that have amplitude-dependent energy-dissipating mechanisms, significant problems arise in determining an equivalent viscous damping value, since it is difficult to relate velocity and amplitude-dependent phenomena. The equivalent viscous damping concept can only be used for linear systems since nonlinear systems must be modeled explicitly.

$$k_{\max} = \frac{F_{p,\max} - F_{n,\max}}{\Delta_p - \Delta_n}$$

$$k_{\min} = \frac{F_{p,\min} - F_{n,\min}}{\Delta_p - \Delta_n}$$

FIGURE B-1: EXAMPLE FORCE-DEFLECTION TEST CURVES USED TO DETERMINE MAXIMUM AND MINIMUM EFFECTIVE STIFFNESS

$$k_{max} = \frac{F_{p,max} - F_{n,max}}{\Delta_p - \Delta_n}$$

$$k_{min} = \frac{F_{p,min} - F_{n,min}}{\Delta_p - \Delta_n}$$

FIGURE B-1: EXAMPLE FORCE-DEFLECTION TEST CURVES USED TO DETERMINE MAXIMUM AND MINIMUM EFFECTIVE STIFFNESS

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Command
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SEISMIC DESIGN CRITERIA FOR BASE ISOLATED U.S. NAVY ESSENTIAL BUILDINGS

ABSTRACT This report presents design criteria for laminated rubber seismic isolation systems for use in construction of Navy essential buildings. Base isolation has been shown to limit seismic motions transmitted to structures and is effective in reducing damage. The design criteria, applicable only to structures having structural detailing as required in Seismic Zones 3 and 4, utilizes two structural load levels and has levels of performance associated with each. A structure designed according to these provisions will withstand the most probable maximum earthquake with inelastic demand ratios of 2.0 in beams and 1.25 in columns.

NAVAL CIVIL ENGINEERING LABORATORY PORT HUENEME CALIFORNIA 93043

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METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
in ft yd mi	inches feet yards miles	<u>LENGTH</u>		cm cm m km
		*2.5	centimeters	
		30	centimeters	
		0.9	meters	
in ² ft ² yd ² mi ²	square inches square feet square yards square miles acres	1.6	kilometers	km ² m ² km ² ha
		<u>AREA</u>		
		6.5	square centimeters	
		0.09	square meters	
oz lb	ounces pounds short tons (2,000 lb)	0.8	square meters	m ² km ² ha
		2.6	square kilometers	
		0.4	hectares	
		<u>MASS (weight)</u>		
tsp Tbsp fl oz c pt qt gal ft ³ yd ³	teaspoons tablespoons fluid ounces cups pints quarts gallons cubic feet cubic yards	28	grams	g kg t
		0.45	kilograms	
		0.9	tonnes	
		<u>VOLUME</u>		
°F	Fahrenheit temperature	5	milliliters	ml ml ml l l l m ³ m ³
		15	milliliters	
		30	milliliters	
		0.24	liters	
		0.47	liters	
		0.95	liters	
		3.8	liters	
		0.03	cubic meters	
		0.76	cubic meters	m ³ m ³
		<u>TEMPERATURE (exact)</u>		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
<u>LENGTH</u>				
mm	millimeters	0.04	Inches	in
cm	centimeters	0.4	Inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
<u>AREA</u>				
cm ²	square centimeters	0.16	square inches	in ²
m ²	square meters	1.2	square yards	yd ²
km ²	square kilometers	0.4	square miles	mi ²
ha	hectares (10,000 m ²)	2.5	acres	
<u>MASS (weight)</u>				
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1,000 kg)	1.1	short tons	
<u>VOLUME</u>				
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m ³	cubic meters	36	cubic feet	ft ³
m ³	cubic meters	1.3	cubic yards	yd ³
<u>TEMPERATURE (exact)</u>				
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price \$2.25, SD Catalog No. C13.10-286.

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1. EXECUTIVE SUMMARY

This document has drawn extensively upon work prepared by an Ad Hoc Committee of the Seismology Committee of the Structural Engineers Association of California (SEAOC) (Kircher, Hart and Romstad, 1989). That document was entitled "An Acceptable Method for Design and Review of Hospital Buildings Utilizing Base Isolation" and was adopted by the State of California Building Safety Board for application to California Hospital Buildings.

The seismic design criteria developed as part of this research is based on limit state design theory and it assumes that the building can be substantially decoupled from potentially damaging earthquake motions by using base isolation. Thus the level of building response can be significantly reduced. A general discussion of seismic isolation, recent advancements and its advantages as it relates to US Navy facilities are documented elsewhere Hart, 1989.

The design criteria presented herein is only applicable to laminated layer rubber seismic isolation systems.

Acceptable performance in the context of this criteria means that the isolators will:

- remain stable for required design displacements,
- provide increasing energy dissipation with increasing displacement,
- not suffer a loss in force resisting capacity under repeated cyclic loading, and
- have quantifiable engineering parameters (e.g., force-deflection characteristics and damping).

The design criteria presented herein is only for US Navy Buildings that have structural detailing consistent with that required in Seismic Zones 3 or 4. In this Seismic Zone special structural detailing requirements exist and therefore system ductility is expected. The design

criteria requires two structural analyses. The first analysis, denoted Analysis 1, is the analysis used to develop the design and it is consistent with the U.S. Navy Essential Building Criteria. This analysis can be a response spectra analysis. Alternately, the first analysis, can be a time history structural dynamic analysis of the isolated building. The second analysis, denoted Analysis 2, is a response spectra analysis that is only intended to provide a lower bound safety net design. Both of these analysis are intended to insure that the isolated structure can experience a very severe earthquake with limited ductility demands and remain stable.

The seismic design criteria utilizes in Analysis 1 two basic levels of ground motion that are denoted EQ-I and EQ-II. The EQ-I earthquake ground motion has approximately a 50% chance of being exceeded in a 50 year time period. Structural elements above the isolation system are required to remain essentially linear elastic for the EQ-I earthquake. Twenty percent of the beams in any one story can not have an inelastic demand ratio (IDR) of more than 1.25 and 10% of the columns must not have an IDR of more than 1.25. A second design earthquake, denoted EQ-II, is the most probable maximum earthquake that can be expected to take place. This design level earthquake has approximately a 10% chance of being exceeded in a 100 year time period. For the EQ-II earthquake the building can respond inelastically but must have sufficient capacity to resist system collapse. The IDR for beams and columns are limited for the EQ-2 earthquake to 2.0 and 1.25, respectively. The IDR limits noted above for EQ-II are for ductile moment resisting space frame building systems. For other systems the IDR limits are as given in Table 4-2 of the Technical Manual TM 5-810-10-1, Seismic Design Guidelines for Essential Buildings.

Analysis 2 is intended to provide a minimum force and displacement capacity safety net design. In this analysis the structural system design in Analysis 1 is assumed to be a fixed base building at the isolation level. It is then subjected to the same EQ-I earthquake response spectra as developed as part of the Analysis 1 scope of work. The beams and

columns in the building when subjected to the EQ-I earthquake, at the fixed base building period, must have an IDR less than 3.

2. SEISMIC DESIGN CRITERIA

A. LATERAL-FORCE DESIGN REQUIREMENTS

A.1 General

Every seismic-isolated building and every portion thereof shall be designed and constructed to resist the deformations and forces produced by lateral displacements and forces as provided in this chapter. The deformations and forces shall be calculated as the effect of a force applied horizontally and the isolation-interface level and at each floor level or roof level above the interface level. The force shall be assumed to come from any horizontal direction.

Isolator units shall be tested to verify deformational properties and design capacity for the lateral displacements prescribed by this document.

Where prescribed wind forces produce greater deformations or stresses, such loads shall be used in lieu of forces resulting from earthquake forces.

These provisions may only be used for buildings that satisfy the following conditions:

- (1) No plan or vertical irregularities exist.
- (2) The elastic period of the building on the isolator system shall not exceed 3.0 sec and shall be at least four times the building period calculated assuming that the building is fixed at the isolator level.
- (3) The building is designed to meet the detailing requirements of Seismic Zones 3 and 4.
- (4) No significant mass eccentricities exist.

A.2 Definitions

The following definitions apply to the provisions of this section:

EQ-I EARTHQUAKE is the earthquake ground shaking corresponding to an event with approximately a 50% chance of being exceeded in a 50 year time period.

EQ-II EARTHQUAKE is the earthquake ground shaking corresponding to an event with approximately a 10% chance of being exceeded in a 100 year time period.

DESIGN DISPLACEMENT is the maximum lateral displacement required by Analysis 1.

MAXIMUM OFFSET is the residual or permanent displacement which would occur in the isolation system at the end of free-vibration response of the isolated structure.

INELASTIC DEMAND RATIO (IDR) is the ratio of the demand force to the capacity force of any structural element. This inelastic demand ratio sets the limit of the force ductility demand on the element, therefore it is element dependent as well as structural system dependent. The limiting values of the IDR's shall be obtained from Table 4-2 of the Technical Manual TM 5-810-10-1, Seismic Design Guidelines for Essential Buildings.

ISOLATION SYSTEM is the collection of structural elements which includes all individual isolator units, all structural elements which transfer force between components of the isolation system, and all connections to other structural elements. The isolation system also includes the wind-restraint system. The force deflection characteristics of the isolation system may be characterized as linear if force remains essentially proportional to deflection, or as nonlinear for systems which either harden or soften with increased deflection or are governed predominantly by friction force.

ISOLATION INTERFACE is the boundary between the upper portion of the structure, which is isolated, and the lower portion of the structure.

ISOLATION UNIT is a flexible structural component of the isolation system which permits large lateral deformations under design seismic load. Conceptually, isolator units may be thought of as either a spring, a damper or a frictional device, or any combination of the above; and may be used either as part of or in addition to the weight supporting system of the building.

WIND-RESTRAINT SYSTEM is the collection of structural elements which provide restraint for the seismic-isolated structure for wind loads. The wind-restraint system may be either an integral part of the isolator unit or may be a separate device.

A.3 Symbols and Notations

The following symbols and notations apply to the provisions of this section:

- D = Lateral seismic displacement of the isolation system (in inches) at the center of rigidity, of the structure in the direction under consideration.
- F_n = Maximum negative force in the isolation system during each cycle of testing.
- F_p = Maximum positive force in the isolation system during each cycle of testing.
- F_x = Lateral force applied at level x .
- g = Acceleration due to gravity.
- k_{eff} = Effective stiffness of isolation system determined by cyclic testing.
- T = Period of seismic-isolated structure in seconds in the direction under consideration.
- V_b = The total lateral force or shear on elements at or below the isolation interface.
- V_s = The total lateral force or shear on elements above the isolation interface.

- W = The total dead load of the structure above the isolation interface, and applicable portions of other loads.
- w_i, w_x = The portion of W which is located or assigned to levels i or x , respectively.
- Z_b = Numerical coefficient related to the seismicity of a region. Seismic Zone 3 and 4 that have Z_b values of 0.3 are 0.4, respectively.
- Δ_p = Maximum positive displacement of isolation system during each cycle of testing.
- Δ_n = Maximum negative displacement of isolation system during each cycle of testing.
- K = Horizontal force factor, as per Tri-Service Manual TM 5-809-10.

B. MINIMUM EARTHQUAKE DISPLACEMENTS AND FORCES

Every seismic-isolated building shall be designed and constructed to resist minimum earthquake displacements and forces as specified by Sections C and D.

The total lateral seismic force above the isolators shall not be less than:

1. The forces resulting from Analysis 2.
2. The base shear corresponding to the design wind load and,
3. The yield level of a stiffness softening system, the ultimate capacity of a sacrificial wind-restraint system, and the static friction level of a sliding system.

If the reduced base shear force is less than these limits, then the Analysis 1 design forces shall be increased proportionately so that the greater of the three limits is satisfied.

C. ANALYSIS 1: DYNAMIC ANALYSIS

C.1 General

This section defines the minimum earthquake displacements and forces that are to be used for design.

C.2 Preliminary Design

A preliminary isolation system design shall be developed to withstand lateral seismic displacements which act in the direction of each of the main horizontal axes of the structure in accordance with Formula (C-1).

$$D = 10 Z_b T \quad (C-1)$$

The isolated-structure period, T , shall be determined using the deformation characteristics of the isolation system in accordance with the formula:

$$T = 2\pi \sqrt{\frac{W}{k_{eff}g}} \quad (C-2)$$

The value of T obtained from Formula (C-2) shall not exceed 3.0 seconds.

The isolation system, the foundation system, and all structural elements below the isolation system shall be preliminary designed to withstand a lateral seismic force, V_b , using all appropriate provisions for a non-isolated structure where:

$$V_b = \frac{k_{eff}D}{1.5} \quad (C-3)$$

The structure above the isolation system shall be preliminary designed to withstand a minimum shear force, V_s , using all appropriate provisions corresponding to the K value for a non-isolated structure where:

$$V_s = \left[\frac{k_{eff} D}{1.5} \right] K \quad (C-4)$$

The total force shall be distributed over the height of the structure above the isolation interface in accordance with the following formula:

$$F_x = \frac{V_s w_x}{\sum_{i=1}^n w_i} \quad (C-5)$$

At each level designated as x , the force F_x shall be applied over the area of the building in accordance with the mass distribution at the level. Forces in each structural element shall be calculated as the effect of force, F_x , applied at the appropriate levels above the base.

C.3 Dynamic Analyses Procedure

Base isolation design displacements and forces shall be obtained from a dynamic analysis. The analytical model of the building shall be three-dimensional and shall include both the deformational characteristics of the isolation system and the deformational characteristics of the building. An elastic Response Spectrum Analysis may be used to calculate the force in all structural elements for the EQ-I earthquake and the isolator stiffness is equal to a k_{eff} value consistent with the EQ-I isolator response. An EQ-II analysis is required to insure that the structure has sufficient ductility capacity to withstand this event. If a response spectra analysis is used then a load deflection inelastic analysis is required as described in The Technical Manual TM 5-810-10-1 to ensure that the system capacity exceeds the EQ-II system demand. The effective stiffness and damping values of the isolation system used in the response spectrum analyses shall be substantiated by tests as specified in Section D. A Time History Analysis may be used in lieu of the response spectra analysis and it requires an analysis for at least six appropriate time histories.

C.4 Seismic Input

A properly substantiated, site-specific response spectra for the EQ-I and EQ-II earthquakes are required. When used in the analysis the earthquake time histories shall be selected from different recorded events and scaled such that their 5% damped response spectrum does not fall below the design or site specific spectrum by more than 10% at any period. For sites within 15 km of a major active fault the seismic impact must incorporate near-fault phenomena.

C.5 Interstory Drifts

Interstory drifts is defined as the ratio of the differential lateral displacement of the top and the bottom of a story to the story height. For Analysis 1, interstory drifts shall not exceed 0.005 when the isolated building is subjected to EQ-I, and 0.010 when subjected to EQ-II.

C.6 Dead Loads for the Isolated Building

Above the isolator the dead loads shall be increase by 10% to incorporate the effects of vertical accelerations on columns and long span beams.

D. ANALYSIS 2

The structure designed in Analysis 1 shall now be assumed to be fixed at the isolator level and the IDR for all beams and columns shall not exceed 3 for a response spectra analysis of the structure in this condition for the EQ-I event.

E. REQUIRED TESTS OF ISOLATION SYSTEM

E.1 General

The deformation characteristics and damping values used in the design and analysis shall be based on existing test data of the system and confirmed by the following tests on selected sample of the components prior to construction. The tests specified are for validating the properties of the base isolation system. They should not be considered as manufacturing quality control requirements.

E.2 Sequence of Tests

The following sequence of tests shall be performed on at least two components of the full size isolation system. For each test cyclic force-deflection and hysteretic behavior of the test specimen shall be recorded. The test specimens shall include the wind restraint system, if such system is used in the design. Specimens tested shall not be used for construction.

1. Twenty cycles of loading at a force corresponding to the design wind force. If a sacrificial wind-restraint system is to be utilized its ultimate capacity shall be established by test.
2. Three cycles of loading at each of the following increments of the design displacement or design force: 0.25, 0.50, 0.75, 1.0, 1.25, 1.50.
3. Ten cycles of loading at 1.50 times the design displacement.

If an isolator is also a vertical load carrying element then item 3 of the above sequence of tests shall be performed for three different vertical loads as follows:

- i) DL

ii) DL + 25% LL + 1.2 (Overturning Force)

iii) DL - 25% LL - 1.2 (Overturning Force)

E.3 Determination of Stiffness Characteristics

The effective stiffness of the system at each test displacement shall be calculated for each cycle of loading as follows:

$$k_{eff} = \frac{F_p - F_n}{\Delta_p - \Delta_n} \quad (E-1)$$

where F_p , Δ_p and F_n , Δ_n are the maximum positive and negative forces and displacements, respectively.

E.4 Determination of Damping Characteristics

The equivalent viscous damping ratio (β) for each cycle of loading shall be calculated as:

$$\beta = \frac{1}{2\pi} \left[\frac{\text{Area of Hysteresis Loop}}{k_{eff} \Delta_{max}^2} \right] \quad (E-2)$$

E.5 System Adequacy

The base isolation system test performance shall be assessed as adequate if:

1. The measured force deflection relationships for all tests specified in Section E.2 has an increase in maximum positive and negative force for all increases in corresponding maximum displacements.
2. There is less than a 10% change in amplitude of zero to maximum positive or negative force, effective stiffness and equivalent viscous damping for any of the cycles of test performed at a given displacement level as specified in Section E.2.

F. ADDITIONAL REQUIREMENTS

F.1 Environmental Conditions

In addition, to the requirements for vertical and lateral loads induced by wind and earthquake, the isolation system shall be designed with consideration given to other environmental conditions including aging effects, creep, fatigue, operating temperature and exposure to moisture or damaging substances.

F.2 Wind Loads

Isolated structures shall resist design wind loads at all levels above the isolator level in accordance with the general wind design provisions. At the isolator level a wind restraint system shall be provided to limit story drift to 0.002 times the story height.

F.3 Fire Resistance

Fire resistance for the isolation system shall meet that required for the building's columns, walls or other structural elements.

F.4 Lateral Restoring Force

The isolation system shall be configured to produce a restoring force sufficient to ensure that the maximum offset of any isolator unit does not exceed 15% of the design displacement calculated in Sections B and C.

The maximum offset of any isolator unit shall be determined by an experimental snap-back test or by an analytical free-vibration time history analysis. If time history analysis is used to determine maximum offset, then the hysteretic behavior of the isolation system shall be modeled explicitly using the force-deflection characteristics determined by Section E.2 tests. For either snap-back testing or time history analysis, mass shall be included, as

necessary, to accurately represent the inertial effects of the isolated structure.

F.5 Vertical Load Stability

The isolation system shall provide a factor of safety of three (3) for vertical loads (dead loads plus live load) in its laterally undeformed state. It shall also be designed to be stable under the full design vertical loads at a horizontal displacement which is the greater of either 1.5 times the design displacement or four times the maximum offset for softening systems and sliding systems, or 1.5 times the design force for hardening systems.

F.6 Overturning

The factor of safety against global structural overturning at the isolation level shall be not less than 1.0 for the load combinations required by US Navy Criteria TM 5-810-10-1. All gravity and seismic loading conditions shall be investigated, except that seismic forces for overturning calculations shall be based on the maximum base shear force for the superstructure; and W shall be used for the vertical restoring force.

Local uplift of individual elements is permitted provided the resulting deflections do not cause overstress or instability of building elements.

F.7 Inspection and Replacement

Access for inspection and replacement of the isolation system shall be provided.

F.8 Quality Control

A quality control testing program for the isolation system shall be established by the design engineer.

F.9 Design Review

Engineering review of the isolation system concept and design is required.

F.10 Lateral Drift of the Structural System

The structure above the isolation system shall conform to the drift criteria in the US Navy Essential Building criteria for EQ-I and EQ-II earthquakes.

F.11 Separations

Minimum separations between the isolated building and surrounding retaining walls or other fixed obstructions shall be not less than the greater of either 1.5 times the design displacement, four times the maximum offset or the minimum distance required for conventional structures. The intent of this provision is to avoid impact with adjacent structures.

APPENDIX A

REFERENCES

- ATC, 1978 *Tentative Provisions for the Development of Seismic Regulations for Building*, Applied Technology Council Report ATC-3-06, Redwood City, California.
- BSSC, 1988, *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*, Building Seismic Safety Council, Washington, D.C.
- ICBO, 1985, 1988, *Uniform Building Code*, International Conference of Building Officials, Whittier, California.
- SEAOC, 1955, 1988, *Lateral Force Requirements*, Seismology Committee of the Structural engineers Association of California.
- TM 5-810-10-1, 1986, *Seismic Design Guidelines for Essential Buildings, Technical Manual*, Departments of the Army, the Navy and the Air Force Report TM 5-809-10-1/NAVFAC P-355/AFM 88-3, Chapter 13, Section A. Washington, D.C.
- TM 5-809-10, 1986, *Seismic Design for Buildings, Basic Design Manual*, Departments of the Army, the Navy and the Air Force Report TM 5-809-10/NAVFAC P-355/AFM 88-3, Chapter 13, Washington, D.C.
- Teal, E.J., *Seismic Drift Control Criteria*, AISC Engineering Journal Vol. 12, No. 2, 1975.

APPENDIX B

CASE STUDY BUILDING - NO ISOLATORS

B.1 DESCRIPTION OF BUILDING

A three-story building is considered in this appendix. The building is a modified version of a building example discussed in Technical Manual TM 5-810-10-1. The lateral seismic resistance is provided by transverse ductile moment-resisting steel frames (DMRSF) and longitudinal steel braced frames (BSF). There are a series of interior vertical load-carrying columns and girders in addition to the space frame, see Figure (B.1-1).

The building is considered to be located in Seismic Zone No. 4 and the soil profile of the site consists of a dense soil where the depth exceeds 200 feet. Table B.1-1 shows roof and typical floor loading. Only the transverse direction is considered in this case study.

B.2 DESIGN USING UBC-85

B.2.1 General

The base shear coefficient is calculated using the 1985 Uniform Building Code (UBC) procedure, and then the resulting base shear in each direction is distributed to the stories. The roof and the floor systems form rigid diaphragms. Therefore, the story forces are distributed to the frames in direct relation to each frame stiffness.

The stresses in beams are checked using working stress criteria. While columns stresses are checked using both working stress and ultimate stress (strength) approaches.

ROOF PLAN

DUCTILE-FRAME
TYP.

2ND & 3RD FLOOR PLANS

LONGITUDINAL BRACED FRAMES

ON LINES A & C

TRANSVERSE MOM
FRAMES, ON LINE

Figure B.1-1

Table B.1-1

Loads:	Roof: DL = 25.7 psf, Floor: DL = 74.5 psf,	LL = 20.0 psf LL = 50.0 psf
Weights:	Roof total weight Floor total weight Total building weight	= 375 kips = 707 kips = 1789 kips

Table B.2-1

Seismic Zone Coefficient (zone 4)	Z = 1.0
Occupancy Importance Factor (essential building)	I = 1.5
Horizontal Force Factor (DMRSF, BSF)	K = 0.67
Soil Profile Coefficient (soil type S2)	S = 1.2
Building Period T	T = 0.67 sec.
Numerical Coefficient C $C = 1 / 15\sqrt{T} < 0.12$	C = 0.081
CS < 0.140	CS = 0.097
Base Shear Coefficient	ZIKCS = 0.098
Base Shear $V = ZIKCS W$	V = 175 kips

B.2.2 Building Period

1. The period is calculated using the 1985 UBC, Chapter 23, Section 2312, Formula 12-3B: $T = 0.1 N$ where N is the number of stories. It follows that $T = 0.1 (3) = 0.3$ seconds. This period is conservative and provides a lower bound.

2. Alternate Method

- a. *Upper Bound:* The period is calculated using the 1985 UBC, Chapter 23, Section 2312, Formula 12-3.

$$T = 2\pi \sqrt{\frac{\sum W_i \delta_i^2}{g \sum f_i \delta_i}}$$

Or using Teal's Formula (AISC Engineering Journals, 2nd and 4th Quarters, 1975), which is its equivalent.

$$T = 0.25 \sqrt{\frac{\Delta}{C_1}} = 0.25 \sqrt{\frac{\text{Roof deflection}}{\text{Roof acceleration}}}$$

In this later formula, the following assumptions are made:

$$\Delta \approx \frac{2}{3} (\text{allowable deflection}) = \frac{2}{3} (0.005 h_n)$$

$$C_1 \approx ZICS \quad \text{where} \quad C = \frac{1}{15\sqrt{T}}$$

Substituting these values of Δ and C_1 into Teal's formula:

$$T = 0.11 \left[\frac{h_n}{ZIS} \right]^{\frac{2}{3}}$$

where h_n is the total height of the building (feet). It follows that:

$$T = 0.11 \left[\frac{33}{1 \times 1.5 \times 1.2} \right]^{\frac{2}{3}} = 0.76 \text{ seconds}$$

- b. *Limiting Value:* Use the Navy's Design Basic Manual Formula (Chapt. 4, Para. 4-3d(5))

$$T = 1.4 C_T h_n^{\frac{3}{4}} \quad \text{where} \quad C_T = 0.035 \text{ for steel frame}$$

$$T = 1.4 (0.035) (33)^{\frac{3}{4}} = 0.67 \text{ seconds}$$

B.2.3 Design Force

The base shear design force is determined for the transverse direction in Table B.2-1 following the 1985 UBC, Chapter 23, Section 2312 procedure.

B.2.4 Distribution of Forces to Frames

The resulting base shear is distributed to the roof and floors to obtain the story forces as shown in Table B.2-2. Then the story forces are distributed to the frames based upon their relative rigidity. In the transverse direction, there are three identical moment resisting frames and in the longitudinal direction there are two K-braced frames, Figure B.1-1.

Because of symmetry there is no "calculated" torsion. However, the "accidental" torsion is the story force times the nominal eccentricity of 5% of the maximum building dimension. The resulting torsional shear is distributed to each frame based on the relative rigidities of the frames, see Table B.2-3.

These total forces, i.e. direct shear plus torsional shear, are then used to size the frames using the Portal Method. Then the preliminary sizes are checked to see if they meet the stress and drift limits of the 1985 UBC using a static force computer analysis.

B.2.5 Checking Transverse Direction, Frame 4

Force distribution to the frame 4

Roof	:	$0.35 \times 58 = 20$ kips
3rd floor	:	$0.43 \times 58 = 25$ kips
2nd floor	:	$0.22 \times 58 = 13$ kips

Figure B.2-1(a) shows the frame model that was used in the static analysis to determine beams and columns internal forces. Figure B.2-1(b) shows the computer results of the analysis. The moments, shears, and axial forces are shown for a load combination 1/1.33 (DL + LL + E).

The stress check for beams are shown in Table B.2-4. The working stress approach is used to check the beams where the elastic moment capacity (M_c) is compared against the moment demand (M_d) using AISC specification. Beams are Grade 36.

The stress check for columns are performed using two approaches: working stress method and strength method.

A. Elastic Approach: (working stress method)

from computer analysis:	exterior column	P = 83	kips
		M = 1108	kip-in
	interior column	P = 147	kips
		M = 1385	kip-in

$$\text{Stability equation AISC spec. (1.6-1a)} \quad \frac{f_a}{F_a} + \frac{C_{\max}}{\left[-\frac{f_a}{F_{ex}} \right]} \frac{f_{bx}}{F_a} < 1$$

$$\text{Stress equation AISC spec. (1.6-1b)} \quad \frac{f_a}{0.6 F_a} + \frac{f_{bx}}{F_{bx}} < 1$$

**Table B.2-2, Transverse Direction
Distribution of Base Shear to Stories**

Floor Level	hx ft.	Wx K.	Wx.hx Kip-ft	$\frac{W_x h_x}{\sum W_x h_x}$	Fx K.	Vx K.	DOTM Kip-ft	OTM
R	33	375	12,375	0.35	61			
3	22	707	15,554	0.43	75	61	671	671
2	11	707	7,777	0.22	39	136	1,496	2,167
1						175	1,925	4,092
Σ		1,789	35,706	1.00	175			

Table B.2-3, Total Force per Frame

Frame	Direct Shear	Torisonal Shear	Design Shear	Force per frame (Kips)
1	0.33	0.03	0.36 Ft	$0.36 \times 175 = 63$
4	0.33	0.00	0.33 Ft	$0.33 \times 175 = 58$
7	0.33	0.03	0.36 Ft	$0.36 \times 175 = 63$

Ft, the base shear force in the transverse direction

Figure B.2-1(a)

Figure B.2-1(b)

Free body diagram showing plastic hinge formation in all beams under ultimate loading conditions.

$$P_{TOTAL} = 1.3 P_{D+L} + \sum \frac{2M_{PB}}{l}$$

$$M_{TOTAL} = 0.55 M_{PB3}$$

$$P_{TOTAL} = 1.3 P_{D+L}$$

$$M_{TOTAL} = M_{col.} = M_{PB3}$$

Figure B.2-2

B. Strength Approach:

criteria: -strong columns weak beam frame
 -assume plastic hinge formation in all beams
 while columns remain elastic

Level	Beam	M_p kip-in	$V=2M_p/L$ kips
R	W14 × 34	1965	14
3	W18 × 55	4032	28
2	W18 × 60	4428	31

			73

-Exterior Column

(Free body diagram, see Figure B.2-2)

$$\begin{aligned} M &= 0.55 M_p \text{ (beam)} = 0.55 \times 4428 = 2436 \text{ kip-in} \\ P &= 1.3 (D+L) + 2M_p/L = 1.3 (67+23) + 73 = 165 \text{ kips} \end{aligned}$$

(AISC specifications, section 2.1)

$$\begin{aligned} M &= 1.3 (D+L+E) = \text{(computer results)} = 1916 \text{ kip-in} \\ P &= 1.3 (D+L+E) = \text{(computer results)} = 144 \text{ kips} \end{aligned}$$

- Interior Column

(Free body diagram, see Figure B.2-2)

$$\begin{aligned} M &= 1.0 M_p \text{ (beam)} = 1.0 \times 4428 = 4428 \text{ kip-in} \\ P &= 1.3 (D+L) + 2M_p/L = 1.3 (46+50) = 255 \text{ kips} \end{aligned}$$

(AISC) specifications, section 2.1)

$$\begin{aligned} M &= 1.3 (D+L+E) = \text{(computer results)} = 2395 \text{ kip-in} \\ P &= 1.3 (D+L+E) = \text{(computer results)} = 255 \text{ kips} \end{aligned}$$

-Design forces: exterior column

$$\begin{aligned} M &= 2436 \text{ kip-in} \\ P &= 165 \text{ kips} \end{aligned}$$

interior column

$$\begin{aligned} M &= 4428 \text{ kip-in} \\ P &= 255 \text{ kips} \end{aligned}$$

Stability equation AISC spec. (2.4-2)

$$\frac{P}{P_{cr}} + \frac{C_{mx}M}{(1 - \frac{P}{P_e}) M_m} < 1$$

Stress equation AISC spec. (2.4-3) $\frac{P}{P_y} + \frac{M}{1.18 M_p} < 1$

Table B.2-5 shows comparison between the allowable drift ratio and the drift ratios obtained from the computer analysis, using the 1985 UBC, Chapter 23, Section 2312(b) criteria.

Table B.2-4, Beams Check

Level	Section	Sx in. ³	L _n in.	L _c in.	F _{bx} ksi	Mc kip-in	M _d
R	W14 × 34	54.6	6.0	7.1	24	1310	897
3	W18 × 55	112.0	6.0	7.9	24	2688	2116
2	W18 × 60	123.0	6.0	8.0	24	2952	2354

Table B.2-5, Columns Check

Description			Exterior Column Level 1	Interior Column Level 1
Column section			W14 × 48	W14 × 68
P	Axial Load (working stress check)	kips	83	148
M	Moment (working stress check)	kip-in.	1108	1385
P	for strength check	kips	165	255
M	for strength check	kips-in.	2436	4428
A	cross sectional area	in. ²	14.10	20.00
S _x	section modulus	in. ³	70.30	103.00
Z _x	plastic modulus	in. ³	78.40	115.00
r _x	radius of gyration	in.	5.85	6.01
r _y	radius of gyration	in.	1.91	2.46
I _x	moment of inertia	in. ⁴	485	723
K _y			1.00	1.00
G _x bottom			1.00	1.00
G _x top			2.21	1.87
K _x			1.47	1.43
L	unbraced length	in.	119	119
(KL/r) _x			29.90	28.31
(KL/r) _y			62.30	48.37
f _a		ksi	5.89	7.40
f _{bx}		ksi	15.76	13.45
F _a		ksi	22.32	24.60
F _{bx}		ksi	30.00	33.00
F' _{ex}		ksi	167.01	186.27
f _a /F _a			0.26	0.30
f _{bx} /F _{bx}			0.53	0.41
C _{mx}			0.85	0.85
C _{mx} • f _{bx} /(1 - f _a /F' _{ex})/F _{bx}			0.46	0.46
f _a /F _a + C _{mx} • f _{bx} /(1 - f _a /F' _{ex})/F _{bx}			0.73	0.66
f _a /0.6F _y + f _{bx} /F _{bx}			0.72	0.65
Strength Approach:				
P _{cr}		kips	535.02	836.47
P _y		kips	705.00	1000.00
P _e		kips	4513.36	7140.21
M _p		kips-in	3920.00	5750.00
M _m /M _p			0.93	1.00
P/P _{cr} + C _{mx} • M/(1 - P/P _e)/M _m			0.90	0.98
P/P _y + M/1.18/M _p			0.76	0.91

B.3 DESIGN USING UBC-88

B.3.1 General

The base shear coefficient is calculated using the 1988 Uniform Building Code (UBC) procedure, and the resulting base shear in each direction is distributed to the stories. Then the story forces are distributed to the frames in direct relation to each frame stiffness. The stresses in beams are checked using working stress criteria. While column stresses are checked using both working stress and ultimate stress (strength) approaches.

B.3.2 Base Shear

The base shear equation of UBC-88, Chapter 23, Section 2312(e)2 is $V = \frac{ZIC}{R_w} W$

where

Z	=	0.40	Seismic Zone Factor (Zone 4)
I	=	1.25	Importance Factor (essential facility)
R_w	=	12	Structural System Coefficient (SMRSF)
S	=	1.2	Site Coefficient (Soil profile S2)
C	=	$\frac{1.25}{T^{2/3}} S$	Numerical Coefficient C

C need not be greater than 2.75

C/R_w must be greater than 0.075

$$\Rightarrow C = \frac{1.25}{T^{2/3}} (1.2) = \frac{1.5}{T^{2/3}}$$

$$\Rightarrow V = 0.0417 CW \quad \text{the base shear formula}$$

Building period

1. *Method A:* The period is calculated using UBC-88, Chapter 23, Section 2312, Equation 12-3

$$T = C_t (h_n)^{3/4}$$

where

$$C_t = 0.035$$

$$h_n = 33 \text{ ft.}$$

$$T = 0.035 (33)^{3/4} = 0.48 \text{ sec.}$$

Check limits:

$$C_A = \frac{1.5}{(0.48)^{2/3}} = 2.44 < 2.75 \quad \text{OK}$$

$$\frac{C_A}{R_w} = \frac{2.44}{12} = 0.203 > 0.075 \quad \text{OK}$$

This method is conservative, upper bound.

2. *Method B:* The period is calculated using Equation 12-5.

$$T = 2\pi \sqrt{\frac{\sum W_i \delta_i^2}{g \sum f_i \delta_i}} \approx 0.25 \sqrt{\frac{\Delta}{C_1}}$$

Teal's formula (AISC Engineering Journals, 2nd and 4th Quarters, 1975).

$$\Delta \approx \frac{2}{3} \Delta_{\text{allow}} = \frac{2}{3} \left[\frac{0.04}{R_w} \right] h_n$$

where Δ is the deflection at roof

h_n Total building height

$$\Delta = \frac{2}{3} \left(\frac{0.04}{12} \right) (33 \times 12) = 0.88 \text{ inches}$$

$$C_1 = \frac{V}{W} = \frac{ZIC}{R_w} = \frac{0.04 (1.25)}{12} \left[\frac{1.5}{T^{2/3}} \right] = \frac{0.0625}{T^{2/3}}$$

Substituting Δ and C_1 values in Teal's formula, it follows that

$$T = 0.25 \sqrt{\frac{0.88 T^{2/3}}{0.0625}} \quad \text{and}$$

$$T = 0.91 \text{ seconds} \quad \text{then}$$

$$C_B = \frac{1.5}{(0.91)^{2/3}} = 1.6 < 2.75 \quad \text{ok}$$

$$\frac{C_B}{R_W} = \frac{1.6}{12} = 0.133 > 0.075 \quad \text{ok}$$

For stress calculation: The value of C shall be not less than 80 percent of the value obtained by using T from Method A.

$$C = 0.80 C_A = 0.80 (2.44) = 1.95$$

For drift calculation: The design lateral forces used to determine the calculated drift may be derived from a value of C based on the period derived from Method B neglecting the 80 percent limitation. Section 2312 (e) 8.

$$\text{use } C_B = 1.60$$

Base Shear

$$V = 0.0417 C W$$

$$V (\text{drift}) = 0.0417 (1.60) W = 0.0667 W$$

$$V (\text{stress}) = 0.0417 (1.95) W = 0.0813 W$$

$$\text{ratio } V(\text{stress}) / V (\text{drift}) = 0.0813/0.0667 = 1.22$$

$$V(\text{stress}) = 0.0813 \times 1789 = 145 \text{ kips}$$

Distribution of Base Shear to Frames

The resulting base shear is distributed to the roof and floors to obtain the story forces as shown in Tables B.3-1(a). The story forces are distributed to the frames based upon their relative rigidity. Then the "accidental" torsional shear is added to each frame force, see Table B.3-1(b).

These total forces, i.e. direct shear plus torsional shear, are then used to size the frames using the Portal Method. Then the preliminary sizes are checked to see if they meet the stress and drift limits of the 1988 UBC using a static force computer analysis.

B.3.3 Checking Transverse Direction, Frame 4

Figure B.3-1(a) shows the frame model that was used in the static analysis to determine beam and columns internal forces. Figure B.3-1(b) shows the computer results of the analysis. The moments, shears, and axial forces are shown for a load combination 1/1.33 (DL + LL + E).

Checking Building Period

For stress analysis, we used coefficient $C = 1.95$. That corresponds to a period of

$$T = \left(\frac{1.5}{C}\right)^{3/2} = 0.67 \text{ sec.}$$

but from the results of the analysis: Equation 12-5 UBC-88,

$$T = 2\pi \sqrt{\frac{375 \times 0.947^2 + 707 \times 0.660^2 + 707 \times 0.289^2}{3 \times 386.4 (17 \times 0.947 + 21 \times 0.660 + 11 \times 289)}} = 0.85 \text{ sec.}$$

Table B.2-6 Drift Check, Transverse Direction

Level	Deflection inches	Drift inches	Drift Ratio (Drift $\times$ 1/K)/h	Allowable Drift Ratio
R	1.020	0.303	0.0035	0.005
3	0.717	0.402	0.0045	0.005
2	0.313	0.315	0.0038	0.005

**Table B.3-1(a) Transverse Direction
Distribution of Base Shear to Stories**

Floor Level	h_x ft.	W_x K.	$W_x \cdot h_x$ Kip.ft	$\frac{W_x h_x}{\sum W_x h_x}$	F_x K.	V_x K.	DOTM Kip.ft	OTM Kip.ft
R	33	375	12,375	0.35	51			
3	22	707	15,554	0.43	62	51	561	561
2	11	707	7,777	0.22	32	113	1,243	1,804
1		1,789	35,706	1.00	145	145	1,595	3,399

Table B.3-1(b) Total Force per Frame

Floor Level	Story Force	Frame 1	Frame 4	Frame 7
R	51	$0.36 \times 51 = 19$	$0.33 \times 51 = 17$	$0.36 \times 51 = 19$
3	62	$0.36 \times 62 = 23$	$0.33 \times 62 = 21$	$0.36 \times 62 = 23$
2	32	$0.36 \times 32 = 12$	$0.33 \times 32 = 11$	$0.36 \times 32 = 12$

Figure B.3-1(a)

Figure B.3-1(b)

$$\begin{aligned} C(\text{drift}) &= \frac{1.5}{(0.85)^{2/3}} = 1.67 \\ V(\text{stress})/V(\text{drift}) &= 1.95/1.67 = 1.17 \end{aligned}$$

B. Strength Approach:

- criteria: - strong columns weak beam frame
 - assume plastic hinge formation in all beams while columns remain elastic.

Level	Beam	M_p kip-in	$V=2M_p/L$ kips
R	W14 × 30	1702	12
3	W18 × 50	3636	25
2	W18 × 55	4032	28
			----- 65

- Exterior Column;

(Free body diagram, see Figure B.2-2)

$$\begin{aligned}
 M &= 0.55 M_p (\text{beam}) = 0.55 \times 4032 = 2218 \text{ kip-in} \\
 P &= 1.3 (D+L) + 2M_p/L = 1.3 (67 + 23) + 65 = 157 \text{ kips}
 \end{aligned}$$

(AISC specifications, section 2.1)

$$\begin{aligned}
 M &= 1.3 (D+L+E) = (\text{computer results}) = 1640 \text{ kip-in} \\
 P &= 1.3 (D+L+E) = (\text{computer results}) = 139 \text{ kips}
 \end{aligned}$$

- Interior Column

(Free body diagram, see Figure B.2-2)

$$\begin{aligned}
 M &= 1.0 M_p (\text{beam}) \\
 P &= 1.3 (D+L) + 2M_p / L
 \end{aligned}$$

(AISC specifications, section 2.1)

$$\begin{aligned}
 M &= 1.3 (D+L+E) = (\text{computer results}) = 2145 \text{ kip-in} \\
 P &= 1.3 (D+L+E) = (\text{computer results}) = 255 \text{ kips}
 \end{aligned}$$

-Design forces: exterior column

$$\begin{aligned}
 M &= 2218 \text{ kip-in} \\
 P &= 157 \text{ kips}
 \end{aligned}$$

interior column

$$\begin{aligned}
 M &= 4032 \text{ kips-in} \\
 P &= 255 \text{ kips}
 \end{aligned}$$

Stability equation

$$\frac{P}{P_u} + \frac{C_{mx} M}{(1 - \frac{P}{P_e}) M_m} < 1$$

AISC spec. (2.4-2)

Stress equation $\frac{P}{P_y} + \frac{M}{1.18 M_p} < 1$

AISC spec. (2.4-3)

Table B.3-3 shows the results of the column check.

Table B.3-2, Beams Check

Level	Section	Sx in. ³	L _n in.	L _c in.	F _{bx} ksi	Mc kip-in	Md
R	W14 × 30	42.0	6.0	7.1	24	1008	855
3	W18 × 50	88.9	6.0	7.9	24	2134	2011
2	W18 × 55	98.3	6.0	7.9	24	2360	2216

Table B.3-3, Columns Check

Description			Exterior Column Level 1	Interior Column Level 1
Column section			W14 × 43	W14 × 68
P	Axial Load (working stress check)	kips	80	148
M	Moment (working stress check)	kip-in.	943	1240
P	for strength check		157	255
M	for strength check		2218	4032
A	cross sectional area	in.	12.60	20.00
S _x	section modulus	in ³	62.70	103.00
Z _x	plastic modulus	in ³	69.60	115.00
r _x	radius of gyration	in.	5.82	6.01
r _y	radius of gyration	in.	1.89	2.46
I _x	moment of inertia	in ⁴	428	723
K _y			1.00	1.00
G _x bottom			1.00	1.00
G _x top			2.21	1.87
K _x			1.47	1.43
L unbraced length	in.		119	119
(KL/r) _x			30.06	28.31
(KL/r) _y			62.96	48.37
f _a	ksi		6.35	7.40
f _{bx}	ksi		15.04	12.04
F _a	ksi		22.21	24.60
F _{bx}	ksi		30.00	33.00
F' _{ex}	ksi		165.30	186.27
f _a /F _a			0.29	0.30
f _{bx} /F _{bx}			0.50	0.36
C _{mx}			0.85	0.85
C _{mx} * f _{bx} /(1 - f _a /F' _{ex})/F _{bx}			0.44	0.32
f _a /F _a + C _{mx} * f _{bx} /(1 - f _a /F' _{ex})/F _{bx}			0.73	0.62
f _a /0.6F _y + f _{bx} /F _{bx}			0.71	0.61
Strength Approach:				
P _{cr}	kips		475.64	836.47
P _y	kips		630.00	1000.00
P _e	kips		3991.96	7140.21
M _p	kip-in		3480.00	5750.00
M _m /M _p			0.93	1.00
P/P _{cr} + C _{mx} * M/(1 - P/P _e)/M _m			0.94	0.92
P/P _y + M/1.18/M _p			0.79	0.85

Defection Check:

Table B.3-4 shows the allowable drift ratio and the drift ratios obtained from the computer analysis, using the 1988 UBC, Chapter 23; Section 2312-h criteria.

$$\text{Allowable Drift Ratio} = \min (0.04/R_w, 0.005) = 0.0033$$

Special requirement of UBC-88

1. Using the UBC-88, Section 2722 (d), the following criteria must be satisfied for columns.

Exterior column only: (Since $E=0$ for interior column)

$$P = 1.0 \text{ DL} + 0.7 \text{ LL} + 4.5 E < 1.7 F_a A$$

$$P = 1.0 \times 67 + 0.7 \times 23 + 4.5 \times 17.5 = 162 < 475 \text{ kips OK}$$

2. Strength Ratio of Columns to Girder; Using the UBC - 88, Section 2722 (f) 5,

$$\frac{\sum Z_c (F_{yc} - F_a)}{\sum Z_b F_{by}} > 1.0 \quad \text{UBC(22-3)}$$

Exterior column	f_a	=	6.35 ksi
	Z_c	=	69.6 qu. in
	Z_b	=	112 qu. in

$$\frac{2(69.6)(50 - 6.35)}{112 \times 36} = 1.51 > 1.0 \quad \text{OK}$$

Interior column	f_a	=	7.4 ksi
	Z_c	=	115 ksi and $Z_b = 112 \text{ ksi}$

$$\frac{2(115)(50 - 7.4)}{2(112)(36)} = 1.22 > 1.0 \quad \text{OK}$$

Table B.3-4, Drift Check, Transverse Direction

Level	Deflection from stress calc. (d)	Deflection (d/1.17)	Drift inches	Drift Ratio	Allowable Drift Ratio
R	0.947"	0.811"	0.245	0.0019	0.0033
3	0.660"	0.566"	0.318	0.0024	0.0033
2	0.289"	0.248"	0.248	0.0019	0.0033

B.4 DESIGN USING THE NAVY'S TECHNICAL MANUAL PROCEDURE

B.4.1 Site Seismicity

Two levels of earthquake motion are considered. At the first level, the structure is designed to remain elastic, or "near elastic", and the second level the structure responses inelastically but remains functional. The site response spectra are developed in accordance with the procedure of TM 5-809-10-1/NAVFAC P-355.1/AFM 88-3, Chapter 13, Section A, Technical Manual, "Seismic Design Guidelines for Essential Buildings", referred to herein as the Technical Manual. Assuming the building is located in seismic zone 4, (UBC Zoning), and following the procedure of the Technical Manual, Chapter 3, Section III, Figures 3-40 and 3-42, and Table 3-4, A_a and A_v values for earthquakes EQ-I and EQ-II are obtained and are shown in Table B.4-1.

Assume also that the soil underlying the building is type S2, i.e. $S_i=1.2$ (The Technical Manual, Table 3-6). For structural steel system the damping values and the damping adjustment factors corresponding to EQ-I and EQ-II are obtained and are shown in Table B.4-2 (as per the Technical Manual, Tables 3-7 and 4-1).

Response spectra obtained from EQ-II represents a more severe earthquake and then the structure designed to these criteria should be checked primarily to satisfy drift limitations. The structure designed to a base shear obtained from EQ-I spectra should be checked using the strength approach since EQ-I represents first yield of the structure. In other words, while the structure designed for UBC criteria is checked using working stress method, the structure designed for EQ-I and EQ-II is checked using the ultimate stress method.

The governing equations for EQ-I and EQ-II response spectra are (The Technical Manual, Eq's. 3-27 and 3-28):

For $T < 4.0$ sec.

$$S_a = 1.22 \times A_v \times S_i \times \text{Damp. Adj. Factor} / T \quad (g)$$

$$\text{but } < 2.5 \times \text{Damp. Adj. Factor} \times A_a \quad (g)$$

It follows that

EQ-I: $S_a = 1.22 \times 0.20 \times 1.2 \times 1.17 / T = .3426/T \quad (g)$
but $S_a < 0.585 \quad g$

EQ-II: $S_a = 1.22 \times 0.45 \times 1.2 \times 0.90 / T = .5929/T \quad (g)$
but $S_a < 1.0125 \quad g$
 $(EQ-II)_{\max} / (EQ-I)_{\max} = 1.73$

Figure B.4-1 shows response spectra for EQ-I and EQ-II.

B.4.2 Response Spectra Analysis / Transverse Direction

B.4.2.1 General

The base shear coefficient and the members sizes obtained using UBC-85 are assumed to be the initial trial design. The EQ-I design spectra is compared to the static load coefficient (ZICS)

T, building period	$S_a(g)$	ZICS	$S_a/ZICS$
0.67 sec.	0.511	0.146	3.5

This ratio is greater than 1.7 (this is the load factor necessary to bring up UBC base shear to the ultimate load criteria), therefore, the structure designed for UBC-85 had to be modified for higher lateral force level. The member sizes that were obtained for use in response spectra analysis are shown in Figure B.4-2.

For dynamic analysis the transverse frames on lines 1, 4, and 7 are identical. Forces at the floors are distributed to frames in proportion to their relative rigidity. While there is no "calculated" torsion in the building, an "accidental" torsional shear is distributed to perimeter frames in both directions. The computer models that were developed for frame 4, is shown in Figure B.4-2. One third of the mass at each floor is carried by frame 4.

B.4.2.2 Natural Frequencies and Mode Shapes Analysis

Using the computer program ETABS the natural periods (T_m) and mode shapes (Φ_{xm}) are obtained in Table B.4-3. Table B.4-4 shows the modal analysis, i.e. modal story participation factors (PF_{xm}) and modal base shear participation factors (α_m), using the Technical Manual procedure, equations 4-1 and 4-2:

$$PF_{xm} = (\sum M_x \Phi_{xm} / \sum M_x \Phi_{xm}^2) \Phi_{xm} \quad (4-1)$$

$$\alpha_m = (\sum M_x \Phi_{xm})^2 / (\sum M_x \sum M_x \Phi_{xm}^2) \quad (4-2)$$

B.4.2.3 EQ-I Response

Table B.4-5 shows the modal base shears ($V_m = \alpha_m S_{am}W$, equation 4-4). Table B.4-6 shows modal story lateral forces (F_{xm} , The Technical Manual, eq. 4-3), modal shears and moments, and modal deflections and drifts (δ_{xm} , D_{xm} , The Technical Manual, eq. 4-5). Then the combinations of the modal values are obtained by taking the Square-Root-of-the-Sum-of-the-Squares (SRSS) of the values of all three modes.

B.4.3.2.4 EQ-II Response

The structural response to EQ-II is in the nonlinear range, therefore the period is assumed to lengthen by $\sqrt{x}$, where x represents the Inelastic Demand Ratio, IDR, of the critical elements in the frame (The Technical Manual, Table 4-2). In this example the x value is 1.25 for columns in a steel moment-resistant frame in essential facility. The same mode shapes obtained for EQ-I analysis is assumed to be valid for EQ-II analysis.

Table B.4-1, Aa & Av Values for EQ-I and EQ-II

Prob. of Exceedance	ATC-3-06 10% in 50 years	EQ-I 50% in 50 years	EQ II 10% in 100 years
Aa	0.40 g	0.20 g	0.45 g
Av	0.40 g	0.20 g	0.45 g

Table B.4-2, Damping Adjustment Factors

	Damping	Damp.Adj. Factor
EQ-I	3 %	1.17
EQ-II	7 %	0.90

Figure B.4-1, Design Response Spectra for EQ-I and EQ-II

**Table B.4-3, The Periods and the Mode Shapes
for the Transverse Direction**

Level	Mass $\frac{K}{ft. \text{ sec}^2}$	Mode 1		Mode 2		Mode 3	
		Φ_{x1}	T_1	Φ_{x2}	T_2	Φ_{x3}	T_3
R	3.88	1.2346	.457	1.1648	.167	.4614	.094
3	7.24	.8404	sec	-.5816	sec	-.7806	sec
2	7.24	.3663		-.7695		.9665	

Σ 18.36

Table B.4-4, Modal Participation Factors, EQ-I

	Mode 1		Mode 2		Mode 3	
	$M_x \Phi_{x1}$	$M_x \Phi_{x1}^2$	$M_x \Phi_{x2}$	$M_x \Phi_{x2}^2$	$M_x \Phi_{x3}$	$M_x \Phi_{x3}^2$
	4.790	5.914	4.519	5.264	1.790	0.826
	6.085	5.114	-4.211	2.449	-5.651	4.411
	2.652	0.972	-5.571	4.286	6.997	6.763
Σ	13.527	12.000	-5.262	12.000	3.136	12.000
PF_m	1.392		-0.511		0.121	
PF_{3m}	0.947		0.255		-0.204	
PF_{2m}	0.413		0.337		0.253	
α_m	0.831		0.126		0.045	

Table B.4-5, Modal Base Shears, EQ-I

		Mode 1	Mode 2	Mode 3
T_m	sec	0.457	0.167	0.094
S_{am}	g's	0.585	0.585	0.585
$\alpha_m S_{am}$		0.486	0.074	0.026
$V_m = \alpha_m S_{am} (\Sigma mg)$	Kips	287.3	43.5	15.4

Table B.4-6, EQ-I Response

Mode	Level	PF _{xm}	F _{xm} K.	V _{xm} K.	DOTM _{xm} Kip-ft	OTM _{xm} Kip-ft	δ _{xm} in.	D _{xm} in.
1	R	1.392	101.7	101.7	1242	000	1.668	.532
	3	.947	129.2	230.9	2540	1242	1.135	.640
	2	.413	56.3	287.3	3065	3782 6847	0.495	.495
2	R	-.511	-37.3	-37.3	-456	000	-.081	.122
	3	.255	34.8	2.5	28	-456	.041	.013
	2	.337	46.0	43.5	464	-484 -20	.054	.054
3	R	.121	8.8	8.8	108	000	.006	.016
	3	-.204	-27.8	-19.0	-209	108	-.010	.036
	2	.253	34.5	15.4	165	-101 63	.013	.013
S	R		108.7	108.7	1327	000	1.670	.547
R	3		136.7	231.7	2550	1327	1.136	.641
S	2		80.5	290.9	3104	3814	.498	.498
S						6848		

Table B.4-7, Modal Base Shears, EQ-II

		Mode 1	Mode 2	Mode 3
$T_m \sqrt{1.25}$	sec	0.511	0.186	0.105
S_{am}	g's	1.013	1.013	1.013
$\alpha_m S_{am}$		0.841	0.127	0.045
$V_m = \alpha_m S_{am} (\sum mg)$	Kips	497.2	75.2	26.7

Table B.4-8, EQ-II Response

Mode	Level	PF _{xm}	F _{xm} K.	V _{xm} K.	DOTM _{xm} Kip-ft	OTM _{xm} Kip-ft	δ_{xm} in.	D _{xm} in.
1	R	1.392	176.1	176.1	2150	000	3.608	1.152
	3	.947	223.6	399.7	4397	2150	2.456	1.385
	2	.413	97.5	497.2	5305	6546 11851	1.071	1.071
2	R	-.511	-64.6	-64.6	-789	000	-.176	.264
	3	.255	60.2	-4.4	-49	-789	.088	.028
	2	.337	79.6	75.2	803	-837 -35	.116	.116
3	R	.121	15.3	15.3	186	000	.013	.036
	3	-.204	-48.2	-32.9	-362	186	-.022	.050
	2	.253	59.6	26.7	285	-362 285	.028	.028
S	R		188.2	188.2	2298	000	3.612	1.182
R	3		236.5	401.1	4412	2298	2.458	1.387
S	2		139.3	503.5	5373	6602	1.077	1.077
S						11851

Figure B.4-2

EQ-I : $1.2D + L \pm E$ (the Tech. Manual Eq. 4-6)

EQ-II : $1.0D + .25L \pm E$ (the Tech. Manual Eq. 4-9)

Figure B.4-3

Table B.4-7 shows the calculations performed to obtain the modal base shear (V_m , The Technical Manual, eq. 4-4). Then Table B.4-8 shows modal story lateral forces (F_{xm} , The Technical Manual, eq. 4-5). Then SRSS rule is applied to obtain modal combination values.

B.4.3 Checking Transverse Direction for EQ-I & EQ-II

The computer results for the static analysis for frame 4 are shown in Figure B.4-3. These results are obtained using the static lateral loads obtained from response spectra analysis for EQ-I and EQ-II.

The stress check for beams is performed using the strength approach, i.e. the ultimate strength method,, where the moment demand is compared against the ultimate moment capacity of the beams. These calculations are shown in Table B.4-9. For the DMRSF in the transverse direction, 20% of the beams at any story are allowed to exceed the flexural strength requirements by up to 25% for EQ-I loading, (The Technical Manual, Para. 4-3e(1)). While for EQ-II loading, the Inelastic Demand Ratio, IDR, (demand/capacity) is allowed to reach 2.0, (The Technical Manual, Table 4-2).

The stress check for columns is performed using also the ultimate strength approach. This was done in the following ways:

EQ-I

- criteria: 1. Strong columns weak beams frame. Assume plastic hinge formation in all beams while columns remain elastic. See figure B.2-2. Use equations 2.4-2 and 2.4-3, AISC Spec. 8th edition.
2. Check the computer output using equations 1.6-1a and 1.6-1b, AISC Spec. 8th edition, and allow an increase of 1.7 of the allowable stresses.

Level	Beam	M_p kip-in	$V=2M_p/L$ kips
R	W18 × 50	3728	26
3	W21 × 122	11970	83
2	W21 × 132	14280	99

			208

1. Exterior column;

$$\begin{aligned} M &= 0.55 M_p (\text{beam}) = 0.55 \times 14280 = 7854 \text{ kip-in} \\ P &= 1.3 (D+L) + 2M_p/L = 1.3 (93) + 208 = 329 \text{ kips} \end{aligned}$$

Interior column;

$$\begin{aligned} M &= 1.0 M_p (\text{beam}) = 1.0 \times 14280 = 14280 \text{ kip-in} \\ P &= 1.3 (D+L) + 2M_p/L = 1.3 (154 + 50) = 265 \text{ kips} \end{aligned}$$

Use equations: AISC 8th edition Spec. (2.4-2 and 2.4-3).

$$\frac{P}{P_{cr}} + \frac{C_{mx} M}{(1 - \frac{P}{P_c}) M_m} < 1, \quad \frac{P}{P_y} + \frac{M}{1.18 M_p} < 1$$

2. From computer output:

Using load combination of (1.2 DL + LL + E), according to The Technical Manual, equation 4-6

$$\begin{aligned} \text{Design forces: exterior column} \quad M &= 6660 \text{ kip-in} \\ P &= 223 \text{ kips} \end{aligned}$$

$$\begin{aligned} \text{interior column} \quad M &= 10426 \text{ kip-in} \\ P &= 235 \text{ kips} \end{aligned}$$

Use equations: AISC 8th edition Spec. (1.6-1a and 1.6-1b)

$$\frac{f_a}{1.7 F_a} + \frac{C_{mx} f_{bx}}{(1 - \frac{f_a}{1.7 F_{ex}}) 1.7 F_{bx}} < 1, \quad \frac{f_a}{0.6 (1.7 F_y)} + \frac{f_{bx}}{1.7 F_{bx}} < 1$$

Table B.4-10 shows columns check calculations for EQ-L

Table B.4-9, Beams Check

Level	Size	Zx qu.in	EQ-I			EQ-II			
			Md Kip-in	Mc	Md/Mc	Md Kip-in	Mc	Md/Mc	IDR
R	W18x50	101	2750	3636	0.76	3728	3636	1.03	2.0
3	W21x122	307	8117	11052	0.73	11970	11052	1.08	2.0
2	W18x132	333	9595	11988	0.80	14280	11988	1.19	2.0

Table B.4-10, Columns Check for EQ-I

Description			Exterior Column Level 1	Interior Column Level 1
Column section			W14 x 132	W14 x 193
P	Axial Load (Technical Manual)	kips	223	235
M	Moment (Technical Manual)	kip-in.	6660	10426
P	for strength check (AISC)		329	265
M	for strength check (AISC)		7854	14280
A	cross sectional area	in.	38.80	56.80
S _x	section modulus	in ³	209.00	310.00
Z _x	plastic modulus	in ³	234.00	355.00
r _x	radius of gyration	in.	6.28	6.50
r _y	radius of gyration	in.	3.76	4.05
I _x	moment of inertia	in ⁴	1530	2400
K _y			1.00	1.00
G _x bottom			1.00	1.00
G _x top			2.19	1.72
K _x			1.48	1.41
L unbraced length		in.	119	119
(KL/r) _x			28.04	25.81
(KL/r) _y			31.65	29.38
fa		ksi	5.75	4.14
fbx		ksi	31.87	33.63
1.7*Fa		ksi	45.81	46.29
1.7*Fbx		ksi	56.10	56.10
1.7*F'ex		ksi	322.78	380.97
fa /1.7Fa			0.13	0.09
fbx/1.7Fbx			0.57	0.60
Cmx			0.85	0.85
Cmx*fbx/(1 - fa/1.7/F'ex)/1.7/Fbx			0.49	0.52
fa/1.7/Fa + Cmx*fbx/(1 - fa/1.7/F'ex)			0.62	0.60
fa/1.7/0.6F _y + fbx/1.7/Fbx			0.68	0.68
Strength Approach:				
Pcr		kips	1777.40	2629.21
P _y		kips	1940.00	2840.00
P _e		kips	14119.89	24397.24
M _p		kips-in	11700.00	17750.00
Mm/M _p			1.00	1.00
P/Pcr + Cmx*M/(1 - P/Pe)/Mm			0.77	0.79
P/P _y + M/1.18/M _p			0.74	0.78

EQ-II

For column check for EQ-II loading, The Technical Manual procedure, Figure 4-2, is followed using the computer output for a load combination of (1.0 DL + 0.25 LL + E), according to equation 4-7 of The Technical Manual.

Design forces: exterior column	M	=	11152	Kip-in
	P	=	274	Kips
interior column	M	=	18013	Kip-in
	P	=	167	Kips

The equations of Figure 4-2 of The Technical Manual are:

$$(1) \frac{M_x}{M_{P_{cx}}} < IDR, \quad (2) \frac{C_{mx} M_x}{M_{U_{cx}}} < IDR \quad \text{where}$$

$$M_{P_{cx}} = 1.18 M_{P_x} \left(1 - \frac{P}{P_y}\right) \quad \text{and} \quad M_{U_{cx}} = M_{P_x} \left(1 - \frac{P}{P_{cr}}\right) \left(1 - \frac{P}{P_{cx}}\right)$$

Table B.4-11 shows columns check calculations for EQ-II where the Inelastic Demand Ratio IDR is allowed to reach 1.25, (The Technical Manual, Table 4-2).

Table B.4-12 shows drift check for EQ-I and EQ-II following The Technical Manual drift ratio limits, para. 4-3(e) 7(a), and para. 4-4(e) 2(a).

Table B.4-11, Columns Check for EQ-II

Col. GR50	P_d K.	M_d K-in	M_{p_x} K-in	$M_{p_{ex}}$ K-in	$M_{u_{ex}}$ K-in	eq.1	eq.2	IDR
W14x132	272	11152	11700	12140	9704	0.92	0.98	1.25
W14x193	176	18013	17750	19713	16510	0.91	0.91	1.25

Table B.4-12, Drifts Check

Level	EQ-I			EQ-II		
	Drift (in.)	Drift Ratio	Allowable Drift Ratio	Drift (in.)	Drift Ratio	Allowable Drift Ratio
R	0.547	0.0041	0.005	1.182	0.0089	0.010
3	0.641	0.0049	0.005	1.387	0.0105	0.010
2	0.498	0.0039	0.005	1.077	0.0084	0.010

APPENDIX C

BUILDING ON BASE ISOLATORS

C.1 Description Building

The same three-story essential building that was considered in Appendix B is also considered here. The lateral seismic resistance is provided by transverse ductile moment-resisting steel frames (DMRSF) and longitudinal steel braced frames (BSF). The building is located in UBC Seismic Zone No. 4 and the soil profile of the site consists of a dense soil where the depth exceeds 200 feet. Only the transverse direction is considered here.

C.2 Site Seismicity

The Navy's Technical Manual procedure is used to define site seismicity. Two levels of earthquake motion are considered. The EQ-I earthquake ground motion has a 50% chance of being exceeded in a 50 year time period. The second design earthquake EQ-II, the most probable maximum earthquake, has approximately a 10% chance of being exceeded in a 100 year time period.

The site response spectra that are used here are the same as those developed in Appendix B for the design of the fixed base building. For structural steel (DMRSF) the damping values and the adjustment factors corresponding to EQ-I and EQ-II are obtained in table C.1.

Table C.1

	Damping	Damping Factor
EQ-I	3 %	1.17
EQ-II	7 %	0.90

C.3 Analysis 1

Analysis 1 is the analysis used to develop the design and it is consistent with the Technical Manual procedure. A 2-D response spectra analysis is performed here. In order to obtain the initial trial design for the frame and the isolators, the portal method is used to size the members using the Preliminary Design provisions defined in the design criteria, (Equations C-1 thru C-5). Then successive iterations were made to insure that the isolated building period does not exceed 3.0 seconds and is approximately four times the fixed base building period. The dead loads were increased by 10% to account for the effects of vertical accelerations on beams and columns members.

Isolation modeling for the ETABS program was done by inserting a "dummy" story level below the base level of the frame model, and assign the column properties to reflect the isolators' stiffness:

$$\begin{array}{lll} \text{Axial Area} & A & = K_a H / E \\ \text{Shear Area} & A_j & = 0 \\ \text{Moment of Inertia} & I & = \frac{k_{eff} H^3}{3E} \end{array}$$

where H is the dummy story height, E is Young modulus, K_a is the isolator axial stiffness, and k_{eff} is the isolator effective lateral stiffness. This isolator effective lateral stiffness was found to be 3.655 kips/inch.

Using the computer program ETABS the following periods and mode shapes are obtained. (Table C.2).

Periods and Mode Shapes
Table C.2

Level	Mass <u>k.sec2</u> ft.	Mode 1		Mode 2		Mode 3	
		ϕ x1	T ₁	ϕ x2	T ₂	ϕ x3	T ₃
R	3.88	0.8241	2.85	1.3514	0.278	0.7664	0.132
2	7.24	0.8131	sec	0.0688	sec	-0.9958	sec
1	7.24	0.7952		-0.8210		0.5926	
B	0.00	0.7545		-1.0819		1.2823	
Modal Participation Factors		1.2368		-0.0168		0.0045	

C.3.1 EQ-I Response

The structural response to EQ-I is in a near elastic range, therefore, the percent damping is 3. Table C.3 shows the modal base shear (V_m), the total lateral force corresponding to mode m.

Table C.3 also shows modal story lateral forces and modal cumulative story shears. Then the combinations of the modal values are obtained by taking the Square-Root-of-the-Sum-of-the-Squares (SRSS) of the values of all three modes.

C.3.2 EQ-II Response

The structural response to EQ-II is assumed to have the same periods and mode shapes as in the linear EQ-I response. Table C.4 shows the modal story lateral forces and the modal cumulative story shears. Then SRSS rule is applied to obtain modal combination values.

Maximum Modal Story Forces (kips)				
LEVEL	MODE 1	MODE 2	MODE 3	SRSS
R	15.28	-1.66	.25	15.37
2	28.13	-.16	-.62	28.14
1	27.51	1.88	.37	27.58
B	.00	.00	.00	.00
Maximum Modal Cumulative Story Shears				
LEVEL	MODE 1	MODE 2	MODE 3	SRSS
R	15.28	-1.66	.25	15.37
2	43.41	-1.82	-.36	43.45
1	70.92	.06	.00	70.92
B	70.92	.06	.00	70.92

EQ-I Response Spectra Analysis
Table C.3

Maximum Modal Story Forces (kips)				
LEVEL	MODE 1	MODE 2	MODE 3	SRSS
R	26.48	-2.88	.44	26.64
2	48.76	-.27	-1.07	48.77
1	47.68	3.26	.63	47.80
B	.00	.00	.00	.00
Maximum Modal Cumulative Story Shears				
LEVEL	MODE 1	MODE 2	MODE 3	SRSS
R	26.48	-2.88	.44	26.64
2	75.24	-3.15	-.63	75.31
1	122.92	.11	.01	122.92
B	122.92	.11	.01	122.92

EQ-II Response Spectra Analysis
Table C.4

Figure C.1

C.3.3 Checking Member Sizes for EQ-I & EQ-II

The computer results for the response spectra analysis for frame 4 are shown in Figure C.1.

C.3.3.1 Checking Beams

The following table shows beams check in accordance with the procedure of The Technical Manual.

Level	Size	Z_x	EQ-I			EQ-II			IDR
			M_d	M_c	M_d/M_c	M_d	M_c	M_d/M_c	
R	W18x40	78.4	1255	2822	0.43	1078	1822	0.38	2.0
2	W21x93	221	3255	7956	0.41	3502	7956	0.44	2.0
1	W21x111	279	4704	10044	0.47	5890	10044	0.59	2.0

C.3.3.2 Checking Columns

Table C.5 shows column check for EQ-I and EQ-II responses.

EQ-I

- criteria:
1. strong columns weak beams frames assume plastic hinge formation in all beams
 2. apply The Technical Manual Procedure

LEVEL	BEAM	M_p kips.in	$V=2M_p/L$ kips
R	W18x40	2822	20 kips
2	W21x93	7956	55 kips
1	W21x111	10044	70 kips
			----- 145 kips

1. Exterior Column;

$$\begin{aligned}
 M &= 0.55 M_p \text{ (beam)} &= 0.55 \times 10044 &= 5525 \text{ kips.in} \\
 P &= 1.3 (D + L) + 2M_p/L &= 1.3 (97) + 145 &= 271 \text{ kips}
 \end{aligned}$$

Interior Column;

$$\begin{aligned} M &= 1.0 M_p (\text{beam}) = 1.0 \times 10044 = 10044 \text{ kips.in} \\ P &= 1.3 (D + L) + 2M_p/L = 1.3 (170 + 50) = 286 \text{ kips} \end{aligned}$$

2. From computer output
(1.2DL + LL + E)

Design Forces:	exterior column	M	=	2684	kips.in
		P	=	107	kips
	interior column	M	=	3417	kips.in
		P	=	236	kips

EQ-II

-From computer output
(1.0DL + 0.25LL + E)

Design Forces:	exterior column	M	=	4032	kips.in
		P	=	127	kips
	interior column	M	=	5922	kips.in
		P	=	205	kips

checking the columns are performed in Table C.5.

C.3.3.3 Checking Drifts

The following table shows drift ratios check for EQ-I and EQ-II.

Level	EQ-I			EQ-II		
	drift		allowable drift	drift		allowable drift
R	0.13	0.0001	0.005	0.23	0.0017	0.010
2	0.22	0.0017	0.005	0.37	0.0028	0.010
1	0.48	0.0038	0.005	0.83	0.0065	0.010

Beam-Column Check for Analysis - 1
Table C.5

Description			Exterior Column Level 1	Interior Column Level 1
Column section		GR 50	W14 × 99	W14 × 132
P	for EQ-I (Strength Check)	kips	271	286
M	for EQ-I (Strength Check)	kips.in	5,525	10,044
P	for EQ-II (Navy Check)	kips	127	205
M	for EQ-II (Navy Check)	kips.in.	4,032	5,922
A	cross sectional area	in.	29.10	38.80
S_x	section modulus	in ³	157.00	209.00
Z_x	plastic modulus	in ³	173.00	234.00
r_x	radius of gyration	in.	6.17	6.28
r_y	radius of gyration	in.	3.71	3.76
I_x	moment of inertia	in ⁴	1110	1530
K_y			1.00	1.00
G_x bottom			1.00	1.00
G_x top			2.21	1.87
K_x			1.47	1.43
L unbraced length		in.	119	119
$(KL/r)_x$			28.35	27.10
$(KL/r)_y$			32.08	31.65
F_a		ksi	26.89	26.95
F'_{ex}		ksi	185.78	203.38
C_{mx}			0.85	0.85
Strength Approach AISC for EQ-I:				
P_{cr}		kips	1,330	1,777
P_y		kips	1,455	1,940
P_e		kips	10,362	15,125
M_p		kips-in	8,650	11,700
M_m/M_p			1.00	1.00
$P/P_{cr} + C_{mx} * M/(1 - P/P_e)/M_m$			0.76	0.90
$P/P_y + M/1.18/M_p$			0.73	0.87
Navy Criteria for EQ-II:				
$M_{pcx} = 1.18 M_{px} (1 - P/P_y)$			9,316	12,347
$M_{ucx} = M_{px} (1 - P/P_{cr}) (1 - P/P_{ex})$			7,728	10,210
M_x / M_{pcx}			0.43	0.48
$C_{mx} M_x/M_{ucx}$			0.44	0.49

C.3.3.4 Load-Deflection Inelastic Analysis, Post-Yielding Capacity

Since a response spectra analysis is used for EQ-II earthquake, a load deflection inelastic analysis is required. A check for the ultimate capacity of the Frame 4 on base isolators is performed. The force-displacement curve is obtained by the following procedure:

1. Define the force that causes the first major yield in the frame. Then calculate the associated roof displacement, these force-displacement value plots the point A on the force-displacement curve.

The forces to produce yielding of a component are obtained by the combinations:

$$1.0 E + 0.8 DL$$

$$1.0 E - 1.2 DL - 1.0. LL$$

2. Determine the first post-yield segment of the curve by freezing the frame at the point of initial major yield, and calculate the incremental force that causes the second yield of the frame. Then calculate the incremental roof displacement. The total force-displacement value plots the point B on the curve.
3. Determine sequential post-yield segments on the curve by repeating the same procedure above.
4. The procedure is repeated until a failure mechanism, or instability occurs.
5. In this analysis, the increase of the total base shear force is distributed to the stories by a dynamic analysis, i.e. response spectra analyses. After each major yield, and in order to determine the next post yielding segment, a dynamic analysis is performed on the frame to calculate the new periods and mode shapes. Then these new periods and mode shapes are used to calculate the percentage of base shear force that is taken by each story.

Limit State Analysis

Figure C.2

Figure C.2 shows the sequence of plastic hinges formations in the frame, and Tables C.6 show a comparison between the force-displacement results for the frame designed with fixed base and isolated base. Figure C.3 shows plots of the base shear-drift capacity curves and the base shear coefficient-drift capacity curves. And finally Table C.7 shows comparison between the weights of the frame members designed for the navy criteria for fixed base and isolated base buildings.

Fixed Base Design

Point	Base Shear kips	V/W	Roof Displ. in.	Roof Drift Ratio
Int.	1392	0.778	2.48	0.0063
A	1512	0.845	2.78	0.0070
B	1581	0.884	3.32	0.0084
C	1617	0.903	4.12	0.0104
D	1635	0.914	5.73	0.0145

Base Isolated Design

Point	Base Shear kips	V/W	Roof Displ. in.	Isolator Displ.	Roof Drift Ratio
Int.	678	0.38	30.95	28.33	0.0067
A	846	0.47	39.25	35.41	0.0098
B	909	0.51	42.5	38.05	0.0114
C	999	0.56	48.05	41.81	0.0169
D	1065	0.60	53.04	44.50	0.0218
E	1098	0.61	57.99	45.89	0.0309

Tables C.6

Figure C.3
Force-Deflection Curves for
Fixed-Base vs Base-Isolated Designs

Table C.7
Comparison Between Frame Total Weight

Base Isolated Design	Beams	:	11712	lb.	(GR36)
	Columns	:	10780	lb.	(GR50)
	Total	:	22492	lb.	
Fixed Base Design	Beams	:	14592	lb.	(GR36)
	Columns	:	14926	lb.	(GR50)
	Total	:	29520	lb.	
Percentage saving in steel weight is 25%.					

C.4 Analysis 2

Analysis 2 is a response spectra analysis that provides a lower bound safety net design. The structure designed in Analysis 1 is assumed to be fixed base and then subjected to EQ-I design earthquake. The Inelastic Demand Ratios of the beams and the columns must be less than 3. Table C.8 shows EQ-I response spectra analysis: the fixed base periods, mode shapes, and modal participation factors. Table C.9 shows maximum modal story forces and maximum modal cumulative story shears and the SRSS modal combination. Figure C.4 shows element forces for load combination $1.2D + 1.0L \pm E$.

Table C.8
Analysis 2, Periods and Mode Shapes

Level	Mass $\frac{k \cdot sec^2}{ft.}$	Mode 1		Mode 2		Mode 3	
		ϕ_{x1}	T_1	ϕ_{x2}	T_2	ϕ_{x3}	T_3
R	3.88	1.0454	0.753	1.2661	0.234	0.6300	0.123
2	7.24	0.8581	sec	-0.2470	sec	-0.9275	sec
1	7.24	0.5792		-0.8587		0.7646	
Modal Participation Factors		1.2052		-0.2577		0.1054	

Analysis 2, Response Spectra Analysis
Table C.9

Maximum Modal Story Forces (kips)				
LEVEL	MODE 1	MODE 2	MODE 3	SRSS
R	71.93	-23.85	4.86	75.93
2	110.17	8.68	-13.34	111.31
1	74.37	30.18	10.99	81.01

Maximum Modal Cumulative Story Shears				
LEVEL	MODE 1	MODE 2	MODE 3	SRSS
R	71.93	-23.85	4.86	75.93
2	182.09	-15.17	-8.48	182.92
1	256.46	15.01	2.51	256.91

EQ-I = 1.2D + L ± E
Analysis 2

Figure C.4

C.4.1 Inelastic Demand Ratios Check

The Inelastic Demand Ratios for beams or columns can not exceed 3.0. Tables C.10 and C.11 show the IDR's checks for the beams and column using the response spectra computer output and the Navy's Technical Manual design criteria.

Table C.10
Beams IDR Check for Analysis 2

Level	Beam	M_p kips.in	M_d kip.in	M_d/M_p	IDR
R	W18x40	2822	2290	0.81	3.0
2	W21x93	7956	6840	0.86	3.0
1	W21x111	10044	11480	1.15	3.0

Beam-Column IDR Check for Analysis - 2
Table C.11

Description			Exterior Column Level 1	Interior Column Level 1
Column section		GR 50	W14 × 99	W14 × 132
P	for analysis-2	kips	116	236
M	for analysis-2	kips.in.	8,089	12,420
A	cross sectional area	in.	29.10	38.80
S _x	section modulus	in ³	157.00	209.00
Z _x	plastic modulus	in ³	173.00	234.00
r _x	radius of gyration	in.	6.17	6.28
r _y	radius of gyration	in.	3.71	3.76
I _x	moment of inertia	in ⁴	1110	1530
K _y			1.00	1.00
G _x bottom			1.00	1.00
G _x top			2.21	1.87
K _x			1.47	1.43
L unbraced length		in.	119	119
(KL/r) _x			28.35	27.10
(KL/r) _y			32.08	31.65
F _a		ksi	26.89	26.95
F' _{ex}		ksi	185.78	203.38
C _{mx}		ksi	0.85	0.85
Navy Criteria:				
$M_{pcx} = 1.18 M_{px} (1 - P/P_y)$			9,393	12,127
$M_{ucx} = M_{px} (1 - P/P_{cr}) (1 - P/P_{ex})$			7,807	9,988
M_x / M_{pcx}			0.86	1.02
$C_{mx} M_x / M_{ucx}$			0.88	1.06

SEISMIC STRENGTHENING OF A TALL CONCRETE BUILDING INCORPORATING BASE ISOLATION

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ABSTRACT

This paper presents a summary of the work involved in developing a seismic strengthening system for an eight-story reinforced concrete office structure located in Southern California. The seismic strengthening scheme includes the installation of base isolators at the ground and the addition of a new exterior frame and supporting foundation. The paper presents a description of the design criteria, structural seismic strengthening, and equipment tie down.

INTRODUCTION

This paper describes the seismic strengthening of an existing concrete frame building. This type of building was particularly common in concrete construction prior to the introduction of special detailing requirements for ductile concrete frames.

The seismic strengthening was base isolation and this method of seismic strengthening accomplishes two major objectives. First, base isolation reduces the seismic forces transmitted to the superstructure enabling the structural system to be seismically strengthened to a performance criteria that included two design limit states. Limit State 1: no yielding of the steel in the members and Limit State 2: no spalling of the concrete. The design criteria was that these limit states not be violated for a design earthquake with a 20% probability of not being exceeded in 50 years. Second,

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base isolation reduces the floor accelerations induced by the earthquake and thus also enabled us to address the Serviceability Limit State of no disruption to human beings or contents of the building during a design level earthquake.

Previous papers presented at the Fourth US Conference on Earthquake Engineering described the building, the design criteria, the method of seismic strengthening and the analytical methods used in the project. The members of the project design team that performed these tasks are identified in Table 1.

DESCRIPTION OF EXISTING BUILDING

The building under consideration is an eight-story rectangular office structure with a partial basement. The building was designed in conformance with the 1964 Uniform Building Code. It was constructed in 1967 and consists of reinforced concrete frames supporting concrete waffle slab floors and a roof. The foundations for the building are case-in-place concrete piles. A plan and elevation of the building are shown in Figures 1 through 3. The base isolators are denoted in these figures as dark lines located between first and second floors.

The concrete frame is representative of many similar structures located throughout the United States which were designed and constructed prior to the current requirements for ductile concrete framing. Material properties obtained from tests on the building as part of the retrofit design process showed that the condition of the existing material is excellent.

The site is located in Los Angeles county in the city of Seal Beach. Geotechnical exploration has shown that the underlying soil materials in the upper layers are predominantly clay ranging in stiffness from medium dense to very stiff. About 25 feet below the ground surface, the soils become predominantly very dense silts to gravelly sands with some very hard clay. The water table is located approximately 20 feet below the ground surface.

The building is supported on drilled and cast-in-place piers typically 3 feet in diameter and 50 feet long. There are some 4-foot diameter piers. The design load capacity of the 3-foot diameter piers, used in the original construction design, was 600 kips.

SEISMIC RETROFIT DESIGN CRITERIA

The building site is located within 0.7 mile of the Newport-Inglewood fault which is as classified as an active fault and it was the source of the 1933 Long Beach earthquake. The design criteria adopted for this retrofit considered near field ground motions and four design earthquakes were used and they are as noted in Table 2.

The limit states and the acceptable levels of risk associated with each of the design earthquakes is summarized in Table 3. The life safety requirements as well as property and business interruption protection beyond conventional building design requirements as specified in the 1991 Uniform Building Code. These requirements reflect the major emphasis on protection of property as well as continuation of operation in the building after a major earthquake.

The existing structure is a concrete frame with limited ductility and thus a critical task was to establish drift limits based upon the strength and ductility characteristics of the existing structural system. These lateral floor motion drift limits incorporated the properties of the existing concrete, the lap splicing of the reinforcing bars, the embedment of the reinforcing bars, and the expected performance of the structures at different levels of interstory displacement. The drift ratio limit selected for design of the building was less than one-half percent for all design earthquakes.

VIBRATION PROGRAM

A series of vibration tests were performed on the building. The objective of the vibration test program was to obtain dynamic characteristics of the building structure, foundation system, and selected floor elements. The results from the test program were used to help and calibrate analytical models for the seismic response and retrofit design.

Two types of vibration tests were conducted on the building:

1. Ambient vibrations;
2. Forced vibrations from man-induced excitation as well as impact from drop weights.

The ambient and forced vibration test programs were conducted over a 3-day period of time and were used to estimate the natural frequencies, model damping ratios, mode shapes and foundation stiffness properties of the building. These test results represented low-strain conditions in the structure and its foundations system.

Motions from the ambient and forced vibration test programs were recorded (1) on the roof, (2) on the central and east stairwells at various floors, (3) on a floor slab between columns, (4) at the top of the raised access floor supporting the computers, (5) at various locations in the basement, and (6) in the parking lot east of the building (free-field location).

In the ambient vibration test program, two transducers were used simultaneously to record building motions. One transducer was located at the roof in the southeast corner as a fixed reference point. The other transducers were moved to different locations within the building and to the one location in the free field. Motions measured by the portable transducers within the building were normalized in the frequency domain by the motions recorded at the fixed roof transducer.

The natural periods of building vibration were computed prior to the vibration test program using two- and three-dimensional finite element computer models. The models were prepared from physical properties of the building defined on the original design drawings. These properties were based on gross concrete sections (not cracked) and did not include foundation flexibility or stiffness. Table 4 summarizes the analytical values predicted from the compute analysis of the building as well as the experimentally determined natural periods picked from the fourier amplitude spectra.

Under the ambient test program, mode shapes associated with each of the fundamental modes of building vibration were computed from ambient building responses recorded at the roof, 7th, 4th, 2nd, 1st floors and basement of the building. In each ambient test, simultaneous records were

made on the portable Ranger seismometer as well as the one fixed seismometer on the roof. After each test, the fourier transform of the record at the portable station was divided by the corresponding transform of the roof record to normalize the results. The amplitude and phase of this transfer function was used to construct the mode shapes.

The amplitude of vertical motion at the basement level in opposite corners provided a measure of building rocking in the fundamental mode. This rocking effect is readily apparent in the fundamental north-south and east-west modes. No rocking was evident in the second mode. This rocking motion was even more apparent when the building was excited using man-induced excitation tests. This result was expected since there is less resistance to rotation in the north-south direction, the direction parallel to the short side of the building. In both directions, the rotations at the foundation level are relatively small because of the presence of pile foundation and firm soil conditions. Consequently, the contribution of foundation rotation to the total translation motion at the roof level is small and on the order of 3 to 5 percent thus soil-structure interaction was not a significant consideration for the building.

Man-induced excitation test of the building were performed by two persons who pushed and pulled on the central stairwell wall at the roof level, exciting the building in its fundamental mode. These tests were performed in the north-south and east-west direction with the push-pull cycle of approximately 1 second, the fundamental north-south period of building vibration. This period forcing produced resonant response amplitudes of building motion that were an order of magnitude larger than those generated during the ambient test. In one of the tests, after steady-state conditions were achieved, the man induced motion was terminated abruptly and free-vibration decay of the building motions was recorded. Damping in the fundamental mode of building vibration was computed using the logarithmic-decrement method. The estimated damping ratios varied between 1.5 and 2 percent of critical damping for both directions of building response, depending on the number of cycles included in the analysis. Damping estimates are given in Table 5.

The series of man-induced impact tests were performed to determine fundamental vertical frequency of two typical interior elevated floor slabs as well as the horizontal natural frequency of a raised access floor system used for supporting equipment. The vertical impact tests were performed on a typical interior waffle slab with the loading applied at the midspan of the slab by a man jumping off a chair onto the floor. The recorded vertical motions of the slab under free vibration decay provided data for computing the natural frequency of 7.5 cycles per second and damping of 14 percent of critical. A similar impact test was conducted on a floor slab adjacent to an exterior column line where the stiffening effects of deep spandrel beams increased the natural frequency to 11 cycles per second and the resulting damping was 11 percent of critical.

The horizontal impact test on a raised access floor system provided a range of natural frequencies with the dominant natural frequency being at 22 Hertz and damping at 10 percent of critical. Results of the vertical and horizontal floor slab vibration tests revealed significantly higher damping than might have been anticipated.

The translational, rotational and torsional foundation stiffness were estimated from the approximate fundamental mode shapes and the basement and roof. Equations of motion for fundamental response modal including the north-south, east-west and torsional directions were solved for the unknown foundation stiffness. Table 6 summarizes the computed foundation theoretical values for foundation stiffness based on elastic half-space computed using the Wong and Luco theory assuming a foundation length-to-width ratio of 2. For the half-space, values assumed for shear wave velocity, density, Poisson's ratio, and material damping ratio were 1,000 feet per second, 110 pounds per cubic foot, 0.33 and 2 percent, respectively. These properties were derived from building dimensions and geotechnical data gathered from the site.

The experimental and theoretical values for translational stiffness, K_x and K_y , in Table 6 are similar, whereas the experimental values for rocking stiffness, $K_{\theta y}$ and $K_{\theta x}$ are approximately 30 to 35 percent greater than the corresponding theoretical value. These results appear to be reasonable when considering the expected stiffness effect of the pile foundations. The lateral contribution of

pile stiffness to the overall translational stiffness of the foundation is not expected to be large because of the relatively large spacing and low flexural stiffness of the piles. On the other hand the axial stiffness of the piles is expected to contribute significantly to the overall rocking stiffness of the foundation, which is consistent with the comparison between experimental and theoretical estimates for rocking stiffness in Table 6.

SEISMIC EVALUATION AND DESIGN

To assess the extent of structural strengthening and foundation upgrade required for the building, a series of three-dimensional linear elastic and two and three-dimensional nonlinear finite element analyses were performed on computer programs known as COMBAT, DRAIN 2D, LPM and ANSR II. These analyses were performed using the design earthquake response spectra as well as synthetic time history motions taken from actual earthquakes recorded from near and distant events of comparable size.

The linear elastic and nonlinear analyses provided a basis for assessing the extent of inelastic deformation imposed upon the nonductile concrete frame structure and its foundation system. The strengthening in conjunction with base isolation was an optimum solution between large interstory drifts across the isolators and inelastic response in the superstructure frame above the base isolators. The objective was to limit isolator deflections to 12 inches under the large near-field Newport-Inglewood event (EQ-4) to accommodate existing utilities that cross the plane of base isolation within the building. To satisfy this drift limit, and still prevent the frame above the isolators from experiencing excessive inelastic deformation and permanent cracking and spalling strengthening had to be provided to the superstructure.

By placing the structure on base isolation, forces generated in the superstructure were significantly lower than those under conventional retrofit strengthening programs. This minimized the need for seismic strengthening of the pile foundations for lateral earthquake loading. However, the retrofit scheme imposed additional vertical loads on the foundation systems that exceeded the

existing pile's axial load capacity. Consequently, additional drilled and cast-in-place concrete piers were added to the foundation as part of the overall retrofit program. The reinforcing steel for these new piles was designed to resist lateral loads under the largest earthquake without yielding. However, the soils surrounding the piles were permitted to respond in the inelastic domain.

SEISMIC STRENGTHENING

Figures 1 and 2 show elevation views of the structure and identify the location selected for the placement of the isolators. This location was chosen because of the ease of construction and also because of the excellent spiral confinement of the interior columns. The isolators selected for the project were designed and supplied by Dynamic Isolation Systems (DIS) of Berkeley California.

The isolation system, located between the first and second floors as shown in Figures 1 and 2, consists of lead rubber bearings having the stiffness properties given in Table 7. K_1 represents the stiffness prior to the lead core yielding and K_2 represents the post yield stiffness. The isolator system is installed in a specially prepared connection in the first floor columns. The isolators are bonded to the end bearing plates and bolted into the connecting anchorage to the columns. This connection procedure provides a safety restraint in the unlikely event the isolators are accidentally forced beyond their design deflection.

To strengthen the existing reinforced concrete exterior perimeter nonductile concrete frame, an additional frame of ductile concrete was applied over a portion of the building exterior to control story drifts in the lower levels immediately above the isolator where computed ductility demands exceeded the acceptable performance requirements. The added exterior frame, shown in Figures 1 through 3, was cast directly against the C-shaped exterior columns and a space was left between the spandrel beam elements of the existing frame and the new frame to accommodate existing exterior treatment. The two frames performed as an integral structure through the common column system.

This retrofit scheme minimized disruption of building functions. Thus, on all but the first floor, normal building operations could be maintained during the 1-1/2-year retrofit construction phase.

An additional 28 drilled and cast-in-place concrete piers 3-foot in diameter and 50-foot in length are installed to support the added weight of the new concrete frame on the exterior of the building as well as seismic overturning loads at the corner columns. These new piers are ductility reinforced to resist bending and axial forces.

ANALYTICAL TASKS

The three primary analytical tasks that were performed each had a different set of objectives as follows:

1. **EQUIVALENT 3-D LINEAR MODEL** - This analytical model was developed to be consistent with the new SEAOC Base Isolation criteria for an elastic 3-D analytical model for building design. This analytical model was a COMBAT 3-D linear model that incorporated the equivalent linear properties of the isolation system. From design perspective this was the primary "workhorse" analytical model since it provided all primary member forces and interstory drifts for design and checking purposes. The model included an appropriate model of the foundation system as well as all structural members. The seismic input used in the model was a response spectra that was obtained by modifying the original design response spectra in order to incorporate the energy dissipation characteristics of the isolation system.
2. **2-D NONLINEAR FRAME MODEL** - This analytical model was developed to be consistent with the new SEAOC Base Isolation criteria for a nonlinear analytical model for verification of interstory drift calculations. Each of the external transverse and longitudinal primary structural frames were modelled separately as a 2-D member by member nonlinear frame, incorporating the nonlinearities of the isolation system and

all the structural members. The Drain 2-D program was used for the analysis. The primary objective of this model was to determine the ductility demand on each column and beam member and to check the interstory drifts. Also as part of this research, it was used to check interstory shears and floor response spectra with the other models. From the individual member moment-curvature relationships maximum values of concrete strain, steel strain and curvature ductility were obtained. Since these were 2-D models and the external frames were the primary seismic resisting elements one-half of the building floor level mass was assigned to each level. It could be argued that this was too severe and ignored the interior framing action through the floor slabs on other column lines however there were two other limitations with this model. The DRAIN 2-D analysis could only include one component of motion. The orthogonal component was ignored. Secondly, the DRAIN 2-D model could not account for the torsion resulting from mass eccentricity. Each of the column lines in the frame were modelled with an isolator between the 1st and 2nd floor levels. The remaining isolators for the one-half building were modelled on an exterior column line that was slaved into the affected floor levels.

3. **3-D NONLINEAR MODELLING** - There were two different 3-D nonlinear analytical models used in this phase of the work. The first used the ANSR II computer program and the second used the LMP computer program. Both models assumed that the superstructure was essentially elastic. In both 3-D models the condensed stiffness matrix of the structure above the isolators was obtained from the linear COMBAT model. This "super-element" was a 24 degree of freedom elastic element that included the stiffness contribution of every member of the lateral force resisting system and the axial properties of each isolator. In both models the isolator is modelled explicitly with a 3-D yielding beam element. During the nonlinear analysis the super-element remains elastic and the isolators exhibit their nonlinear properties. In addition to the difference

in the two computer programs solution techniques, the structural difference in the two 3-D nonlinear models was that the ANSR model assumed the structure below the isolators was rigid whereas the LPM model explicitly modelled the structure below the isolators.

ANALYSES PERFORMED

For the purposes of reporting the results the components of the spectra compatible time histories for EQ4 and the various combinations used are delineated as follows (L and T refers to the longitudinal and transverse axes of the building respectively):

DRAIN 2-D

COMP 1 - Array 7 - 230°

COMP 2 - Array 7 - 140°

COMP 3 - Array 6 - 230°

COMP 4 - Array 6 - 140°

3-D ANALYSES

Set 1 L - COMP 1, T - COMP 2

Set 2 L - COMP 2, T - COMP 1

Set 3 L - COMP 3, T - COMP 4

Set 4 L - COMP 4, T - COMP 3

For the 2-D analyses only one component of the frequency compatible time histories was used, resulting in a total of 4 analysis for each of the longitudinal and transverse frames. For the 3-D analysis 100% of each component was used in the longitudinal and transverse directions, as designated above, resulting in a total of four sets of time history analyses. These 3-D analyses were performed with and without a mass eccentricity. The mass eccentricity was calculated to be 3.5% in each of the longitudinal and transverse directions and was applied in both directions simultaneously.

The results presented in Table 8 indicate that the maximum displacements in each of the two primary directions predicted by the COMBAT analysis are conservative when compared to the maximum unidirectional results of the transverse and longitudinal ANSR nonlinear analyses.

The results given in Table 9 are for the corner displacements at nodes 1 and 56 for the two different 3-D nonlinear analytical models. The ANSR model did not include the sub-basement structure whereas the LPM model did. The results presented in Table 10 and 11 are for the Drain 2-D models of the external longitudinal and transverse frames, respectively.

The results of the 2-D nonlinear frame analyses presented in Tables 10 and 11 are compared to the center of mass results presented in Table 8, since the 2-D analyses could not incorporate any torsional response. The results of Table 10 for the longitudinal frame for earthquake components 1 through 4 are compared directly with sets 1 through 4 of Table 8. The isolator displacements and maximum interstory drifts are in excellent agreement for this direction of loading.

CONCLUSIONS

The seismic strengthening of an existing building using base isolators is a viable and desirable method. The benefits are the reduction of earthquake forces to the superstructure without including the large accelerations other strengthening methods give and also the minimum impact that this strengthening method has on the occupants in the building during the rehabilitation construction phase.

Table 1. Seismic Strengthening Design Team

FUNCTION	TEAM MEMBER
Prime Consultant, Structural Engineering; and Architecture	Englekirk & Hart, Inc. Gary C. Hart (engineer of record), Dennis Drag, Bill Wallace, Jim Elmlinger, Jack Martin
Geotechnical; Ground Motion Estimation; Ambient Vibration Test; and Structural Design Review	Dames & Moore Bill Gates, Neville Donivan, Uday Patil, C.B. Crouse, Mike Mehrain
Structural Dynamic Computer Consultant Analysis	Ewing & Associates Bob Ewing
Mechanical	Meckler Group M.H. Meckler
Electrical	P. Bryne & Associates Pat Bryne
Material Testing	Accu-Test Engineering Laboratories, Inc.
Non Structural and Building Contents	Dames & Moore Bill Gates, Bill Graf

Table 2. Design Earthquakes

Earthquake	Description
EQ-1	Distant earthquake (48 miles/79 km) of approximately Magnitude 8 or greater on the San Andreas. A highly probable event in the life span of Building 80.
EQ-2	Earthquake with approximately a 50% probability of being exceeded in 50 years, with an estimated recurrence interval of 72 years.
EQ-3	Earthquake with approximately a 10% probability of being exceeded in 50 years, with an estimated recurrence of 475 years.
EQ-4	Nearby earthquake (approximately 0.7 miles/ 1 km) of Magnitude 7.0 on the Newport-Inglewood fault. A Maximum Credible Event having an estimated probability of occurrence that is less than 2% in 50 years, or a recurrence interval of more than 2475 years.

Table 3. Expected Building Performance After Base Isolation

	EQ-1	EQ-2	EQ-3	EQ-4
At Risk	Magnitude 8.0 San Andreas	Lower Level 50% in 50 yr	Upper Level 10% in 50 yr	Maximum credible event on Newport-Inglewood fault 1 km from building
Life Safety	Minimal injury	Minimal injury	Minor injury	Minor injury loss of life not expected
Property - Building - Equipment	1/2% Minimal	1% Minimal	2% Minor	5% Minor
Operations	Minutes- hours	Minutes- hours	hours	Hours to days

Table 4. Natural Periods of Building Vibration, Experimental and Predicted

Mode	N-S Direction	E-W Direction	Torsion
1	1.04* (1.09)**	0.83 (0.82)	0.69 (0.61)
2	0.33 (0.35)	0.27 (0.27)	-- (0.20)

* Units are in seconds.

** Numbers in () are analytical predictions

Table 5. Model Damping Values (Percent of Critical)

From Man-Induced Excitation Test				
Mode	Cycles	N-S Direction	Cycles	E-W Direction
1	4	1.9	4	2.2
	15	1.4	10	1.3

Table 6. Foundation Stiffness

Direction	Stiffness	Experimental	Theoretical
N-S	K_x	2.18×10^9 lb/ft	1.81×10^9 lb/ft
	K_{oy}	1.15×10^{13} lb-ft	0.876×10^{13} lb-ft
E-W	K_y	1.62×10^9 lb/ft	1.66×10^9 lb/ft
	K_{ox}	3.14×10^{13} lb-ft	2.33×10^{13} lb-ft
Torsional	K_{oz}	1.78×10^{13} lb-ft	2.22×10^{13} lb-ft

Table 7. Isolator Properties

Isolator No.	K_1 Kips/in.	K_2 Kips/in.	F_y Kips	Isolator Location by Column No.
1	140.5	10.8	117.8	1, 6, 51, 56
2	164.2	15.2	111.4	2, 3, 4, 5, 7, 12, 13, 18, 19, 24, 25, 28
3	181.4	14.9	141.6	29, 32, 33, 38, 39, 44, 45, 50, 52, 53, 54, 55
4	6.0	6.0	Elastic	8, 9, 10, 11, 14, 15, 16, 17, 20, 21, 22, 23, 26, 27, 30, 31, 34, 35, 36, 37, 40, 46
5	9.2	9.2	Elastic	41, 43, 47, 49

Table 8. Comparison of Center of Mass Results

Analysis Method	LONGITUDINAL			TRANSVERSE		
	Isolator Displ.	Roof Displ.	Max. Drift	Isolator Displ.	Roof Displ.	Max Drift
COMBAT	12.2	13.0	.0011	12.3	13.7	.0016
ANSR						
Set 1	10.5	11.3	.0008	9.40	10.4	.0014
Set 2	9.4	10.2	.0007	10.4	11.5	.0011
Set 3	10.2	11.1	.0009	11.8	12.9	.0012
Set 4	11.9	12.7	.0009	10.2	11.3	.0011

Table 9. Comparison of 3-D Non-Linear Results

Seismic Input	Analysis Model	Mass Eccentricity	Isolator Displacement - Node 1			Isolator Displacement - Node 56		
			Long	Tans	SRSS	Long	Trans	SRSS
Set 3	LPM	None	10.2	12.7	14.2	10.0	11.9	13.2
Set 3	ANSR	None	10.2	12.7	14.6	10.2	11.5	13.8
Set 4	LPM	None	13.1	10.6	14.8	12.8	9.9	14.2
Set 4	ANSR	None	11.9	10.3	14.2	11.9	10.0	14.3
Set 3	LPM	Yes	10.7	14.6	16.2	9.3	10.1	11.2
Set 3	ANSR	Yes	10.7	14.2	16.7	9.7	9.5	11.7
Set 4	LPM	Yes	14.0	12.0	16.5	11.9	8.5	12.8
Set 4	ANSR	Yes	12.9	11.3	16.2	10.9	8.9	12.5

Table 10. DRAIN 2-D Longitudinal Frame

Parameter	Comp. 1	Comp. 2	Comp. 3	Comp. 4	Average
Max. Roof Displacement	11.1	10.4	10.8	13.0	11.3
Max. Isolator Deflection	10.2	9.6	9.9	12.0	10.4
Max. Interstory Drift	0.0006	0.0006	0.0007	0.0007	0.0006
Max. Base Shear	14.3%	13.7%	14.0%	16.0%	14.5%

Table 11. DRAIN 2-D Transverse Frame

Parameter	EQ Comp. 1	EQ Comp. 2	EQ Comp. 3	EQ Comp. 4	Average
Max. Roof Displacement	11.3	10.0	11.0	12.7	11.2
Max. Isolator Deflection	9.4	8.4	9.1	10.7	9.4
Max. Interstory Drift	0.0015	0.0013	0.0015	0.0016	0.0015
Max. Base Shear	14.7%	13.7%	14.4%	15.9%	14.7%

FIGURE 1. NORTH WALL ELEVATION

FIGURE 2. EAST WALL ELEVATION

FIGURE 3. 2ND & 3RD FLOOR PLAN

WORKSHOP ON SEISMIC BASE ISOLATION FOR DEPARTMENT OF ENERGY FACILITIES

DR. JAMES M. KELLY

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LECTURE 2

Base Isolation in Japan

Introduction

After a slow start, base isolation research and development in Japan has increased rapidly. The first large base-isolated building was completed in 1986. Base-isolated buildings in Japan require special approval from the Ministry of Construction. Even so, by mid-1990, 25 buildings were completed or under construction and a total of 50 had been approved.

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

IMPLEMENTATION OF BASE ISOLATION IN JAPAN

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

In contrast, base isolation research and development was slow to start in Japan, but since the first large base-isolated building was completed in 1986 the use of base isolation has increased very rapidly. There are now (late 1988) seventeen base-isolated buildings either completed or under construction and many others in the design phase or undergoing analysis.

In Japan the large construction companies have taken the leading role in developing base isolation technology. These companies have decided that base isolation is superior to conventional seismic design and can provide a competitive edge in the construction industry. They are therefore aggressively marketing the technology to potential clients. The first buildings on base isolation were, accordingly, demonstration projects with the construction companies building them for their own use. More recent projects have been for a variety of clients and have included office buildings, manufacturing facilities, and apartment blocks.

The most common form of base isolation used in Japan is steel-laminated natural rubber bearings with additional devices to enhance the damping in the system. The rubber compounds used have very low intrinsic damping and the additional elements, such as steel rods, viscous dampers, hydraulic shock absorbers, and lead plugs, among others, have been used to produce the necessary damping in the system. In a way these designs mirror the research history in the United States where in extensive shaking table

testing many energy-absorbing mechanisms were tried in parallel with the rubber bearings. As the experimental research progressed over a period of ten years, the various additional dampers were shown to be unnecessary, inconvenient and sometimes deleterious and the damping was incorporated in the elastomer itself through appropriate compounding. The high-damping natural rubber isolation system used in the San Bernardino building evolved from this research program. Recently Bridgestone Corporation developed a natural rubber with high damping and several of the construction companies in Japan are evaluating this approach. These isolators have been installed in one demonstration building as described below.

DEMONSTRATION BUILDINGS ON BASE ISOLATION SYSTEMS

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

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

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

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

Observations of the earthquake response of the building over a three-year period after completion and installation of the seismometers reported by Tada *et al.* [2] have

Figure 1 Base-Isolated Dwelling in Yachiyo City

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

Figure 4 Building Plan and Bearing Layout

Figure 3 Cross Sections of Building

been highly favorable although the ground motions experienced have in most cases been quite small. The roof accelerations have always been less than the ground accelerations, sometimes dramatically so, and as expected the acceleration reduction ratio increases with increasing ground motion. The highest recorded input during the initial monitoring period occurred in late 1985 when a peak ground motion of 10.2%g produced a peak roof acceleration of 2.4%g. A summary of the recorded data is shown in Figure 5. The observation program is continuing and in December 1987 an earthquake with a ground motion of 13.1%g was recorded. The maximum roof acceleration in this case was only 3.5%g, demonstrating the remarkable attenuation of the isolation system.

Almost certainly as a result of the entirely favorable response of the building in Yachiyo City a number of much larger buildings using base isolation were constructed in Japan with many being completed in late 1986 and 1987. In almost every case these buildings were designed and built by construction companies as demonstration buildings and are used by the companies as offices and laboratories. These buildings were built to test the feasibility of the isolation systems, to observe their responses to earthquakes and to demonstrate the technology to potential clients.

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

The building is used as an office building for the Okumura Corporation Research Institute. It has a total floor area of 1330 m² and a total weight of 2250 tonnes. A test program on the building has been carried out jointly by the Okumura Corporation and the Central Research Institute of the Electric Power Industry (CRIEPI). Static tests on the elements and the building and forced- and free-vibration tests on the building were carried out. The building was extensively instrumented with seismometers and its response to earthquakes has been observed as part of this experimental program. The results of the various studies have been reported by Abe et al. [3] and Aoyagi et al. [4].

Figure 5 Peak Acceleration Distribution at the Yachiyo City Building

Figure 6 Okumura Company Demonstration Building
in Tsukuba Science City

Figure 7 Isolators in Okumura Building

Figure 8 Coil Type Elasto-Plastic Steel Dampers

Figure 9 Section through Building

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

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

The fact that the dampers do not yield until 3 cm displacement results in the building being very responsive to small low-frequency input producing an amplification of the input at these frequencies. This is to be expected since for small input the system is almost undamped. To compensate for this effect an additional damper in the form of a shear lead plug has been added to increase the damping for low-level inputs. As of this writing no information on the response with these additional elements in place has been reported.

The next major base-isolated structure to be completed was constructed by the Ohbayashi Corporation as a research laboratory at the company's research institute. It is again both a demonstration building and a test structure for the Ohbayashi Corporation's base isolation system.

The building is a five-story reinforced concrete structure 1624 m² in total area. It is 22 m long, 15 m wide and 22 m high. The isolation system uses 14 natural rubber isolators 756 mm diameter and 406 mm thick. The isolators are in two rows along the long sides of the building and are designed to provide a horizontal period of about 3 seconds and a vertical frequency of 15 Hz. Damping is provided by 96 vertical steel rods between the foundation and the superstructure, each rod being 32 mm in diameter. They are arranged in groups of eight between the isolators. The building and the isolation elements are shown in Figures 11 through 14.

The building is extensively instrumented with strong motion accelerographs and has experienced some earthquakes (Takeda *et al.* [5]). In one event a peak ground acceleration of 4.4%g was recorded with 0.71%g being recorded at the roof of the building. This is to be compared with the accelerations recorded in an adjacent administration building, an entirely conventional four-story reinforced concrete structure. In this building, the

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

Figure 12 Isolators and Cantilever Type Elasto-Plastic Steel Dampers

Figure 14 Section through Building

Figure 13 Plan of Building—Layout of Isolators and Dampers

roof acceleration was recorded at 5.9%g.

The next building to be completed was the Oiles Company Technical Center in Fujisawa City, Kanagawa Prefecture. This is currently the largest base-isolated building in Japan and is used as a laboratory facility at the research institute of the Oiles Industry Company. Oiles Industry is a manufacturer of self-lubricating bearings, roller and sliding bearings for bridges. Oiles has an agreement with the Development Finance Corporation of New Zealand to manufacture lead-filled base isolation bearings in Japan. The company commissioned the design and construction of the demonstration building at their plant in Fujisawa City. The building was designed and built by the Sumitomo Construction Company.

The Oiles Technical Center (Figure 15) is a five-story reinforced concrete frame building 36 m by 30 m in plan (Figure 16), with column grids at 6.0 m and 9.0 m centers. A vertical section of the building is shown in Figure 17. As a technical center, the building houses the Departments of Research, Technical Development, Quality Assurance, Technical Information, and Mechanical Development. The Oiles Patent Office and Computer Center are also located in the building.

Construction began in April 1986 and was completed in February 1987. It has a floor area of 4800 m² and a total weight of 7500 tonnes above the isolators. Thirty-five lead-filled elastomeric isolators support the structure with diameters ranging from 650 mm to 800 mm. The lead plugs vary from 130 mm to 160 mm in diameter. All bearings are 363 mm high.

The building was designed with a base shear coefficient of 0.2, the same as that required for a conventional building. No economy was therefore possible in the design and the rationale for construction was improved seismic safety, damage-free performance in major earthquakes and its use as a demonstration building. For this last purpose the building serves three useful functions. First, it is used to show building owners, architects, and structural and building service engineers, typical details and configurations necessary to construct and maintain a base-isolated building. Second, it is used as a full-scale isolation test laboratory. Both forced- and free-vibration tests have been conducted on the building and these are repeated from time to time to check the durability and reliability of the isolation system. Useful data on the recovery of hysteretic damping after free-vibration testing have also been obtained. Third, the building is instrumented

Figure 15 Oiles Industries Ltd. Demonstration Building in Fujisawa City

Figure 16 Plan of Building Showing Location of Isolators

Figure 17 Section through Building

with twenty-seven strong motion accelerographs, eight displacement transducers, and nine stress sensors in the piles. Records of structural response have been obtained for several earthquakes [6]. While most of these have been low-level events for which the hysteretic dampers have barely yielded, measurable attenuation in floor accelerations has been demonstrated and the design assumptions have been confirmed. Figure 18 illustrates acceleration records from the building during the east Tokyo earthquake of March 18, 1988. The measured peak ground acceleration was 6.2%g and at the roof 4.3%g. The relative displacement at the isolation system for this input was just under 1 cm.

A laboratory building at the research institute of the Kajima Corporation was completed at about the same time as the Oiles Technical Center. This structure, the Environmental Engineering Laboratory located near Tokyo, is a functional building that doubles as a full-scale experimental model for advanced construction technology. The facility consists of two adjacent buildings, an acoustic/environmental vibration laboratory and a thermal control/air flow/equipment laboratory. The two buildings are connected by a corridor and an atrium roof (Figure 19). The thermal control laboratory was constructed using conventional precast concrete technology and is supported on a standard concrete foundation. The acoustic laboratory is of similar construction but is supported on a base isolation system consisting of eighteen laminated rubber bearings, fourteen hysteretic dampers, and an unknown number of oil buffers (energy-absorbing devices) as shown in Figure 20. The foundations and the base floors of the two buildings therefore differ fundamentally. Figure 21 shows the locations of the various supports and isolation devices used for the acoustic laboratory and Figure 22 shows a section through the building.

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

Soft vertical rubber springs are utilized to isolate the building and filter out micro-tremors caused by external ground-transmitted vibrations. Fail-safe blocks which limit ultimate system displacements are used to provide secondary protection against primary system failure. The horizontal frequency of the building is 0.5 Hz and the vertical

OILES TECHNICAL CENTER
(BASE-ISOLATED BUILDING)

5F 43.0 (cm/s²)

RC 2-STORY BUILDING
(BASE-FIXED)

2F 83.4 (cm/s²)

at Base 61.0 (cm/s²)

LRB

37.2 (cm/s²) (EW(X)-Direction)

GL-15m

38.0 (cm/s²)

GL-20m

MAXIMUM RECORDED ACCELERATION during East of Tokyo Earthquake
(EW(X)-Direction) (March 18th. 5:34'44"am, 1988)

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

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

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

Figure 19 Kajima Corporation Demonstration Building
(Environmental Engineering Laboratory)

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

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

Figure 22 Cross-Section of Kajima Laboratory

frequency at the design weight is 5 Hz. This low vertical frequency is achieved by thick layers of soft natural rubber. The bearing shown in Figure 23 has a shape factor of 5.2 and consists of five rubber layers each 48 mm thick and four steel insert plates each 5 mm thick. The bearing has a design load of 3.1 MPa and a design shear displacement of 20 cm.

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

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

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

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

The Takenaka Construction Company began a study of base isolation systems in 1982 and in 1984 constructed a full-scale test model using a 550 ton coal silo (Figures 28

165tf RUBBER BEARING

Figure 23 Rubber Bearing in Kajima Building

HYSTERETIC DAMPER

Figure 24 Hysteretic Steel Damper in Kajima Building

Figure 25 Oil Buffer for Kajima Building

Figure 26 Fail-Safe Concrete Blocks in Kajima Isolation System

Figure 27 Sliding Bearings for Passageway between
Isolated and Fixed Laboratories

through 30) mounted on natural rubber bearings provided by the Bridgestone Corporation. The damping in the isolators was very low, equivalent to about 2.4%, for an isolated period of 2.0 sec, and to provide adequate damping viscous fluid dampers (Figure 31) provided by Oiles Industries were included in the system. Free-vibration tests were carried out and the observed damping was about 12% [8]. The performance of the system in the tests was so satisfactory that the company designed and built its own demonstration building using this system. Known as the Funabashi Dormitory (located near the Nishi-Funabashi station about an hour's train ride from Tokyo), the building was designed and built as a research facility. The Takenaka technical research laboratory has instrumented the building and is presently collecting performance data.

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

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

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

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

The base isolation system for the building consists of 14 steel reinforced natural rubber bearings (Figures 34 and 35) provided by the Bridgestone Corporation, one at

Figure 28 Takenaka Company Isolated Test Model (Coal Silo)

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

Figure 20 Section through Takenaka Test Model

Figure 31 Rubber Bearings and Oil Dampers in Takenaka Test Model

Figure 32 Funabashi Dormitory: Takenaka Company Demonstration Building

Figure 33 Cross-Section of Funabashi Dormitory
Showing Architectural Design

Figure 34 Isolators and Dampers in Funabashi Dormitory

Figure 35 Rubber Bearings in Funabashi Dormitory

Figure 36 Viscous Dampers Used in Funabashi Dormitory

each pile location, supplemented by eight dampers. Of the 14 bearings, eight carry a vertical load of 150 tonnes, and six carry a vertical load of 200 tonnes. The 150-tonne bearings are 686 mm in diameter and 186 mm thick, while the 200-tonne bearings are 766 mm in diameter and 166 mm thick. Thus, the vertical pressure on the bearings is 4.06 MPa and 4.34 MPa, respectively.

The viscous polymer based dampers are unique to this system. Each damper is about 150 cm² in plan with a total height of about 30 cm. A viscous polymer is contained within the lower part of the damper. The upper bearing plate of the damping mechanism (see Figure 36) is fixed to the superstructure and positioned on top of the viscous fluid. The drag of the viscous fluid on the bearing plate as it displaces horizontally during an earthquake produces a damping force proportional to the relative velocity. The building, the base isolation system, and its performance are described in reference [9].

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

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

The building, shown in Figure 37, is a reinforced concrete structure with two stories above ground and one at basement level. The building has a total floor area of 550 m², weighs 830 tonnes and is carried on 12 natural rubber isolators provided by the Bridgestone Corporation. They are 435 mm in diameter and 220 mm in height and provide a

Figure 37 Base-Isolated Christian History Museum in Oslo

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

natural period of 1.9 seconds. The system includes 12 steel dampers of the type used in the Yachiyodai House and the Okumura Corporation building. With these features, the period of the isolated structure is 1.3 seconds. Another type of damper is incorporated to provide damping at small displacements when the steel dampers are less effective. This damper consists of lead bars in the form of a *J* attached between the foundation and the superstructure. A view of the complete isolation system is shown in Figure 38 and details are given in reference [10].

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

In a joint research program, Tohoku University and Shimizu Construction Company constructed two full-sized test buildings side by side on the Sendai campus in northern Japan. Construction was completed in May 1986. The buildings are three-story reinforced concrete structures, one conventionally founded and the other base isolated. The dimensions and construction methods for the superstructure were otherwise identical. Each building has a rigid frame structure, and the outer walls consist of light-weight concrete panels. The plan dimensions of each building are 6 m by 10 m, the total combined area being 180 m². The site consists of an 18 m layer of loam with gravel with an average shear wave velocity of 310 m/sec overlying sandy tuff. The site frequency from micro-tremor observations is about 4 Hz.

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

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

Figure 39 Shimuzu Company Test Buildings at Tohoku University

Figure 40 Layout of Test Buildings Showing Isolators and Dampers

Oil Damper Type A

Oil Damper Type B

Figure 41 Dimensions of Elastomeric Bearings and Oil Dampers

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

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

The buildings have been instrumented with 11 strong motion accelerographs installed in the base rock at elevation -27 m, near the surface in the free field, on the base slab, and on the roofs of the two buildings. Thirty earthquakes were recorded [11] between June 1, 1986 and July 20, 1987, most of which were low-level events. The largest acceleration recorded on the ground surface was 981 gals (.091g) and the corresponding peak accelerations at the roofs were 272 gals for the conventional building and 42 gals for the isolated structure. One of the strongest earthquakes recorded at the site was the earthquake of February 6, 1987 (magnitude 6.7, epicentral distance of 168 km). The values of maximum acceleration for this earthquake are shown in Figure 42 and the time histories of acceleration are shown in Figure 43.

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

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

A departure from the previous base-isolated demonstration buildings is that of the Taisei Corporation. This building was due to be completed in late 1988 on the grounds of the Taisei Technological Research Center in Yokohama. In contrast to other base isolation systems which use natural rubber bearings as primary isolation elements, the

X-DIRECTION

Y-DIRECTION

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

Figure 44 Amplification Factors for Test Buildings

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

Figure 46 Relationship between Damping and Shear Strain

isolation system for this building uses a sliding system called TASS (Taisei Shake Suppression System).

The building shown in Figure 47 is a cast-in-place, four-story reinforced concrete structure with a plan area of 323 m². The building is supported on eight TASS bearings which use a teflon/stainless steel sliding surface and carry the entire vertical load of the building. A separate rubber bearing that carries no vertical load is positioned near each sliding bearing (Figures 48 through 50) to act as a restoring force and to control sliding displacement.

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

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

Figure 47 Taisei Corporation Demonstration Building on TASS System

Figure 48 Vertical Load Support and Sliding Components

Figure 49 Horizontal Elastomeric Spring Used in TASS System

Figure 50 TASS System in Demonstration Building

COMMERCIAL BUILDINGS USING BASE ISOLATION

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

- (a) soft ground sites are to be avoided (liquefaction potential);
- (b) basin-shaped areas should be carefully studied for wave amplification and enhancement of long-period motions;
- (c) height is restricted to a maximum of twenty stories;
- (d) eccentricities should not be excessive;
- (e) base shear coefficient should be greater than 0.15 and superstructure should be designed for ductility;
- (f) the design philosophy is to allow for no damage for level 1 earthquakes ($PGV = 25$ cm/sec), minor damage for level 2 earthquakes ($PGV = 50$ cm/sec), and to activate fail-safe systems for level 3 earthquakes ($PGV = 75$ cm/sec);
- (g) time history analyses must be performed using El Centro 1940 NS, Taft 1952 EW, and Hachinohe 1968 NS and EW (long-period motion);
- (h) elastomeric bearings should be designed to carry an average pressure of less than 10 MPa; the allowable average shear strain (extreme lateral displacement divided by total rubber thickness) is a function of average pressure and is generally less than 200%;
- (i) aging and fire-proofing of isolators should be checked; and
- (j) provisions should be made for replacing the isolation mechanism if necessary.

Several construction companies have gained approval for commercial buildings that are now under construction and others are awaiting approval.

Ohbayashi-Gumi Co. has completed an office building in Shibuya for Shimizu Co. This is a five-story (Figure 51) reinforced concrete building of 560 m² on twenty Bridgestone natural rubber isolators with 108 round steel bar dampers. It was granted Ministry of Construction approval in 1987 and was completed in 1988. Details of the building and the isolation system are shown in Figure 52.

Ohbayashi-Gumi has also recently completed an electron microscope laboratory at the Tsukuba Mukizai Research Institute. This building is only one story (Figure 53) and is 834 m² in area. It uses 32 Bridgestone bearings and 48 round steel bars as dampers. The need for isolation in this building is not dictated by seismic design requirements for the building but by the need to provide a high degree of seismic protection to the extremely sensitive instrumentation that it houses. The system provides a building which is both vibration isolated and seismically isolated. The design of the building was approved in 1987 and construction completed in 1988. In late 1987 Ohbayashi-Gumi was granted approval for an eight-story office building of 461 m² which is now under construction.

The Okumura Construction Co. in collaboration with Tokyo Kenchiku Structural Engineering completed a four-story apartment of 225 m² in 1987. It is carried on 12 rubber bearings and has 7 of the coil-type elasto-plastic dampers developed by Okumura. The same collaboration has built a 102 m² research laboratory for the Fujita Corporation in Kanagawa. It is a three-story reinforced concrete building on 4 rubber bearings with the coil-type dampers.

The Okumura Construction Company was granted permission in 1987 to build a three-story apartment house of 192 m² on 10 rubber bearings with 7 coil-type dampers.

The Shimizu Construction Co. has built two base-isolated buildings in addition to its demonstration building. One is a four-story office building of 170 m² on 14 lead-rubber bearings provided by Oiles Industries (see Figure 54). It is located in Ibaraki and is a branch office of the company. The second is an eight-story office building for the Bridgestone Corporation on 12 Bridgestone bearings. The dampers in this building are round steel cantilever bars.

In addition to the design and construction of the Oiles Technical Center, Sumitomo has recently completed the design of a ten-story apartment building and a seven-story office building both of which are to be isolated with lead-rubber bearings. The

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

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

Figure 53 Electron Microscope Building by Ohbayashi-Gumi Ltd

Figure 54 Ibaraki Branch Office of Shimuzu Corporation

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

Located in Nagoya City and to be known as the Asano Building, the seven-story office building is unusual in that it is to be isolated just below the second floor (Figure 56). The total weight to be supported above the isolators is 3800 tonnes. Ten lead-rubber bearings of diameter 800 mm and 900 mm and two horizontal stoppers are to be installed on a mezzanine floor within the first story of the building. The motivation for isolating at this level is to eliminate the trenches around the building which are necessary if the isolators are located in the basement. Such a detail permits the building to extend to the lot line and optimizes the use of expensive commercial real estate.

A consequence of this configuration has been the need to quantify the fire resistance of elastomeric bearings. Sumitomo has conducted tests on various bearings using different fire protection materials wrapped around each. Performance has been observed in three-hour tests at 1000 degrees centigrade in accordance with the Japanese equivalent of the ASTM standard. Excellent resistance has been demonstrated and this work appears to confirm previous work done by the Malaysian Rubber Producers' Research Association of England on unprotected bearings. The design for the Asano Building has been submitted for approval and should be under construction before the end of 1988.

The Fujita Corporation has built a small three-story reinforced concrete building on isolation bearings provided by Oiles Industries Ltd. The bearings are 450 mm in diameter and are 350 mm high, there are four in the isolation system and each carries a load of 85 tonnes. The building was initially constructed as fixed at the base and remained so for six months after which the isolators were installed. Forced vibration tests and observations of its earthquake response were carried out for both isolated and non-isolated states. Small earthquakes that occurred when the building was both fixed and isolated

Figure 55 Itoh Mansion under Construction by
Sumitomo Construction Company

Figure 56 Asano Building under Construction by Sumitomo Construction Company

and comparisons of the responses indicate that the amplification factor for ground acceleration was smaller for the isolated condition. The effectiveness of the system should of course increase as the earthquake intensity increases. The building on the four bearings is shown in Figure 57 and a close-up view of two of the bearings is shown in Figure 58.

Nikken Sekkei is designing a 6-story, 3.5 m by 22.5 m computer center using 28 low-damping natural rubber bearings 700 mm high and 600 mm in diameter. The period of the structure on rubber alone is about 3.0 seconds. The system will incorporate a set of friction dampers designed by Sumitomo Metal Industries, Ltd. The system will include 12 of the friction dampers. The total weight of the building is 7000 tonnes, and the floor area is 4500 m² and it is 24.8 m high. Approval is anticipated in May 1989 and construction will begin shortly after. An artist's rendering of the building is shown in Figure 59.

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

Figure 57 View of Fujita Corporation Base-Isolated Building

Figure 58 Isolators under Fujita Corporation Building

Figure 59 Rendering of Proposed Nikken Sekkei Base-Isolated Building

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

All the companies market base isolation very aggressively using videotapes, high-quality brochures, presentations on national television, technical papers in national and international conferences and numerous other formats. Most companies offer a multifaceted approach to isolation including approaches to seismic isolation, vibration isolation (against traffic-induced vibration), floor isolation systems for computer centers and energy-absorbing devices for interior walls.

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

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

Recent Developments in Base Isolation in Japan

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

Civil Structures

All base isolation projects in Japan are approved by a standing committee of the Ministry of Construction. At the present time (6/30/91) a total of fifty-three buildings have been licensed and the list will no doubt continue to grow in the near future. A wide variety of isolators are being used in Japan and at the present time the most common are the high damping rubber bearing and the lead rubber bearing. The current list of buildings is included in Table I. Many of the completed buildings have experienced earthquakes and their response can, in some cases, be compared with adjacent conventional structures. In such cases, the response of the isolated building has been judged to be highly favorable, although none of the buildings has been struck by a nearby large earthquake.

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

Dai-Ni Computer Center

The Dai-Ni Computer Center is a six-story reinforced concrete building in Sendai which is in a highly seismic area of northern Japan. The computer center houses the computers for the building and production records for the electric power utility.

The building has a floor space of 10,000 square meters (108,000 sq.ft.), and is at the present time, the largest base-isolated building in Japan. It will accommodate a large number of main frame computers and hard disk data storage equipment. It has large internal clear spans to facilitate location of the equipment. The standard bay is 9.5 m (31 ft) by 7.0 m (23 ft). As a result of its six-stories, the large column spacing and the type of equipment in the building, the column loads are very large. There are a total of forty bearings of three different sizes. The isolators are 900 mm (35 in.) diameter, 1000 mm (39 in.) diameter, and 1200 mm (46 in.) diameter and the vertical loads range from 400 metric tons (830 kips) to 800 tons (1760 kips). The thickness of rubber in all of the isolators is 200 mm (8 in.) The details of the structure and the isolators are given in Figs. 1-5.

The design of the structure was based on four standard earthquakes scaled to two levels; namely, Level 1 with a peak velocity of 35 cm/sec and Level 2 with a peak velocity of 50 cm/sec. The system was analyzed using non-linear time history analysis, and it was determined that the superstructure would remain elastic at level 2 inputs, the isolation system would experience 200 mm (8 in.) displacement, and the base shear would be below 0.15'g. The response is much less than would be expected from a fixed base structure, and at this level the computer equipment is expected to continue to function during and after a large earthquake.

There is a large conventionally designed office building next to this building. The buildings are connected by expansion joints designed for 35 cm (14 in.) of movement and the seismic gap between the two buildings is 40 cm (16 in.). If the displacement should exceed 40 cm (14 in.) (which is twice the design displacement) bumpers made of hollow rubber tubes would be engaged. Both the isolated and the adjacent conventional structure are instrumented with strong motion seismographs.

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

Nuclear Structures

Seismic isolation is also gaining support in the Japanese nuclear industry. This is especially true among companies and utilities participating in the development of the

advanced demonstration fast breeder reactor plant (DFBR) whose construction is expected to start in the late 1990's. At the present, an important factor inhibiting the commercialization of FBRs in Japan is the construction cost. The cost of Monju, a demonstration plant with a capacity of 280 MWe, exceeds that of commercial light water plants of the 1000 MWe class [13]. Seismic isolation is expected to significantly reduce the cost of seismic design of FBRs and facilitate the standardization of FBR designs for a wide range of siting conditions. For example, Shiojiri et al. [14], demonstrated that the weight of a pool type FBR reactor vessel can be reduced by half if horizontal isolation is used.

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

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

To date a large number of reduced-scale and full-scale steel-laminated elastomeric bearings composed of high damping and unfilled rubbers as well as lead rubber

bearings have been tested to various load levels up to failure by CRIEPI [16] and other program participants [13] [17]. These tests have demonstrated the effectiveness of elastomeric bearings for isolating nuclear structures and the high reliability with which they can be manufactured in Japan. Another major achievement of this program includes the development of a tentative design response spectrum for seismically isolated FBRs [18].

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

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

Research

Tohoku Test Facility

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

A set of high damping isolators using the compound used in the Foothills Community Law and Justice Center (FCLJC) building, and the Los Angeles Fire Command and Control Center (FCCC) building, and modelled on the isolators developed for the PRISM reactor of General Electric Corp., were installed from the period April 1989 to November 1990. These isolators have a high shape factor and the compound is very non-linear being very stiff at small strains. The isolators were designed to give a frequency of 0.75 Hz at a shear strain of 50%. Over the period of observation, 37

earthquakes were recorded, but all provided very small levels of input generating extremely small displacements in the isolators and resulting in high stiffness in the isolators. The observed frequency of the isolated building at these levels of input is around 2.5 Hz which when compared with the frequency of the unisolated buildings, around 3.5 Hz, shows that the these levels of input, no isolation effect is achieved. This is also shown by the fact that the amplification factors for both the isolated and unisolated buildings is about the same.

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

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License	Name of building	Designer/ Constructor	Type of isolation device	Building	
				Story	(m ²)
1 83/3	Yachiodai house	Tokyo Arch.	RB + PCfriction	RC 2F	114
2 85/11	Tsukuba Reserch bldg.	Unichika, etc/ Okumura	RB + Loop Type Steel	RC 4F	1,330
3 86/1	Tech. Researsch bldg.	Ohbayashi/ Ohbayashi	RB + Hysteretic Steel Bar	RC 5F1B	1,624
	Tohoku Univ. bldg.	Shimizu/ Shimizu	RB + Oil Damper ----- High-damping Rubber	RC 3F	417
4 86/3	Oiles Corp. TC bldg.	Yasui sekkei/ Sumitomo	Lead Rubber	RC 5F	4,765
5 86/4	Taketomo dormitory	Takenaka/ Takenaka	RB + Viscous Damper	RC 3F	1,530
6 86/5	Nishi Chofu Audio bldg.	Kajima/ Kajima	RB + Hysteretic Steel Bar	RC 2F	655
7 86/7	Ohiso Kirisitan museum	Tokyo Arch./ Okumura	RB + Loop Type Steel	RC 2F	293
8 86/12	APT Fukamiya	Tokyo Arch./ Okumura	RB + Loop Type Steel	RC 4F	682
9 87/2	Shibuya Shimizu Daiichi	Ohbayashi/ Ohbayashi	RB + Hysteretic Steel Bar	RC 5F1B	3,385
10 87/2	Research Ins. Exp. bldg.	Fujita/ Fujita	Lead Rubber	RC 3F	395
11 87/6	Tsuchiura bldg.	Shimizu/ Shimizu	Lead Rubber	RC 4F	634
12 87/6	Inorganic material Res.	MOC, etc/ Ohbayashi	RB + Hysteretic Steel Bar	RC 1F	616

RB : Rubber bearing

Table 1 - List of Seismic Isolation Buildings in Japan

License	Name of building	Designer/ Constructor	Type of isolation device	Building (m ²)	
				Story	
13 87/7	Taisei Tech. Ins. bldg.	Taisei/ Taisei	Sliding Plate + Rubber	RC 4F1B	1,029
14 87/7	Koushinzuka No.3 bldg.	Tokyo Arch./ Okumura	RB + Hysteretic Steel Bar	RC 3F	476
15 87/12	DS Toranomon bldg.	Shimizu/ Shimizu, JV	RB + Hysteretic Steel Bar + Oil Damper	RC 8F	3,360
16 88/2	Minami koshigaya bldg.	Sumitomo/ Sumitomo	Lead Rubber	RC 10F	470
17 88/2	Ichinoe dormitory	Kumagai/ Kumagai	RB + Hysteretic Steel Bar	RC 3F	771
18 88/6	14 F - PR bldg.	Tokyo Arch./	RB + Loop Type Steel	RC 14F	16,394
19 88/6	Takenaka clean room bldg.	Takenaka/ Takenaka	Multistage RB + Viscous Damper	RC 2F	406
20 88/6	JAPCO Atagawa bldg.	Taisei/ Taisei	Sliding Plate + Rubber	RC 1F	140
21 88/6	Ogawa bldg.	Kumagai/ Kumagai	High - damping Rubber	RC 4F	1,186
22 88/6	Asano bldg.	Sumitomo/ Sumitomo	Lead Rubber	RC 7F	3,256
23 88/8	Kusukida bldg.	Hazama/ Hazama	High - damping Rubber	RC 4F1B	1,044
24 88/12	Ichikawa company house	Kounoike/ Kounoike	RB + Loop Type Steel + Lead Damper	RC 2F	298
25 88/12	Tohoku electric Daini computer center	Higashi Nippon / Shimizu	High - damping Rubber	RC 6F	10,032
26 88/12	Tokyu machine parts center	Tokyu/ Tokyu	High - damping Rubber	RC 2F	255

RB : Rubber bearing

Table 1 - continued

License	Name of building	Designer/ Constructor	Type of isolation device	Building (m ²)	
				Story	
27	88/12	Gerontology res. ins.	Kume Arch. / Ohbayashi	RB + Hysteretic Steel Bar	RC 2F1B 1,112
28	89/2	M-300 Oiles rest home	Mitsui Home / Mitsui Home	Lead Rubber	W 2F 310
29	89/2	Harvest - hills house	Okumura / Okumura	RB + Loop Type Steel	RC 6F 1,115
30	89/2	Nishi Chofu Reform Plan	Kajima	RB + Hysteretic Steel Bar + Buffer	RC 2F 655
31	89/4	Toshin 24 Omori bldg.	Kajima / Kajima	RB + Hysteretic Steel Bar	SRC 9F1B 7,579
32	89/4	Haseko Experiment house	Haseko / Haseko	RB + Loop Type Steel + Lead Damper	RC 3F 681
33	89/4	Minami Otsuka bldg.	Sumitomo / Sumitomo	Lead Rubber	RC 12F2B 6,020
34	89/4	Tobishima Tech. Ins. bldg.	Tobishima / Tobishima	High - damping Rubber + Rubber Damper	RC 3F 478
35	89/6	C.P. Fukusumi bldg.	Nikken Design	RB + Lead Damper + Friction Damper	RC 5F 4,407
36	89/6	Maeda company house	Maeda / Maeda	Lead Rubber	RC 4F 653
37	89/8	Toho Gas control bldg.	Taisei / Taisei	RB (Friction Type) + Rubber Spring	RC 3F 1,800
38	89/8	Toda Tsudanuma dom.	Toda / Toda	RB + Hysteretic Steel Bar	RC 2F 202
39	89/10	Mitsui Home [M-300]	Mitsui Home / Mitsui Home	Lead Rubber + Back-up Ring	W 2F 214
40	89/10	Koganei company house	Fujita / Fujita	Lead Rubber	RC 3F 714

RB : Rubber bearing

Table 1 - continued

License	Name of building	Designer/ Constructor	Type of isolation device	Building	
				Story	(m ²)
41	89/10	Operation center	Fujita / Fujita	Lead Rubber	RC 2F1B 8,657
42	89/12	Urawa Kogyo Kuki factory	Hazama / Hazama	High - damping Rubber	RC 5F 1,525
43	90/2	Kokudo Kaihatsu Tec. Ins.	Kokudo Kaihatsu/ Kokudo Kaihatsu	RB + Viscous Damper	RC 3F 955
44	90/2	Hiroshima Nokyo	Tokyo Arch. /	Lead Rubber	RC 3F 5,423
45	90/2	C - 1 building	Nihon Sekkei/ Shimizu	Lead Rubber	SRC 7F1B 37,845
46	90/2	Hydrodynamics Res. Ins.	Takenaka / Takenaka	RB + Viscous Damper	RC 3F 628
47	90/4	Mitsui Kashiwa factory	Mitsui / Mitsui	High - damping Rubber	RC 4F 2,187
48	90/4	Tsukuba Tec. Ins.	Hazama / Hazama	RB + Lead Damper + Friction Damper	RC 2F 908
49	90/4	Nishimatsu Yamato dom.	Nishimatsu / Nishimatsu	RB + Hysteretic Steel Bar	RC 8F 1,922
50	90/6	Kawaguchi dom.	Daimatsu / Daimatsu	Lead Rubber	RC 4F 659
51	90/6	PNC computer center	Nikken Sekkei/ Shimizu	RB + Lead Damper	RC 3F 3,309
52	90/6	Ando Tec. Ins.	Ando / Ando	Lead Rubber	RC 3F 1,930
53	90/6	Toyo Rubber company house	Kumagai / Kumagai	RB + Hysteretic Steel Bar	RC 7F 3,520
		Chube Elec. Power Co. administration center	Shimizu + kajima / Shimizu + kajima	High - damping Rubber Lead Rubber	SRC 6F 6,000 6,000

RB : Rubber bearing

Table 1 - continued

LAYOUT OF MRB

Fig. 1: Foundation and Side View of Data Center

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

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Fig. 2: Detail of Rubber Bearing

Bridgestone Bearing
(Made in Japan)

ANL Bearing ('89.4~)
(Made in U.S.A)

ANL Bearing ('90.10~)
(Made in England)

Fig. 3: High Damping Rubber Bearings Installed in Demonstration Building

Fig. 4: Earthquake Responses of Demonstration Building

(a) Equivalent Stiffness

(b) Equivalent Damping Ratio

Fig. 5: Mechanical Characteristics of Both Bearings

WORKSHOP ON SEISMIC BASE ISOLATION FOR DEPARTMENT OF ENERGY FACILITIES

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LECTURE 1

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

Historical Development of Base Isolation

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

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

beneficial to suppress any possible resonance at the isolation frequency.

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

Early Efforts at Base Isolation

The severe earthquake in 1908 in the Italian region of Messina-Reggio killed 160,000 people, mainly by crushing them under fallen masonry. Following this earthquake a commission was [Acad. dei Linc. 1909] formed consisting of practising engineers and university professors. The Commission was charged with recommending structural engineering design methods for the reconstruction of the damaged area. Two proposals arose from the Commission's studies: one suggested separating a building from its foundation by a layer of sand or by use of rollers; the other favored a fixed foundation. The Commission finally decided on the fixed foundation approach with constraints on several construction details and a requirement that the building be able to support at least 8% of its weight as a horizontal force. At exactly the same time, a medical doctor from Scarborough in England applied for a British patent [Calantarientis 1909] on an earthquake-resistant design approach which proposed separating a building from its foundation by a layer of sand or talc. Dr. Calantarientis' interest in earthquake engineering, although presumably amateur, was not superficial, for he designed

wind restraints which would prevent the building from moving in high winds, and, recognizing that reducing damage to the building would require the possibility of a large shift between the building and the ground, he showed how the utilities and access to the building had to be designed for this. Why a medical doctor in England, not known as a seismic region, should be interested in earthquake engineering is not clear, but the name is Armenian and he may have been at the time a recent immigrant from the Near East where earthquakes and the collapse of masonry buildings are a recurrent threat.

Possibly the first person to use this concept in a building was Frank Lloyd Wright. His design of the Imperial Hotel completed in Tokyo in 1921 was in complete contrast to accepted practice at the time and was extremely controversial. Under the site was an 8 ft layer of fairly good soil and below that a layer of soft mud. This layer appeared to Wright as a "good cushion to relieve the terrible shocks. Why not float the building on it?" [Wright 1977]. He tied the building to the upper layer of good soil by closely spaced short piles that penetrated only as far as the top of the soft mud. The building performed extremely well in the devastating 1923 Tokyo earthquake. It was a very highly decorated building with appendages of many kinds, and buildings of this sort generally are badly damaged in earthquakes. The only damage was to statuary in the courtyard of the hotel [Reithman 1980]. The famous architect's intuitive idea of floating the building "as a battleship floats on the ocean" appears to have worked.

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

In a modified approach, called the soft first-story method, the first-story columns yield during an earthquake, producing an energy-absorbing action and controlling the displacements.

However, to produce enough damping the displacements would still have to be several inches and a yielded column has a greatly reduced buckling load, so that column instability and collapse are inevitable.

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

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

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

system was activated during the 1985 Mexico earthquake; the ground motions in the area of the city where the structure is located were of low intensity.

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

Research on Base Isolation at EERC

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

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

Many ideas were explored in pursuit of increased damping; steel energy absorbing devices, hydraulic dampers and oil-extended rubber were tried. A practical system and more convincing demonstration of the concept was achieved in 1978 by the use of bearings made by

commercial techniques and used with a more realistic five-story three-bay model weighing forty tons. A film was made of the tests showing graphically the substantial reductions in acceleration and the rigid-body movement of the structure. This film has been shown perhaps thousands of times to professional and lay audiences and has had a major effect in improving acceptance for the isolation concept.

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

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

For reasons unrelated to the seismic rehabilitation, the proposal to convert the Masonic Hall to a performing arts center was abandoned, and the isolation rehabilitation design was

not implemented. However, the project provided considerable experience in the practical details of isolation systems and a clear picture of the steps needed to design a cost-competitive isolation system using rubber bearings.

Modern Approaches to Seismic Isolation

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

The mechanics of isolation bearings is now very well understood and the design of a system for a building at a specific site is a straightforward task. When a building is analyzed on rubber bearings for earthquake attack, it is usual to model the combined system by a linear viscously damped model. If this model is used, very simple solutions will result. If the fixed-base fundamental frequency of the building is much higher than that of the isolated system, say 3 Hz as compared to 0.5 Hz for the isolated case, the first mode of the isolated building is mainly a rigid body mode with all deformation occurring in the rubber. The second mode has a frequency about 50% to 100% above the first fixed-base frequency. The seismic input to the structure can be treated as an equivalent lateral load which is proportional to the rigid body mode. Since it is a characteristic of a linear vibrating system that all modes are mutually orthogonal, all modes higher than the first will be orthogonal to the input motion, so that if there are high energies in the earthquake ground movement at the frequencies of these higher modes, this energy cannot be transmitted into the building. Thus, the isolation system works not by absorbing these energies but by deflecting them. This is the most attractive feature of the simple rubber isolation system. If other elements are added for the purpose

of increasing the damping or controlling displacement, this simple result no longer holds. Accelerations will then be induced in the higher modes and will produce stresses in the higher levels of the building and cause accelerations in equipment items and other contents of the building.

A simple form of rubber bearing isolation system was used for a three-story school in the small town of Lambesc near Marseilles in France. The school includes three buildings, each separated by a seismic gap. There are 152¹ natural rubber laminated bearings in the isolation system. The school buildings had originally been designed to be built of prefabricated concrete but the seismic code for the region was changed before construction began. The system could not have satisfied the changed seismic requirements without a substantial increase in cost. The use of the isolators allowed the system to satisfy the new code and saved the community a great deal of money. In this building there are no wind restraints or additional elements to enhance the damping and the period of the isolated building is around 1.70 seconds.

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

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

not been used.

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

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

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

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

earthquake a horizontal crack was observed at the foot of the wall as well as a residual displacement of about 6 cm.

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

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

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

The first base-isolated building in the United States is the Foothill Communities Law and Justice Center located in the municipality of Rancho Cucamonga in San Bernardino County.

The building is a thirty-million dollar legal services center for the county. It is four stories high with a full basement and sub-basement for the isolation system. The building sits on 98 isolators which are multilayer natural rubber bearings reinforced with steel plates. The superstructure of the building has a structural steel frame stiffened by braced frames in some bays.

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

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

This high-damping rubber system has been adopted for a recently completed building in Los Angeles. The building is a Fire Department Command and Control Facility for Los Angeles County. The building houses the computer systems for the emergency services program of the county and the facility is required to remain functional after an extreme event. The decision to use seismic isolation for this project was based on a comparison of conventional and isolation schemes designed to provide the same degree of protection. On this basis, the isolated design was estimated to cost less, by 6%, than the conventional design. It is worth noting that in most projects the isolated design is compared with conventional code design and

generally the isolated design is estimated to cost around 5% more. However, code design is a minimal level of design, requiring only that the building should not collapse. It does not mean that there will be no structural damage under strong ground shaking. When equivalent levels of protection are compared, isolated designs are always less expensive. These costs are first costs. Life cycle costs are even more favorable for isolated designs.

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

**DYNAMIC AND FAILURE CHARACTERISTICS
OF BRIDGESTONE ISOLATION BEARINGS**

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Foreword

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Argonne National Laboratory (ANL) has been deeply involved in the development of seismic isolation for use in nuclear facilities for the past decade. Under the funding and direction of the U.S. Department of Energy (USDOE), ANL has been developing methodology needed to evaluate the usefulness and effectiveness of seismic isolation for advanced liquid metal-cooled reactors (LMRs). Because of the thin-walled construction, LMR systems are quite sensitive to seismic loads and the plant designers are constantly faced with the need to balance the desire for thicker vessels/piping to resist earthquake loads against the desirability of thin sections for better thermal performance and lowest cost. Seismic isolation is emerging as one of the more promising strategies to handle these conflicting design requirements. Seismic isolation is also growing rapidly worldwide as a cost-effective and reliable design strategy for a wide range of critical and important facilities (e.g., hospitals, computer centers, etc.).

Shimizu Corporation of Japan has a test facility at Tohoku University in Sendai, Japan. The test facility was constructed in 1986 and has two buildings: one is base isolated and the other is conventionally founded. The buildings are full-size, three-story reinforced concrete structures. The dimensions and construction of the superstructures are identical. For the past several years, Shimizu Corporation has installed a number of different isolation systems in the isolated building at the test facility to study the response of base isolation systems to actual earthquake motions. The Sendai region is a very seismically active area and the test facility has experienced a large number of earthquakes since its inception.

In September 1988, ANL and Shimizu Corporation reached an agreement to initiate a joint study program, utilizing existing strengths of both Shimizu and ANL to study the response of isolated structures under earthquakes. The joint study program is based on an equal-cost sharing by Shimizu and ANL. The funding of ANL's work is provided by NSF. In its simplest terms, Shimizu provides their twin-building facility at Tohoku University, complete with technical staff and data acquisition systems, while ANL supplies the isolator bearings to be installed at the test facility, and ANL performs most of the analysis needed for understanding each significant seismic event and correlating the results with appropriate laboratory tests. In addition, individual bearing tests are conducted by both Shimizu and ANL.

The overall plan of action is as follows. ANL and Shimizu mutually chose a seismic isolation system which has considerable promise not only for nuclear reactor application, but, more importantly, for general application to all types of structures. The system selected must be tested in a laboratory using near- to full-size isolation bearings. Such laboratory tests must be thorough and must include tests of the bearings to ultimate failure. Once the system was selected, Argonne procured the isolation bearings and furnished these to Shimizu Corporation for installation at the test facility and for performance of additional laboratory tests. The system

selected was fully coordinated with the current USDOE program on large-size isolator bearings for advanced LMR plants.

The first isolation system installed in the isolated building at the test facility was a high shape factor-high damping rubber bearing system. The bearing was designed by Argonne and was manufactured by Fluorocarbon (now FURON) of Texas, USA. The bearings were made from a highly damped rubber compound designated by the compounder, LTV Energy Products, as 243-62. The shape factor value for these bearings is 24, the same as those used in the General Electric Company Power Reactor Inherently Safe Module Reactor (PRISM).

The bearings were installed in the test building in April 1989 for earthquake observation and data collection. From April 1989 to July 1990, thirty-seven (37) earthquake motions have been recorded. However, only ten of these earthquakes had recorded values of ground surface maximum accelerations equal to or greater than 3 gal. The seismological data of all those earthquakes such as the hypocenter, the magnitude, and the distances are given in ANL/Shimizu Report, Shmz-004. ANL has performed detailed analysis of three of these earthquakes: one which has the largest amplitude of acceleration; one which has the longest time duration; and one which was the first earthquake that occurred after the installation of the bearings. Numerical simulation results obtained with ANL's computer code--Seismic Isolation System Evaluation Code (SISEC)--have been correlated with earthquake observation data. Comparison of the structural responses of the ordinary and base-isolated buildings are described in ANL/Shimizu Report, ANL-001.

The shear modulus of high damping rubber compounds varies with the magnitude of shear strain. For example, at 4% shear strain, the shear modulus is about 300 psi; at 50% shear strain the shear modulus drops to 120 psi. For shear strain in the range of 100-200%, the shear modulus of the rubber compound has a fairly constant value of about 100 psi. The design frequency of the test building, 0.75 Hz, is based on a shear modulus of 120 psi at 50% of shear strain. As mentioned earlier, the values of maximum acceleration at the ground surface of the 37 recorded earthquakes were quite small ranging from 3 gal to 87.8 gal, with most events at the lower values. Thus, the isolation bearings had only about 5-6% of shear deformations. Since the horizontal stiffness of isolation bearings in the small shear strain range is sufficiently high, the response of the isolated building did not differ too much from that of an unisolated building in small earthquakes. Thus, no definite conclusions can be drawn from the observed earthquake response data and the numerical simulations.

At a ANL/Shimizu meeting in June 1990, it was decided that the high shape factor-high damping isolation bearings installed in the test facility should be replaced by a set of medium shape factor-low shear modulus - high damping isolation bearings to achieve improved response of the building over a wide range of earthquakes, including small earthquakes. A new compound developed by the Malaysian Rubber Producers Research Association (MRPRA) has a shear modulus that is about half that of the shear modulus of the high damping rubber developed by LTV. Also, the value of the shear modulus is less dependent on shear strain. It would improve the response of the test building under small earthquakes, if the isolator bearings were made of low shear modulus compound.

In July 1990, ANL requested Rubber Consultants of the United Kingdom to manufacture ten bearings using the newly developed compound of MRPRA. The design and technical specifications of bearings were provided by ANL. Eight bearings made by Rubber Consultants were shipped to Japan in October 1990: six for installation in the Sendai test building and two for laboratory tests in Japan. Two additional bearings were sent to the Earthquake Engineering Research Center (EERC) at the University of California, Berkeley for laboratory testing in the USA. The design, procurement, installation, and testing of this set of new bearings are all part of the ANL/Shimizu Joint Program. Results of laboratory tests are given in ANL/Shimizu Reports Shmz-006 and ANL-002. Comparison of structural responses of ordinary and base-isolated buildings will be described in subsequent ANL and Shimizu reports when earthquake observations and data collection are completed and analyzed.

Bridgestone Co. of Japan is one of the leading seismic bearing manufacturers in the world. Their bearings have very good performance records. In the early stages, the Bridgestone bearings were manufactured with low damping natural rubber compound and were used with supplementary dampers. As the technology evolved, the supplementary dampers have been eliminated and the damping has been incorporated in the isolators themselves by appropriate compounding. It appears that Bridgestone's high damping isolators are made of a blend of filled natural and synthetic rubbers, whereas in the United States, the high damping compound is a carbon filled natural rubber. To compare the properties of the two different kinds of high damping compounds, ANL has purchased six bearings from Bridgestone: four of which are made of high shear modulus-high damping rubber; two from low shear modulus-high damping rubber. These bearings were ready-made, off-the-shelf bearings. Bridgestone Co. was not informed that their bearings were purchased for the purpose of performing laboratory tests. Laboratory testing was performed at the Earthquake Engineering Research Center (EERC) at the University of California. This report describes the results of laboratory tests of the four high shear modulus, high damping isolator bearings.

A list of ANL and Shimizu reports pertaining to the joint study program is given in Appendix B.

Dynamic and Failure Characteristics of Bridgestone Isolation Bearings

by

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Abstract

High Damping Bridgestone Isolation Bearings are used in several recently completed base isolated buildings in Japan. These isolators carry the vertical load, provide low horizontal stiffness and generate significant damping, thus obviating the need for additional mechanical energy dissipating mechanisms. These bearings are produced in several sizes up to very large diameters and are guaranteed by the manufacturer to be able to sustain shear strains of around 400%.

Four high damping bearings were obtained from Bridgestone and tested at the Earthquake Engineering Research Center to verify their dynamic properties and their failure mechanisms.

The test program included fully reversed horizontal shear tests at 0.5 Hz. These tests involved five cycles of horizontal shear from a low of 5% shear strain to a maximum of 350% shear strain, each series carried out at four different levels of vertical pressure, namely, zero pressure, 500 psi, 1000 psi, and 1500 psi. Each bearing was subjected to five cycles of vertical loading centered at three levels of vertical pressure. Each bearing was subsequently tested under monotonic loading to failure under the four levels of vertical pressure. It was found that the failure strain is very high and is somewhat sensitive to pressure ranging from 475% at the highest pressure, 10.34 MPa (1500 psi), to 560% at zero pressure. At both zero and 3.45 MPa (500 psi) pressure the displacement at failure exceeds the diameter of the bearing.

The most significant aspect of the bearing failure besides the high levels of strain is the fact that the bearing stiffness increases by a factor of 6 when the strain is greater than 250%. The breaking force at failure is at least 6 times the force at 200% strain. It follows that if a structure is designed to be just at yield when the isolators are at 200% strain the structure will have reached collapse before the isolators fail. This means that the weakest element in the overall structural system will not be the isolation system and that the concept of collapse that is assumed to apply to conventional buildings will apply to buildings isolated on elastomeric bearings.

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1. INTRODUCTION

The concept of base isolation is now well accepted and the number of buildings using this earthquake resistant technique is increasing rapidly. It is now generally accepted that a base isolated building will perform better than a conventional fixed base building in moderate or strong earthquakes. Several isolated buildings have experienced moderate earthquakes and have performed well. The major benefit of isolation, in the buildings in which it has been used so far, has been to reduce the effects of seismic forces on contents and internal equipment. In buildings such as hospitals, emergency preparedness centers, computer manufacturing facilities and data centers, the reduction of damage to equipment is of sufficient importance to justify the increased cost of isolated construction. Examples of these buildings have been or are being constructed with isolation in the western United States.

The use of isolation is being most aggressively pursued in Japan where the number of buildings completed, under construction or approved for construction now exceeds fifty. Most of these buildings use natural rubber isolators as part or all of the isolation system and most of these isolators are provided by Bridgestone Corporation. In early designs the Bridgestone isolators were manufactured in a low damping natural rubber compound and were used with supplementary dampers of which a wide variety were developed. As the technology has evolved, the supplementary dampers have been eliminated and the damping has been incorporated in the isolators themselves by appropriate compounding. In the United States the high damping compound is a carbon filled natural rubber, but the Bridgestone high damping rubber appears to be a blend of filled natural and synthetic rubbers. Isolators using this compound have been

installed in several buildings in Japan. The latest application is to a data center for an electric utility. The eight-story building is supported on high damping rubber isolators that carry up to 800 tons (1765 kips) and are 1.2 m (47.2 in.) in diameter. Bearings with this vertical load capacity allow the designer to use large internal spans in the building. In addition because of their large size they are able to sustain very large horizontal displacements eliminating any concerns raised by the uncertainties that might be present when long period systems are proposed.

The uncertainty in long period ground motion has consistently been raised as reason for hesitation in the use of base isolation. Since, the mechanics of fault rupture and ground motion at long periods are well understood, long period ground motion can be predicted more accurately than high frequency motion, from a knowledge of site fault type and soil type. The performance of an isolated building with large elastomeric isolators, which give it a period of around 2 to 2 1/2 seconds, is certain to be better than its performance would be if it were designed as a fixed base structure with a moment resisting frame with the same period.

Another concern in the design of a base isolated building is that the designer of a conventional structure will ensure that there is a high degree of redundancy and ductility in the design thus enabling the structure to survive, albeit with damage, an earthquake of very large magnitude. In a base isolated building this redundancy and ductility do not appear to be present. The isolation system appears to be the only line of defense and if this fails, collapse seems to be inevitable. This concern is often raised, and lack of a clear answer may have led to the abandonment of some promising isolation projects.

It is the purpose of this report to show that with properly designed and manufactured isolators it is possible to ensure that the isolation system will not fail before the structure of the isolated building above it fails. Thus, the ultimate state of the overall structure will be the failure of the structural frame (as in a conventional structure) but the input level of earthquake motion that will produce this failure will be very much larger than that needed to fail the same structural frame if it were conventionally based.

The program described here is the result of a joint effort between the Earthquake Engineering Research Center at the University of California, Berkeley, and the Reactor Engineering Division of Argonne National Laboratory. This report covers tests of the dynamic characteristics and failure mechanisms of Bridgestone high damping isolation bearings. Four test bearings were obtained from Bridgestone Corporation and tested at the Structural Research Laboratory at EERC. The tests have confirmed that the bearings have excellent damping characteristics and that they are able to sustain very large shear deformations before failure occurs. The failure mechanism in all four bearings was tearing of the rubber itself with no failure of the rubber-steel bond. The bearings were tested at four different levels of vertical pressure 0, 3.45, 6.90, 10.34 MPa (0, 500, 1000, 1500 psi). The failure strain was only slightly affected by the pressure but a significant increase in damping resulted from the increase in pressure. The damping factor was fairly constant over the range of strain up to 200% and at the highest pressure of 10.34 MPa (1500 psi) averaged 15% as opposed to around 11% at zero pressure.

The results of the test series provide further evidence of the value of base isolation as a seismic resistant design strategy and should allow the structural engineering profession to use

the technique with confidence. The results indicate that a building using base isolation will not only be protected against the most extreme earthquakes but will perform better than conventional structures under moderate or strong earthquakes.

2. BRIDGESTONE TEST BEARINGS

The dimensions of the test bearings provided by the Bridgestone Corporation are shown in Fig. 2.1. The bearings have 22 thin layers of 2 mm (.079 in.) thick rubber and 21 steel shims which are 0.8 mm (0.031 in.) thick. The shim diameter is 240 mm (9.45 in.) and there is 2.5 mm (0.098 in.) of cover for a total diameter of 245 mm (9.65 in.). The bearings have oversize end plates that permit them to be bolted to the test machine.

The compound used in the bearings is designated KL401 and this identifies it as the high stiffness, high damping rubber which is a blend of natural and synthetic rubber with around 31% carbon filler. With a nominal vertical load of 30 tonnes (66.2 kips) corresponding to 6.52 MPa (945 psi) vertical pressure, these bearings are represented by the manufacturer as providing a horizontal frequency of 0.85 Hz at 100% and a vertical frequency of 29.5 Hz. This very high vertical frequency is a consequence of the extremely thin rubber layers. The vertical frequency of a rubber isolator is controlled by the ratio of the diameter to the thickness of the individual rubber layers. This is expressed by the shape factor S , defined for a circular bearing by

$$S = \frac{\Phi}{4t}$$

where Φ is the diameter and t is the thickness of an individual layer. Here, $S = 30$, which is very high and it has been shown [1] that for this range of shape factors the elementary formulas for vertical stiffness which assume that the rubber is incompressible do not apply. Compressibility of the material must be taken into account. This adds some problems in the prediction of the vertical stiffness since the compressibility of rubber - and particularly filled rubber - is not well understood.

3. TEST FACILITIES

The testing program was carried out on a base isolation test machine at the Earthquake Simulator Laboratory and is described in what follows.

Single Bearing Test Machine

Test Setup

The mechanical characteristic tests for the elastomeric bearings were performed in a test machine capable of subjecting single bearings to simultaneous generalized horizontal and vertical dynamic loadings. Fig. 3.1.1 shows the details of the test machine.

The test machine consists of two rigid reaction frames supporting one horizontal actuator and two vertical actuators. The test bearing is mounted on a force transducer which measures shear force, axial force and bending moment. The force transducer is located on a braced pedestal which is attached to a base block. The base block consists of a concrete block and a wide flange steel beam to provide anchorage to the test floor. Loads are applied to the test bearing by a beam; vertical loads by two vertical actuators to simulate gravity loads on the bearings, and the horizontal loads by a horizontal actuator which acts along the longitudinal axis of the load beam.

The rigid pedestal is placed between the transducer and the test floor to maximize the length of the vertical actuators so that the change in the vertical load component due to horizontal displacements during testing is not significant. Two lateral struts are connected to opposite ends of the load beam to stabilize the setup in the transverse direction. For constant axial loads applied to the test bearing the vertical actuators are under force control, which means that

the vertical load is maintained constant and is independent of the horizontal displacements of the load beam. In addition, the differential displacement between the two vertical actuators is maintained at zero, ensuring that the load beam is kept horizontal.

Control of the hydraulic system is performed by an MTS 443 Controller. The hydraulic actuator can develop a maximum dynamic load of 338.94 kN (76.2 kips) at a hydraulic pressure of 20.67 MN/m^2 (3000 psi). The maximum travel of the horizontal actuator is 152.4 mm (± 6 in.) (i.e., 304.8 mm or 12 in. stroke). Maximum piston velocity is 769.62 mm/sec. (30.3 in/sec.) and the servo-valve on the actuator has a flow capacity of 200 gpm. If displacements in excess of the 152.4 mm (± 6 in.) limit are required, the setup can be modified to obtain a maximum displacement of 254 mm (10 in.) for loading in one horizontal direction only. A maximum load of 1334.4 kN (300 kips) can be applied by the two vertical actuators, each with a servo-valve capacity of 25 gpm.

Signal control is performed by an IBM-AT 286 personal computer. Control signals for both components of loading can be completely general in nature.

Instrumentation

A total of 14 channels of data were recorded for the tests of the elastomeric bearings in the single bearing test machine. The following is a brief description of the components that were measured.

The loads applied by the hydraulic actuators were measured by precalibrated load cells. The compression load on the bearing is calculated by summation of the measured forces in the two vertical actuators. Shear force, axial force and bending moment are measured by a force

transducer located underneath the bearing. A linear variable differential transformer (LVDT) built into the horizontal actuator measures the horizontal displacement of the load beam (this being the lateral displacement of the bearing). Linear potentiometers attached to both of the vertical actuators provide feedback signals for the control of the vertical load. Four direct current differential transformers (DCDTs) measure the vertical displacements of the load beam near the corners of the top of the bearing. To observe any shortening of the pedestal assemblage and other components below the test bearing, two DCDTs were connected between the base block and the bottom bearing plate (situated between the bearing and the force transducer).

4. TEST PROGRAM

(i) Horizontal Tests

Each bearing was subjected to an identical test program with the one variation; that the vertical pressure was different for each bearing. Three sequences of horizontal displacement cycles were imposed on each bearing. The sequences included five cycles of displacement at each level of strain as follows

Sequence 1 $\pm 5\%$, $\pm 10\%$, $\pm 25\%$, $\pm 50\%$, $\pm 75\%$, $\pm 100\%$

Sequence 2 $\pm 100\%$, $\pm 150\%$, $\pm 200\%$, $\pm 250\%$

Sequence 3 $\pm 200\%$, $\pm 250\%$, $\pm 300\%$, $\pm 350\%$

Each bearing was subjected to these loading histories with only a few minutes between sequences to verify that the data collected was secure.

(ii) Vertical Tests

Each bearing was subjected to five cycles of vertical loading centered around 3.45, 6.90 and 10.34 MPa (500, 1000 and 1500 psi) vertical pressure. The variation in the vertical load in each of these tests corresponded to ± 0.69 MPa (± 100 psi). This particular test series is close to the limits of the test system capability. Since the vertical displacements generated by the varying vertical load are of the order of several thousandths of an inch and the minimum resolution of the LVDTs used on the test rig is one thousandth of an inch, the results of this test series are not very accurate. The measurement of the extremely small vertical deformations of the bearing requires very sensitive equipment not available at EERC.

The horizontal and vertical tests were carried out at a frequency of 0.5 Hz.

(iii) Failure Tests

After the dynamic test sequence was completed each bearing was loaded monotonically at a rate of 2.5 in/min. to failure. Failure of a bearing was assumed when the bearing could carry no further load. The vertical load on the bearings during the failure tests was the only variable. The complete test program is given in Table 1.

5. TEST RESULTS

5.1 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 is computed from the secant measured from peak to peak in each hysteresis loop. The equivalent viscous damping is computed using the formula given in Appendix A. This is somewhat different than that used by Bridgestone, but it is felt to be more accurate particularly for cycles below 150% maximum strain when the hysteresis loops are well rounded. For larger strains where the peak force and peak displacement occur at the same point, the differences between the method used here and that used by Bridgestone should be negligible. At each level of maximum strain there is a certain amount of softening between the first cycle and subsequent cycles. The effective stiffness and damping for the first and fifth cycles are given in Table 2.

The loads and displacements were very small in the tests at $\pm 5\%$, both close to the resolution limit of the transducers and the accuracy of the results is less than that at the higher strains. Accordingly, the results at 5% strain have not been included in the table although the hysteresis loops are given in the figures. The force displacement plots for Sequences I - III with zero vertical pressure are shown in Figs. 5.1.1 - 5.1.3, with 500 psi (3.45 Mpa) vertical pressure in Figs. 5.1.4 - 5.1.6, with 1000 psi (6.90 Mpa) vertical pressure in Figs. 5.1.7 - 5.1.9, and with 1500 psi (10.34 Mpa) vertical pressure in Figs. 5.1.10 - 5.1.12. The change in the appearance of the hysteresis loops as the strains increase is clear from these plots. The loops change from being quite elliptical to being elongated with parallel sides and strong hardening.

The reduction in the damping ratios in the higher strain cycles is a consequence of the strong hardening of the material when the strain exceeds 200%. The damping factor quoted is based on modeling the bearing as an elastic and linear viscous element. This model predicts that the energy dissipation is quadratic in the displacement and the effective stiffness is independent of the displacements. In the actual component, the energy dissipation is not quadratic but varies roughly as displacement to the power 1.5 and the effective stiffness increases at the higher strains. Both factors act to reduce the damping factor. This reduction however, is of very little significance to the response of a system using the isolators since at these large strains it is futile to predict the dynamic response of the isolated structure by linear modeling. The important aspect of the bearing behavior is the energy dissipated and this continues to increase while the strong hardening of the elastomer eliminates the possibility of resonant response.

The relationship between the hysteresis loops generated at the various strain levels is shown in Figs. 5.1.13 - 5.1.16 where only the first cycle has been plotted for all four levels of pressure. These diagrams indicate it might be useful to use one model to analyze the response at the design level, for example, for strains that do not exceed 200%, and a second model for extreme events at strains above that. It should be emphasized, however, that the dynamic tests were carried out within a time frame of a few hours and that the large strain cycles followed many cycles at lower levels. Thus, the results at the larger strains are obtained on bearings having undergone many tests in rapid succession. The response of a bearing might be somewhat different if it were to be tested to $\pm 350\%$ strain initially. Unfortunately, neither time nor number of bearings allowed such tests to be made.

5.2 Vertical Load Tests

Each bearing was loaded by five cycles of vertical loading corresponding to $\pm 0.70 \text{ MPa}$ ($\pm 100 \text{ psi}$) centered on 3.45 MPa (500 psi), 6.90 MPa (1000 psi) and 10.34 MPa (1500 psi). The resulting hysteresis loops are shown in Figs. 5.2.1 - 5.2.12. The effective vertical stiffnesses determined from these tests are summarized in Table 3. The measured stiffnesses are sensitive to the average vertical pressure. The stiffness increases with pressure over the range and provides a nearly constant vertical frequency. The area of the loops was measured and the damping is around 8% at 3.5 MPa (500 psi), 12% at 6.90 MPa (1000 psi) and 16% at 10.34 MPa (1500 psi).

5.3 Influence of Pressure on the Dynamic Properties

Isolation bearings are generally used at pressure levels ranging from 5 to 7 MPa (700 to 1000 psi) although for some very conservative designs, pressures as low as 1.72 MPa (250 psi) have been proposed. Bearings with low shape factors and the range of height to width ratios that result from the selection of a low pressure tend to be somewhat sensitive to vertical load due to the fact that the vertical load is a significant fraction of the bearing buckling load. For bearings of the Bridgestone type which have very large shape factors and a squat aspect ratio the buckling load is so much larger than the design vertical load that the influence of vertical load on the horizontal characteristics through the stability of the bearing is negligible. However, the vertical pressure does play a role in the horizontal response and in these bearings the effect must be through an interaction between pressure and shear in the elastomer. This is a reasonable conjecture since for such large shape factors it has been shown [1] that the normal

assumption of incompressible material behavior does not hold. Thus, the vertical load on the bearing produces a volume change in the material and this could interact with the shear behavior to modify the horizontal response.

The dynamic tests were carried out at four levels of vertical pressure, 0, 3.45, 6.90, 10.34 MPa (0, 500, 1000 and 1500 psi). The bearing stiffness at the various peak strains were computed using peak to peak measurements in the resulting hysteresis loops. The stiffnesses are given in Table 2 and shown in Fig. 5.3.1. At smaller strain levels, less than 200%, there is no identifiable effect of pressure on stiffness; above 200% the stiffness is smaller at the higher pressure but the effect is small and can be ignored.

The pressure, however, has a very definite effect on the damping. It is obvious from the hysteresis loops that their area for fixed strain increases with increasing pressure, leading to much larger damping factors.

The damping factors for each pressure level and each peak strain level are computed using the formula developed in Appendix A and are given in Table 2 and shown in Fig. 5.3.2. The damping values over the range of strains up to 200% varies very little with strain and averages 12% for 0 MPa (0 psi), 12% for 3.45 MPa (500 psi), 13.6% for 6.90 MPa (1000 psi) and 15% for 10.34 MPa (1500 psi).

These are very high values for damping especially when it is recalled that for the high damping bearings with the natural rubber compound used in the U.S. [2], the damping drops steadily with strain. The fact that the Bridgestone compound retains its high values of damping over such a wide range of strain is a significant advance in rubber technology. Furthermore, the fact that the higher pressure can generate higher damping factors with no detrimental

effects would encourage the use of higher bearing pressures.

Since the horizontal natural frequency of an elastomeric isolation system is governed by the ratio of pressure to shear modulus, using a higher pressure could allow the use of a stiffer elastomer. Since it is generally easier to increase the damping in the rubber by increasing the stiffness, the result is an increase in damping both from the increased pressure and from the increased stiffness. The alternative to using a stiffer rubber is of course to use a smaller bearing. This may be advantageous since the level of damping is already so high that very little further benefit may be possible by further increase.

5.4 Failure Test Results

Each bearing was loaded monotonically to failure at a rate of 6.35 cm/min. (2.5 in/min.). The appearance of one of the bearings at moderate and at large deformations are shown in Figs. 5.4.1 - 5.4.4. The force displacement curves for the four levels of pressure are superposed in Fig. 5.4.5. The force displacement curve for the bearing with zero vertical load lies below the others but continues to a displacement which is larger than the others. The failure of this bearing was also different from the others in that the force rises continuously to the point of failure whereas for the loaded bearings the curve near failure is more rounded, increasingly so with increasing pressure. In the heaviest loaded bearing the force displacement curve reaches a maximum and begins dropping before the rubber fails. The peak load in the bearing is developed at 507% strain in the case of the bearing with 6.90 MPa (1000 psi) pressure and the rubber fails at 516% strain. For the bearing with 10.34 MPa (1500 psi) the peak load is developed at 473% strain and the rubber fails at 513% strain. In the bearing with zero pres-

sure the failure was instantaneous, in the others it was more gradual. It is interesting to note that the displacement at failure exceeds the diameter of the bearing in the case of the bearings with 0 and 3.45 MPa (0 and 500 psi) vertical pressure.

The deformation of the bearings in terms of stress versus strain is shown in Fig. 5.4.6. This shows that the stress-strain relation of the elastomer is trilinear. An initial stiffness of around 2.276 Mn/m (13 kips/inch) applies for a strain of up to around 15%, the bearing has a stiffness of 350 kN/m (2 kips/inch) up to a strain of around 250% and the third segment of the force displacement curve has a stiffness of 2.276 Mn/m (13 kips/inch) from 250% to failure.

These results have implications for the analysis of buildings using this type of isolation system. The initial stiffness is significant for estimating the response of the isolated building to wind load and ground borne vibrations from traffic and other sources of low level vibration. For seismic loading at design levels the initial stiffness can be ignored and the system analyzed as a linear system with viscous damping. For an estimate of the response under earthquake loading beyond the design level, the system can be analyzed using a bilinear model with the second stiffness much larger than the first. The analysis will be non-linear and a realistic modeling of the energy dissipation can be made by combining viscous damping and hysteretic damping.

The strongly stiffening character of the force displacement curves also has implications for design. For example, for the bearing with 6.90 MPa (1000 psi) pressure the force level at 200% strain which could safely be considered as the design level is around 44.5 kN (10 kips) and at the maximum of the force displacement curve it is 275.7 kN (62 kips). Thus, if a superstructure is designed just to be at yield level at the design strain of the isolators, the

superstructure would have to be able to sustain a base shear of six times the yield level in order that the isolation system would fail before the superstructure would collapse. Although this is possible it is highly unlikely. With this approach to isolation design it can be anticipated that the isolation system is not the weakest link. Current codes that provide design requirements for base isolated buildings namely OSHPD [3], SEAOC [4] and UBC 1991 [5] are very conservative in comparison to corresponding codes for conventionally based structures. Some of this conservatism could be relaxed on the basis of these test results.

The failure mechanism of the bearings at all levels of pressure was tearing of the rubber. No evidence of bond failure was observed. Photographs of the failed surfaces in all four bearings illustrating the rippled failure surface characteristic of tearing of the rubber are shown in Figs. 5.4.7 - 5.4.10.

6. CONCLUSION

The test program has confirmed the extremely high quality of the Bridgestone isolators. They have been shown to be capable of extremely large shear strains before failure even under high levels of vertical pressure. The fact that the failure mechanism is only slightly affected by pressure and that the stiffness is unaffected by pressure while the damping is increased by pressure can be used in the design process to lead to smaller isolators than those currently used.

The damping in the bearings was found to be somewhat less than that claimed by Bridgestone. However, the difference in damping is not an important issue. It is not generally realized that the effect of an isolation system of the elastomeric type is the result of the difference between the fixed base frequency of the superstructure and the overall frequency of the isolated building. When the ratio between these is large the participation factor of the higher modes that involve structural deformation is low. This means that the displacement that results from the earthquake input is almost entirely in the isolation system. Also, if there is large energy in the earthquake input at these higher frequencies this energy is not transmitted into the structure as it is in conventionally based structures. This effect is achieved even in the absence of damping. Damping is only needed to counteract the possibility of resonance at the isolation frequency. In fact, damping can be viewed as a contaminant of the isolation process. The more damping there is in the isolation system - and especially if this damping is produced by non-linear mechanisms such as mechanical dampers - the more energy leaks into the higher modes thus counteracting the beneficial effects of the isolation system. Further, the difference in the dynamic response of a structure on a system with the 17% equivalent viscous damping

and that obtained in the test program, 15%, is completely negligible. A two percent difference is only significant for systems with almost no damping.

The entire search to find damping mechanisms to accompany isolation systems has been a misplaced effort. Much more important for the design of practical isolation systems is the fact that the elastomer hardens very strongly after a level of shear strain has been exceeded. In these tests the stiffness increases by a factor of six beyond 250% shear strain. Thus, if 200% shear strain is taken as the nominal design level, at which level of base shear the superstructure is just at the yield point, then the base shear must increase by at least a factor of six before the isolation system fails. This means that the failure mechanism for the entire structure will be in the superstructure and not in the isolators. This means that conceptually, collapse of an isolated structure is no different from that of a conventional structure with, however, the proviso that the level of earthquake impact that produces the failure must be much greater for the isolated structure due to the large displacement capacity of the isolation system. The strong hardening response of the system beyond the design level means that if the system exceeds this level, the period shortens significantly and the system becomes non-resonant.

The results of this program should give the designer confidence that base isolated buildings can be designed and built and that their performance in moderate and strong earthquakes will be superior to conventional buildings and that the isolation system will provide damage control to the structure and to internal equipment and contents. The performance in very strong earthquakes much beyond those assumed for design will also be superior to conventional structures but will have the same type of collapse mechanisms. These results should lead to increased confidence in the use of this promising new technology for buildings of all

types and particularly for buildings housing sensitive internal equipment such as data centers, computer manufacturing facilities, telephone exchanges and buildings that must be able to continue operation after strong earthquakes such as emergency control centers and hospitals. The possibility of the use of isolation for nuclear power facilities is also enhanced by the results of this test program.

References

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- [3] "Requirements for the Design and Construction of Base Isolated Hospitals," Office of Statewide Health Planning and Development (OSHPD), Hospital Building Safety Board, State of California, Sacramento, (1987).
- [4] "General Requirements for the Design and Construction of Seismically Isolated Structures," Structural Engineers Association of California, October 10, 1989.
- [5] "Earthquake Regulations for Seismic-Isolated Structures," UBC, Chapter 23, ICBO, (1990).

Table 1

TEST MATRIX FOR BRIDGESTONE BEARINGS						
FILENAME	EERC	BEARING	VERTICAL	TIME	RATE	REMARKS
TEST#	ID	ID	PRESSURE	(sec)		
901023.01	# 1	HR030-2	500 psi	13	120	Vertical test (+100psi)SPV=20
901023.02	"	"	1000 psi	"	"	"
901023.03	"	"	1500 psi	"	"	"
901023.04	"	"	1000 psi	70	60	Horizontal test(strain:5-100%)SPH=154.6
901023.05	"	"	"	55	"	Horizontal test(strain:100-250%)SPH=395
901023.06	"	"	"	"	"	Horizontal test(strain:200-350%)SPH=536
901023.07	# 4	HR030-5	500 psi	13	120	Vertical test(+100psi)SPV=20
901023.08	"	"	1000 psi	"	"	"
901023.09	"	"	1500 psi	"	"	"
901023.10	"	"	1500 psi	70	60	Horizontal test(strain:5-100%)SPH=158
901023.11	"	"	"	"	"	Horizontal test(strain:100-200%)SPH=395
901023.12	"	"	"	"	"	Horizontal test(strain:200-350%)SPH=546
901023.13	# 2	HR030-3	500 psi	13	120	Vertical test(+100psi)SPV=20
901023.14	"	"	1000 psi	"	"	"
901023.15	"	"	1500 psi	"	"	"
901023.16	"	"	1500 psi	70	60	Horizontal test(strain:5-100%)SPH=158
901023.17	"	"	"	"	"	Horizontal test(strain:100-200%)SPH=395
901023.18	"	"	"	"	"	Horizontal test(strain:200-350%)SPH=546
901023.19	# 3	HR030-4	500 psi	13	120	Vertical test(+100psi)SPV=20
901023.20	"	"	1000 psi	"	"	"
901023.21	"	"	1500 psi	"	"	"
901023.22	"	"	0 psi	70	60	Horizontal test(strain:5-100%)SPH=158
901023.23	"	"	"	"	"	Horizontal test(strain:100-200%)SPH=395
901023.24	"	"	"	"	"	Horizontal test(strain:200-350%)SPH=546
901023.25	"	"	"	290	25	Shear failure,SPH=1000
901024.01	# 2	HR030-3	500 psi	5	120	Zero
901024.02	"	"	"	290	25	Shear failure,SPH=1000
901024.03	# 1	HR030-2	1000 psi	5	120	Zero
901024.04	"	"	"	290	25	Shear failure,SPH=1000
901024.05	# 4	HR030-5	1500 psi	5	120	Zero
901024.06	"	"	"	290	25	Shear failure,SPH=1000

Table 2
TEST PROGRAM
HORIZONTAL SHEAR TEST

BEARING ID: HRO30-4 EERC ID: No. 3 VERTICAL PRESSURE: 0 PSI													
File No.	901023.22					901023.23				901023.24			
γ (=)	10%	25%	50%	75%	100%	100%	150%	200%	250%	200%	250%	300%	350%
K_{eff} (1st cy)	12.58	9.22	7.28	5.89	5.14	4.86	4.53	4.41	4.62	3.28	3.84	4.5	4.49
Kips-in (5th cy)	11.21	8.75	6.54	5.45	4.67	4.61	3.94	3.80	3.74	3.13	3.48	3.52	3.62
β_1 (1st cy)	12.25	12.24	12.01	11.95	12.09	13.18	12.17	11.39	9.97	13.64	10.02	8.74	7.89
(%) (5th cy)	13.39	13.14	12.06	12.09	13.86	12.01	12.70	11.61	10.61	13.22	10.47	9.4	8.4
W_D (1st cy)	0	1	4	6	10	11	21	35	50	31	43	62	70
Kips-in (5th cy)	0	1	3	6	11	10	19	30	43	29	39	52	60
G (1st cy)	311	228	180	145	127	120	112	109	114	81	95	111	111
psi (5th cy)	277	216	162	135	115	114	97	94	93	77	86	87	90

BEARING ID: HRO30-3 EERC ID: No. 2 VERTICAL PRESSURE: 500 PSI													
File No.	901023.16					901023.17				901023.18			
γ (=)	10%	25%	50%	75%	100%	100%	150%	200%	250%	200%	250%	300%	350%
K_{eff} (1st cy)	13.11	10.22	7.77	6.43	5.56	5.27	4.89	4.6	4.51	3.4	3.79	4.46	4.5
Kips-in (5th cy)	12.7	9.41	7.08	5.79	4.98	4.94	4.15	3.78	3.71	3.18	3.49	3.6	3.62
β_1 (1st cy)	10.22	12.91	12.6	12.15	12.8	13.39	12.85	12.11	10.59	14.2	10.68	9.06	8.42
(%) (5th cy)	11.21	11.65	11.87	12.77	14.03	13.56	13.55	12.89	11.88	14.49	11.5	9.98	9.06
W_D (1st cy)	0	1	4	7	11	12	24	38	53	33	44	63	77
Kips-in (5th cy)	0	1	3	7	11	11	22	34	48	31	42	56	67
G (1st cy)	324	253	192	159	137	130	121	114	114	84	94	110	111
psi (5th cy)	314	232	175	143	123	122	103	93	92	79	85	89	89

BEARING ID: HR030-2													
EERC ID: No. 1													
VERTICAL PRESSURE: 1000 PSI													
File No.	901023.04					901023.05				901023.06			
$\gamma (=)$	10%	25%	50%	75%	100%	100%	150%	200%	250%	200%	250%	300%	350%
K_{eff} (1st cy)	12.66	9.61	7.36	5.89	5.08	4.87	4.58	4.53	4.83	3.31	3.91	4.65	4.75
Kips in (5th cy)	12.71	8.6	6.32	5.41	4.72	4.55	3.9	3.77	3.84	3.13	3.54	3.7	3.71
β_1 (1st cy)	13.71	12.99	12.48	13.24	14.01	15.45	13.86	12.24	10.57	15.68	10.99	9.11	8.29
(%) (5th cy)	13.73	11.08	14.07	15.1	14.91	15.03	14.66	13.02	11.31	15.25	11.74	10.18	9.46
W_D (1st cy)	0	1	4	7	11	13	24	38	55	50	45	64	78
Kips-in (5th cy)	0	1	3	7	11	12	22	34	48	31	43	57	70
G (1st cy)	313	257	182	145	125	120	113	112	119	82	97	115	117
psi (5th cy)	314	213	156	134	117	113	96	93	96	77	87	92	92

BEARING ID: HR030-5													
EERC ID: No. 4													
VERTICAL PRESSURE: 1500 PSI													
File No.	901023.10					901023.11				901023.12			
$\gamma (=)$	10%	25%	50%	75%	100%	100%	150%	200%	250%	200%	250%	300%	350%
K_{eff} (1st cy)	15.76	11.04	8.56	6.13	5.09	4.66	4.38	4.28	4.33	2.97	3.52	4.15	4.2
Kips in (5th cy)	11.31	8.45	6.32	5.08	4.48	4.36	3.74	3.46	3.44	2.77	3.14	3.23	3.19
β_1 (1st cy)	12.71	14.52	14.34	14.84	16.39	17.13	15.77	14.06	12.81	18.58	13.24	10.85	9.92
(%) (5th cy)	14.64	14.75	14.52	15.97	16.52	16.16	16.96	15.46	13.67	17.61	14.11	12.4	11.59
W_D (1st cy)	0	0	1	4	7	13	26	41	60	37	50	70	85
Kips-in (5th cy)	0	1	4	7	12	12	24	36	50	33	48	62	75
G (1st cy)	389	273	211	151	126	115	108	106	107	73	87	103	104
psi (5th cy)	279	209	156	125	111	108	92	86	85	68	78	80	79

Table 3
BRIDGESTONE BEARING TEST
VERTICAL TEST PROGRAM

	BEARING ID: HR030-4 EERC ID: No. 3			BEARING ID: HR030-3 EERC ID: No. 2			BEARING ID: HR030-2 EERC ID: No. 1			BEARING ID: HR030-5 EERC ID: No. 4		
PRESSURE psi	FILENAME	Keff Kips-in	Keff Tf-cm	FILENAME	Keff Kips-in	Keff Tf-cm	FILENAME	Keff Kips-in	Keff Tf-cm	FILENAME	Keff Kips-in	Keff Tf-cm
500	901023.19	1343.50	236.24	901023.13	1519.35	267.16	901023.01	1225.34	215.46	901023.07	1385.17	243.57
1000	901023.20	3084.68	542.41	901023.14	3246.07	570.79	901023.02	3455.01	607.53	901023.08	3221.99	566.55
1500	901023.21	4023.93	707.57	901023.15	4213.51	740.90	901023.03	4245.8	746.58	901023.09	4406.49	774.84

Pressure	Δp	P	K_v kips/inch	f_v
psi	psi	kips	Avg.	Hz.
500	± 100	35.1	1368.34	19.51
1000	± 100	70.1	3251.94	21.30
1500	± 100	105.2	4222.43	19.82

Fig. 3.1.1 Single Bearing Test Machine

BRIDGESTONE-BEARING TEST
DIAGNOSTIC_TEST-HORIZONTAL_SHEAR

Filename: 901023.22
BENC ID: NO. 3
BEARING ID: HRO30- 4
Vertical load: 2 Kips

Shear strain Stiffness
48 1 7.28 Kips/in
97 1 5.14 Kips/in

Shear Modulus
180 psi
127 psi

Fig. 5.1.1 Force Displacement Plots for Sequence I - Vertical Zero Pressure

BRIDGESTONE-BEARING TEST
DIAGNOSTIC TEST-HORIZONTAL SHEAR

Filename: 901023.23
ESRC ID: No. 3
DEARING ID: H030-4
Vertical load: 1 Kips

Shear strain
146 %
245 %

Stiffness
4.53 Kips/in
4.62 Kips/in

Shear Modulus
112 psi
114 psi

Fig. 5.1.2 Force Displacement Plots for Sequence II - Vertical Zero Pressure

Fig. 5.1.3 Force Displacement Plots for Sequence III - Vertical Zero Pressure

Fig. 5.1.4 Force Displacement Plots for Sequence I - 500 psi (3.45 Mpa)
Vertical Pressure

BRIDGESTONE-BEARING TEST
DIAGNOSTIC TEST-HORIZONTAL SHEAR

Filename: 901023.17
EERC ID: No. 2
BEARING ID: BR030-3
Vertical load: 35 Kips

Shear strain Stiffness Shear Modulus
147 % 4.89 Kips/in 121 psi
245 % 4.61 Kips/in 114 psi

Fig. 5.1.5 Force Displacement Plots for Sequence II - 500 psi (3.45 Mpa) Vertical Pressure

BRIDGESTONE-BEARING TEST
DIAGNOSTIC_TEST-HORIZONTAL_SHEAR

Filename: 901023.18
BERC ID: H0. 2
BEARING ID: H030- 3
Vertical load: 29 Kips

Shear strain	Stiffness	Shear Modulus
196 %	3.40 Kips/in	84 psi
245 %	3.79 Kips/in	94 psi
294 %	4.46 Kips/in	110 psi
339 %	4.50 Kips/in	111 psi

Horizontal load/deflection

Fig. 5.1.6 Force Displacement Plots for Sequence III - 500 psi (3.45 Mpa) Vertical Pressure

BRIDGESTONE-HEARING TEST
DIAGNOSTIC TEST-HORIZONTAL_SHEAR

Filename: 901023.04
ENG ID: NO. 1
HEARING ID: HRO30- 2
Vertical load: 72 Kips

Shear Modulus
182 psi
125 psi

Shear strain Stiffness
47 k
96 k
736 Kips/in
508 Kips/in

Fig. 5.1.7 Force Displacement Plots for Sequence I - 1000 psi (6.90 Mpa) Vertical Preload

File name: 901023.05
 BEARING ID: 00030- 2
 Vertical load: 72 Kips
 BRIDGE STONE-BEARING TEST
 DIAGNOSTIC TEST-HORIZONTAL SHEAR
 Shear strain Stiffness Shear Modulus
 147 4.58 Kips/in 113 psi
 244 4.83 Kips/in 119 psi

Fig. 5.1.8 Force Displacement Plots for Sequence II - 1000 psi (6.90 Mpa) Vertical Pressure

BRIDGESTONE-DEANNING TEST
DIAGNOSTIC_TEST-HORIZONTAL_SHEAR

Filename: 901023.06
EERC ID: No. 1
DEANNING ID: HR030- 2
Vertical load: 72 Kips

Shear strain	Stiffness	Shear Modulus
193 %	3.31 Kips/in	82 psi
241 %	3.91 Kips/in	97 psi
209 %	4.65 Kips/in	115 psi
333 %	4.75 Kips/in	117 psi

Horizontal load/deflection

Fig. 5.1.9 Force Displacement Plots for Sequence III - 1000 psi (6.90 Mpa) Vertical Pressure

Fig. 5.1.10 Force Displacement Plots for Sequence I - 1500 psi (10.34 Mpa) Vertical Pressure

BRIDGESTONE-BEARING TEST
DIAGNOSTIC TEST-HORIZONTAL SHEAR

Filename: 901023.11
ENC ID: No. 4
BEARING ID: H030-5
Vertical load: 106 Kips

Shear strain Stiffness Shear Modulus
147 438 Kips/in 108 psi
246 433 Kips/in 107 psi

Fig. 5.1.11 Force Displacement Plots for Sequence II - 1500 psi (10.34 Mpa) Vertical Pressure

BRIDGESTONE-BEARING TEST
DIAGNOSTIC TEST-HORIZONTAL SHEAR

Filament: 901023.12
BENC ID: No. 4
BEARING ID: UR030- 5
Vertical load: 01 Kips

Shear strain	Stiffness	Shear Modulus
196 μ	297 Kips/in	73 psi
245 μ	352 Kips/in	87 psi
294 μ	415 Kips/in	103 psi
339 μ	420 Kips/in	104 psi

Horizontal load/deflection

Fig. 5.1.12 Force Displacement Plots for Sequence III - 1500 psi (10.34 Mpa) Vertical Pressure

BRIDGESTONE-BEARING TEST
SHEAR LOAD - DEFLECTION

BEARING ID: HR030-4

VERT' PRESSURE: 0 PSI

STRAIN RANGES: 5% 10% 25% 50% 75% 100% 150% 200% 250% 300% 350%

Horizontal load/deflection

Fig. 5.1.13 First Cycle Force Displacement Plots @ Zero Vertical Pressure

BRIDGESTONE-BEARING TEST
SHEAR LOAD - DEFLECTION

BEARING ID: HR030-3
VERT PRESSURE: 500 PSI
STRAIN RANGES: 5% 10% 25% 50% 75% 100% 150% 200% 250% 300% 350%
Horizontal load/deflection

Fig. 5.1.14 First Cycle Force Displacement Plots @ 500 psi (3.45 Mpa)
Vertical Pressure

BRIDGESTONE-BEARING TEST
SHEAR LOAD - DEFLECTION

BEARING ID: HR030-2

VERT PRESSURE: 1000 PSI

STRAIN RANGES: 5% 10% 25% 50% 75% 100% 150% 200% 250% 300% 350%

Horizontal load/deflection

Fig. 5.1.15 First Cycle Force Displacement Plots @ 1000 psi (6.90 Mpa)
Vertical Pressure

BRIDGESTONE-BEARING TEST SHEAR LOAD - DEFLECTION

BEARING ID: HR030-5
VERT PRESSURE: 1500 PSI
STRAIN RANGES: 5% 10% 25% 50% 75% 100% 150% 200% 250% 300% 350%
Horizontal load/deflection

Fig. 5.1.16 First Cycle Force Displacement Plots @ 1500 psi (10.34 Mpa)
Vertical Pressure

BRIDGESTONE-BEARING TEST
VERTICAL STIFFNESS

Filename: 901023.01
EERC ID: No. 1
BEARING ID: HR030- 2
Ave.Vert.Pressure: 507 PSI

Cycle Vertical Stiffness
first 1225.34 Kips/in 215.46 tf/cm
fifth 1233.91 Kips/in 216.97 tf/cm

Fig. 5.2.1 Force Displacement Plots for Bearing #1 under Vertical Cycles Centered at 500 psi (3.45 Mpa)

BRIDGESTONE-BEARING TEST VERTICAL STIFFNESS

Filename: 901023.13
 EERC ID: NO. 2
 BEARING ID: HR030-3
 Avg. Vert. Pressure: 492 PSI

Cycle Vertical stiffness
 first 1519.35 Kips/in 267.16 tf/cm
 fifth 1466.04 Kips/in 257.79 tf/cm

Fig. 5.2.2 Force Displacement Plots for Bearing #2 under Vertical Cycles Centered at 500 psi (3.45 Mpa)

BRIDGESTONE-BEARING TEST VERTICAL STIFFNESS

Filename: 901023.19
ESRC ID: No. 3
BEARING ID: HR030- 4
Ave. Vert. Pressure: 490 PSI

Cycle Vertical stiffness
first 1343.5 Kips/in 236.24 tf/cm
fifth 1344.56 Kips/in 236.43 tf/cm

Fig. 5.2.3 Force Displacement Plots for Bearing #3 under Vertical Cycles Centered at 500 psi (3.45 Mpa)

BRIDGESTONE-BEARING TEST
VERTICAL STIFFNESS

Filename: 901023.07
ERRC ID: No. 4
BEARING ID: HR030-5
Avg. Vert. Pressure: 500 PSI

Cycle Vertical Stiffness
first 1305.17 Kips/in 243.57 tf/cm
fifth 1351.76 Kips/in 237.69 tf/cm

Fig. 5.2.4 Force Displacement Plots for Bearing #4 under Vertical Cycles Centered at 500 psi (3.45 Mpa)

BRIDGESTONE-BEARING TEST
VERTICAL STIFFNESS

Filename: 901023.02
 EERC ID: No. 1
 BEARING ID: HR030- 2
 Ave.Vert.Pressure: 1035 PSI

Cycle	Vertical Stiffness
first	3455.01 Kips/in
fifth	3664.69 Kips/in
	607.53 tf/cm
	644.4 tf/cm

Fig. 5.2.5 Force Displacement Plots for Bearing #1 under Vertical Cycles Centered at 1000 psi (6.90 Mpa)

BRIDGESTONE-BEARING TEST
VERTICAL STIFFNESS

Filename: 901023.14
BRC ID: No. 2
BEARING ID: BR030-3
Avg. Vert. Pressure: 999 PSI

Cycle Vertical Stiffness
first 3246.07 Kips/in 570.79 tf/cm
fifth 3240.06 Kips/in 569.73 tf/cm

Fig. 5.2.6 Force Displacement Plots for Bearing #2 under Vertical Cycles Centered at 1000 psi (6.90 Mpa)

BRIDGESTONE-BEARING TEST
VERTICAL STIFFNESS

Filename: 901023.20
ESRC ID: No. 3
BEARING ID: HR030- 4
Ave.Vert.Pressure: 990 PSI

Cycle Vertical Stiffness
first 3084.68 Kips/in 542.41 tf/cm
fifth 3102.98 Kips/in 545.63 tf/cm

Fig. 5.2.7 Force Displacement Plots for Bearing #3 under Vertical Cycles Centered at 90 psi (6.90 Mpa)

BRIDGESTONE-BEARING TEST
VERTICAL STIFFNESS

Filename: 901023.08
SERC ID: No. 4
BEARING ID: HR030- 5
Ave.Vert.Pressure: 1007 PSI

Cycle Vertical Stiffness
first 3221.99 Kips/in 566.55 tf/cm
fifth 3193.67 Kips/in 561.57 tf/cm

Fig. 5.2.8 Force Displacement Plots for Bearing #4 under Vertical Cycles Centered at 1000 psi (6.90 Mpa)

BRIDGESTONE-BEARING TEST
VERTICAL STIFFNESS

Filename: 501023.03
SERC ID: No. 1
BEARING ID: BR030- 2
Ave.Vert.Pressure: 1508 PSI

Cycle Vertical Stiffness
first 4245.8 Kips/in 746.58 tf/cm
fifth 4160.69 Kips/in 731.61 tf/cm

Fig. 5.2.9 Force Displacement Plots for Bearing #1 under Vertical
Cycles Centered at 1500 psi (10.34 Mpa)

BRIDGESTONE-BEARING TEST
VERTICAL STIFFNESS

Filename: 901023.15
EERC ID: No. 2
BEARING ID: UR030- 3
Avg. Vert. Pressure: 1510 PSI

Cycle Vertical Stiffness
first 4213.51 Kips/in 740.9 tf/cm
fifth 4293.43 Kips/in 754.96 tf/cm

Fig. 5.2.10 Force Displacement Plots for Bearing #2 under Vertical Cycles Centered at 1500 psi (10.34 Mpa)

BRIDGESTONE-BEARING TEST
VERTICAL STIFFNESS

Filename: 901023.21
EERC ID: No. 3
BEARING ID: HR030- 4
Ave. Vert. Pressure: 1514 PSI

Cycle Vertical Stiffness
first 4023.93 Kips/in 707.57 tf/cm
fifth 3656.36 Kips/in 642.93 tf/cm

Fig. 5.2.11 Force Displacement Plots for Bearing #3 under Vertical Cycles Centered at 100 psi (10.34 Mpa)

BRIDGESTONE-BEARING TEST
VERTICAL STIFFNESS

Filename: 901023.09
EERC ID: No. 4
BEARING ID: UR030-5
Ave. Vert. Pressure: 1514 PSI

Cycle Vertical Stiffness
first 4406.49 Kips/in 774.84 tf/cm
fifth 4270.79 Kips/in 750.97 tf/cm

Fig. 5.2.12 Force Displacement Plots for Bearing #4 under Vertical
Cycles Centered at 1500 psi (10.34 Mpa)

BRIDGESTONE-BEARING TEST EFFECTIVE STIFFNESS - STRAIN PLOTS

pressure P=0 psi: □——□
 pressure P=500 psi: ○-----○
 pressure P=1000 psi: △-----△
 pressure P=1500 psi: ◇-----◇
 Effective stiffness/Strain

Fig. 5.3.1 Influence of Vertical Load and Strain on Bearing Stiffness

BRIDGESTONE-BEARING TEST DAMPING - STRAIN PLOTS

pressure P=0 psi: □—
pressure P=500 psi: ○—
pressure P=1000 psi: △—
pressure P=1500 psi: ◇—

Damping/Strain

Fig. 5.3.2 Influence of Vertical Load and Strain on Bearing Damping

Fig. 5.4.1 Configuration of Bearing before Failure Tests

Fig. 5.4.2 Bearing Appearance at 200% Strain

Fig. 5.4.3 Bearing Appearance at 300% Strain

Fig. 5.4.4 Bearing Appearance at 500% Strain

BRIDGESTONE-BEARING TEST SHEAR FAILURE

BEARING ID: HR030-4 HR030-3 HR030-2 HR030-5
 VERT PRESSURE: 0 psi 500 psi 1000 psi 1500 psi
 STRAIN RANGE : 561 % 550 % 507 % 473 %

Horizontal load/deflection

Fig. 5.4.5 Force Displacement Curves for Bearings in Failure Tests

BRIDGESTONE-BEARING TEST SHEAR STRESS-STRAIN DIAGRAMS

BEARING ID: HR030-4 HR030-3 HR030-2 HR030-5
VERT PRESSURE: 0 psi 500 psi 1000 psi 1500 psi
STRAIN RANGE : 561 % 550 % 507 % 473 %

Shear Stress/Shear Strain

Fig. 5.4.6 Stress-Strain Curves for Bearings in Failure Tests

Fig. 5.4.7 Appearance of Failure Surface @ Zero Vertical Pressure

Fig. 5.4.8 Failed Bearing (held apart for clarity) @ 1000 psi (6.90 Mpa)
Vertical Pressure

Fig. 5.4.9 Appearance of Failure Surface @ 1000 psi (6.90 Mpa) Vertical Pressure

Fig. 5.4.10 Failed Bearing @ 1500 psi (10.34 Mpa) Vertical Pressure

Appendix A

Estimation of Equivalent Viscous Damping

When the force displacement characteristic of an element or component is modeled by the standard structural model, the force is related to the displacement through

$$F = K\Delta + C\dot{\Delta}$$

If such an element is subjected to a sinusoidal displacement

$$\Delta = \Delta_0 \sin \omega t$$

the resulting force is

$$F = K\Delta_0 \sin \omega t + C\omega\Delta_0 \cos \omega t$$

The maximum force is given by the value of ωt for which $\dot{F} = 0$, namely,

$$\tan \omega t = \frac{K}{C}\omega$$

and the force

$$F_{\max} = K\Delta_0 \frac{K/C \omega + C \omega/K}{(1 + (K/C\omega)^2)^{1/2}}$$

or

$$F_{\max} = K\Delta_0 (1 + (\frac{C\omega}{K})^2)^{1/2}$$

The effective stiffness K_{eff} defined by

$$K_{eff} = \frac{F_{\max}}{\Delta_{\max}} = K (1 + (\frac{C\omega}{K})^2)^{1/2}$$

The total energy dissipated by the element in a complete cycle is given by

$$W_D = \int_0^{2\pi/\omega} F(t) \dot{\Delta}(t) dt = \pi C \omega \Delta_0^2$$

Thus, in a sinusoidal test we measure the area of the hysteresis loop and K_{eff} and from these we can determine K and C .

If the K and C were associated with a vibrating mass m in a single degree of freedom system

$$K/m = \omega_0^2 \text{ and } C/m = 2\omega_0\beta$$

where ω_0 is the fundamental frequency and β is the fraction of critical damping. It follows that

$$C = \frac{2\beta}{\omega_0} K$$

and thus

$$W_D = 2\pi K \frac{\omega}{\omega_0} \Delta_0^2 \beta$$

leading to an estimate of β as

$$\beta = \frac{\omega_0}{\omega} \frac{W_D}{2\pi K \Delta_0^2}$$

This formula shows that the viscous model predicts that the energy dissipation should be linear in the frequency and quadratic in the displacement. Many practical materials do not show these dependencies, in fact, for most materials used in structural applications the damping is independent of frequency over the frequency ranges that occur. Such materials are best modeled by the hysteretic model that relates the force F to the displacement Δ through

$$F = K(1 + i \tan \delta) \Delta$$

where $\tan \delta$ is the loss factor. Such a model is at a specific frequency ω equivalent to

$$F = K\Delta + \frac{1}{\omega} K \tan \delta \dot{\Delta}$$

and thus using the results of the previous analysis we have

$$K_{eff} = K(1 + \tan^2 \delta)^{1/2}$$

$$W_D = \pi K \tan \delta \Delta_0^2$$

In carrying out a sinusoidal test the quantities that can be measured easily and accurately are the area encompassed by the hysteresis loop, namely, W_D and the rectangular area that circumscribes the loop, namely, $4 F_{max} \Delta_{max}$. Knowing these we have

$$K_{eff} = \frac{F_{max}}{\Delta_{max}}$$

$$K = \frac{K_{eff}}{(1 + \tan^2 \delta)^{1/2}}$$

$$\frac{\tan \delta}{(1 + \tan^2 \delta)^{1/2}} = \frac{W_D}{\pi F_{max} \Delta_{max}} = \frac{a}{\pi}$$

These are easily solved to give

$$\tan \delta = \frac{a}{\pi} \left(1 - \frac{a^2}{\pi^2}\right)^{-1/2}$$

and

$$K = K_{eff} \left(1 - \frac{a^2}{\pi^2}\right)^{1/2}$$

These are the formulas used in this report to estimate the damping in these bearings, with the additional assumption that for the vibrating single degree of freedom system the loss factor $\tan \delta$ is twice the damping factor β . It should be noted that Bridgestone uses the formula

$$\beta = \frac{W_D}{2\pi K \Delta_0^2}$$

where the K is computed from the force where $\Delta = \Delta_{max}$, i.e., $K = F(\Delta_{max})/\Delta_{max}$. The results should be the same but the estimation of $F(\Delta_{max})$ is more difficult and less accurate than the estimate of F_{max} and $F_{max} \Delta_{max}$. Using a smaller value for $F(\Delta_{max})$ results in a higher β . The choice of $F(\Delta_{max})$ is a subjective one when the curve is a well rounded one. When the hysteresis loop becomes sharply pointed as it does at higher strains, $F(\Delta_{max})$ and F_{max} coincide and the results should be identical.

Appendix B

List of ANL and Shimizu Joint Study Reports

compiled by

Ralph W. Seidensticker and Yao-Wen Chang

Argonne National Laboratory

<u>Report No.</u>	<u>Title/Authors/Date</u>
ANL/Shimizu Shmz-001	Outline of Study and Site Experiment of Base-Isolated Building in Sendai T. Kuroda, M. Kobatake, Y. Nitta, H. Watanabe, M. Saruta, and K. Okada, July 1989
ANL/Shimizu Shmz-002	Outline of Study and Test on Characteristics of Individual Bearing T. Kuroda, M. Kobatake, Y. Nitta, H. Watanabe, M. Saruta, and K. Okada, November 1989
ANL/Shimizu Shmz-003	Recent Earthquake Observation Records at Sendai T. Kuroda, M. Kobatake, Y. Nitta, H. Watanabe, M. Saruta, and K. Okada, August 1990
ANL/Shimizu Shmz-004	Final Report on Joint US/Japanese Program on Seismic Isolation Study T. Kuroda, M. Kobatake, Y. Nitta, H. Watanabe, M. Saruta, and K. Okada, January 1991
ANL/Shimizu Shmz-005	Phase II Outline of Study and Site Experiment (UK Bearing) T. Kuroda, M. Kobatake, Y. Nitta, H. Watanabe, M. Saruta, and S. Hayashi, January 1991
ANL/Shimizu Shmz-006	Phase II Outline of Study and Test on Characteristics of Individual Bearing T. Kuroda, M. Kobatake, Y. Nitta, K. Ohta, H. Watanabe, M. Saruta, and S. Hayashi, June 1991
ANL/Shimizu ANL-001	Comparison of SISEC Code Simulation with Earthquake Data of Ordinary and Base-Isolated Buildings. C. Y. Wang and J. Gvildys, August 1990
ANL/Shimizu ANL-002	EERC Lab Report 89-101, Performance Evaluation of ANL Sendai Bearing J. M. Kelly, I. D. Aiken, and D. Clyde, September 1989
ANL/Shimizu ANL-003	Dynamic and Failure Characteristics of Bridgestone Bearings J. M. Kelly, March 1991

ANL/Shimizu Mechanical Characteristics of Low Modulus High Damping Rubber Isolators for
ANL-004 a Base Isolated Demonstration Building
J. M. Kelly, June 1991

WORKSHOP ON SEISMIC BASE ISOLATION FOR DEPARTMENT OF ENERGY FACILITIES

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LECTURE 3

Linear Theory of Base Isolation

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

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

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

and

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

It is convenient to work with relative displacements:

$$v_s = u - u_b \quad (3.3)$$

and

$$v_b = u_b - u_g \quad (3.4)$$

in terms of which the equations of motion become:

$$m \ddot{v}_b + m \ddot{v}_s + c \dot{v}_s + k v_s = -m \ddot{u}_g \quad (3.5)$$

and

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

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

$$M \ddot{v}_b + c_b \dot{v}_b + k_b v_b = -M \ddot{u}_g \quad (3.7)$$

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

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

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

where

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

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

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

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

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

(iii) the damping factors for the structure and isolation system, β_s and β_b , respectively,

where $\beta_s = \frac{c}{2m\omega_s}$ and $\beta_b = \frac{c_b}{2M\omega_b}$, are of the same order of magnitude as ε .

The undamped natural modes of the system

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

are given by the two equations:

$$(-\omega_n^2 + \omega_b^2) \phi_b^n + (-\gamma \omega_n^2) \phi_s^n = 0 \quad (3.9a)$$

and

$$(-\omega_n^2) \phi_b^n + (-\omega_n^2 + \omega_s^2) \phi_s^n = 0 \quad (3.9b)$$

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

The characteristic equation for ω_n is

$$(1 - \gamma) \omega_n^4 - (\omega_b^2 + \omega_s^2) \omega_n^2 + \omega_b^2 \omega_s^2 = 0 \quad (3.10)$$

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

$$\omega_{1,2}^2 = \frac{1}{2(1-\gamma)} \left\{ (\omega_s^2 + \omega_b^2) \pm \left[(\omega_s^2 + \omega_b^2)^2 - 4(1-\gamma) \omega_s^2 \omega_b^2 \right]^{1/2} \right\} \quad (3.11)$$

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

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

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

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

and

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

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

$$\omega_b^* = \omega_b \quad (3.15)$$

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

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

The mode shape, ϕ^1 , is given by:

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

or:

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

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

$$\phi^1 = \left\{ \begin{array}{c} 1 \\ \omega_b^2 / \omega_s^2 \end{array} \right\} \quad (3.18)$$

To the same order of ϵ , we find that:

$$\phi^2 = \left\{ \begin{array}{c} 1 \\ -\frac{1}{\gamma} \left[1 - (1-\gamma) \frac{\omega_b^2}{\omega_s^2} \right] \end{array} \right\} \quad (3.19)$$

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

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

$$v_b = q_1 \phi_b^1 + q_2 \phi_b^2 \quad (3.20)$$

and

$$v_s = q_1 \phi_s^1 + q_2 \phi_s^2 \quad (3.21)$$

and reduce the basic matrix equation to the two equations:

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

and

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

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

$$L_n = \frac{\phi^{nT} M^* r^*}{\phi^{nT} M^* \phi^n} \quad (3.24)$$

The computation of L_1 involves the following matrix multiplications

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

where

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

Retaining only terms to order ε , we have:

$$L_1 = 1 - \gamma \varepsilon \quad (3.26)$$

For L_2 , the same computations give:

$$L_2 M_2 = M + m a \quad (3.27)$$

where

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

and

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

Since

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

and

$$M_2 = M \frac{(1-\gamma)[1-2(1-\gamma)\epsilon]}{\gamma}$$

then

$$L_2 = \gamma \epsilon \quad (3.28)$$

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

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

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

$$M\ddot{x} + C\dot{x} + Kx = P \quad (3.29)$$

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

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

which can be interpreted as:

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

where

$$K.E. = \frac{1}{2} \dot{\mathbf{x}}^T \mathbf{M} \dot{\mathbf{x}} \quad \text{is the kinetic energy;}$$

$$P.E. = \frac{1}{2} \mathbf{x}^T \mathbf{K} \mathbf{x} \quad \text{the potential energy;}$$

$$\dot{W} = \dot{\mathbf{x}}^T \mathbf{P} \quad \text{the external work rate; and}$$

$$D = \dot{\mathbf{x}}^T \mathbf{C} \dot{\mathbf{x}} \quad \text{is the dissipation of energy.}$$

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

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

where the ϕ^n are given by:

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

The global quantities take the form:

$$K.E. = \frac{1}{2} \sum_{i=1}^N \dot{q}_i^2 \quad (3.33)$$

and:

$$P.E. = \frac{1}{2} \sum_{i=1}^N \omega_i^2 q_i^2 \quad (3.34)$$

and:

$$\dot{W} = \sum_{i=1}^N p_i \dot{q}_i \quad (3.35)$$

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

$$\phi^{nT} M \phi^m = \delta_{mn} \quad (3.36)$$

with:

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

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

$$\sum_{m=1}^N \sum_{n=1}^N \dot{q}_m \dot{q}_n A_{mn} \quad (3.37)$$

where $A_{mn} = \phi^{mT} C \phi^n$.

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

$$D = \sum_m \sum_n \dot{q}_m \dot{q}_n A_{mn} \quad (3.38)$$

and:

$$D = \sum_n 2\omega_n \beta_n \dot{q}_n^2 \quad (3.39)$$

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

$$2\omega_n \beta_n = \phi^{nT} C \phi^n \quad (3.40)$$

The corresponding terms in the simple isolation model are:

$$2\omega_n^* \beta_n^* = \frac{\phi^{nT} C^* \phi^n}{\phi^{nT} M^* \phi^n} \quad (3.41)$$

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

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

$$2\omega_b^* \beta_b^* = 2\omega_b \beta_b (1 - 2\gamma\epsilon) \quad (3.42)$$

and

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

Using the result that

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

we have

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

Since

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

we have

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

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

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

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

and set

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

in the modal equations. We then have

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

and

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

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

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

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

$$A_s = \frac{\varepsilon}{2 \beta_b^*} \quad (3.51)$$

For the amplification of base motion we have:

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

which to the same order in ε is:

$$A_b = \frac{1 - \gamma \varepsilon}{2 \beta_b^* (1 - \frac{3}{2} \gamma \varepsilon)} = \frac{1}{2 \beta_b^*} (1 + \frac{1}{2} \gamma \varepsilon) \quad (3.53)$$

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

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

and

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

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

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

The exact expressions for all values of ω are:

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

and

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

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

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

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

and

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

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

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

and

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

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

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

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

and

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

where now:

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

and

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

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

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

and

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

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

$$S_A(\omega, \beta) = \omega S_V \quad (3.66)$$

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

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

and

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

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

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

and when unisolated this is:

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

When isolated this becomes:

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

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

$$\begin{aligned} C_s &= \varepsilon \left\{ \frac{(1 - \gamma\varepsilon)^2 S_A^2(\omega_b^*, \beta_b^*)}{(1 - \gamma\varepsilon)^2 \varepsilon^2} + \frac{[1 - (1 - \gamma)\varepsilon]^2 S_A^2(\omega_s^*, \beta_s^*)}{\frac{(1 + \gamma\varepsilon)^2}{(1 - \gamma)^2}} \right\}^{1/2} \\ &= \left\{ S_A^2(\omega_b^*, \beta_b^*) + \varepsilon^2 (1 - \gamma)^2 (1 - 2\varepsilon) S_A^2(\omega_s^*, \beta_s^*) \right\}^{1/2} \end{aligned} \quad (3.72)$$

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

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

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

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

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

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

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

BASE AND FLOOR DISPLACEMENT

BASE AND FLOOR DISPLACEMENT

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

**MECHANICAL CHARACTERISTICS OF
LOW MODULUS HIGH DAMPING NATURAL RUBBER ISOLATORS FOR
A BASE ISOLATED DEMONSTRATION BUILDING**

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Foreword

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Argonne National Laboratory

Argonne National Laboratory (ANL) has been deeply involved in the development of seismic isolation for use in nuclear facilities for the past decade. Under the funding and direction of the U.S. Department of Energy (USDOE), ANL has been developing methodology needed to evaluate the usefulness and effectiveness of seismic isolation for advanced liquid metal-cooled reactors (LMRs). Because of the thin-walled construction, LMR systems are quite sensitive to seismic loads and the plant designers are constantly faced with the need to balance the desire for thicker vessels/piping to resist earthquake loads against the desirability of thin sections for better thermal performance and lowest cost. Seismic isolation is emerging as one of the more promising strategies to handle these conflicting design requirements. Seismic isolation is also growing rapidly worldwide as a cost-effective and reliable design strategy for a wide range of critical and important facilities (e.g., hospitals, computer centers, etc.).

Shimizu Corporation of Japan has a test facility at Tohoku University in Sendai, Japan. The test facility was constructed in 1986 and has two buildings: one is base isolated and the other is conventionally founded. The buildings are full-size, three-story reinforced concrete structures. The dimensions and construction of the superstructures are identical. For the past several years, Shimizu Corporation has installed a number of different isolation systems in the isolated building at the test facility to study the response of base isolation systems to actual earthquake motions. The Sendai region is a very seismically active area and the test facility has experienced a large number of earthquakes since its inception.

In September 1988, ANL and Shimizu Corporation reached an agreement to initiate a joint study program, utilizing existing strengths of both Shimizu and ANL to study the response of isolated structures under earthquakes. The joint study program is based on an equal-cost sharing by Shimizu and ANL. The funding of ANL's work is provided by NSF. In its simplest terms, Shimizu provides their twin-building facility at Tohoku University, complete with technical staff and data acquisition systems, while ANL supplies the isolator bearings to be installed at the test facility, and ANL performs most of the analysis needed for understanding each significant seismic event and correlating the results with appropriate laboratory tests. In addition, individual bearing tests are conducted by both Shimizu and ANL.

The overall plan of action is as follows. ANL and Shimizu mutually chose a seismic isolation system which has considerable promise not only for nuclear reactor application, but, more importantly, for general application to all types of structures. The system selected must be tested in a laboratory using near- to full-size isolation bearings. Such laboratory tests must be thorough and must include tests of the bearings to ultimate failure. Once the system was selected, Argonne procured the isolation bearings and furnished these to Shimizu Corporation for installation at the test facility and for performance of additional laboratory tests. The system

selected will be fully coordinated with the current USDOE program on large-size isolator bearings for advanced LMR plants.

The first isolation system installed in the isolated building at the test facility was a high shape factor-high damping rubber bearing system. The bearing was designed by Argonne and was manufactured by Fluorocarbon (now FURON) of Texas, USA. The bearings were made from a highly damped rubber compound designated by the compounder, LTV Energy Products, as 243-62. The shape factor value for these bearings is 24, the same as those used in the General Electric Company Power Reactor Inherently Safe Module Reactor (PRISM).

The bearings were installed in the test building in April 1989 for earthquake observation and data collection. From April 1989 to July 1990, thirty-seven (37) earthquake motions have been recorded. However, only ten of these earthquakes had recorded values of ground surface maximum accelerations equal to or greater than 3 gal. The seismological data of all those earthquakes such as the hypocenter, the magnitude, and the distances are given in ANL/Shimizu Report, Shmz-004. ANL has performed detailed analysis of three of these earthquakes: one which has the largest amplitude of acceleration; one which has the longest time duration; and one which was the first earthquake that occurred after the installation of the bearings. Numerical simulation results obtained with ANL's computer code--Seismic Isolation System Evaluation Code (SISEC)--have been correlated with earthquake observation data. Comparison of the structural responses of the ordinary and base-isolated buildings are described in ANL/Shimizu Report, ANL-001.

The shear modulus of high damping rubber compounds varies with the magnitude of shear strain. For example, at 4% shear strain, the shear modulus is about 300 psi; at 50% shear strain the shear modulus drops to 120 psi. For shear strain in the range of 100-200%, the shear modulus of the rubber compound has a fairly constant value of about 100 psi. The design frequency of the test building, 0.75 Hz, is based on a shear modulus of 120 psi at 50% of shear strain. As mentioned earlier, the values of maximum acceleration at the ground surface of the 37 recorded earthquakes were quite small ranging from 3 gal to 87.8 gal, with most events at the lower values. Thus, the isolation bearings had only about 5-6% of shear deformations. Since the horizontal stiffness of isolation bearings in the small shear strain range is sufficiently high, the response of the isolated building did not differ too much from that of an unisolated building in small earthquakes. Thus, no definite conclusions can be drawn from the observed earthquake response data and the numerical simulations.

At a ANL/Shimizu meeting in June 1990, it was decided that the high shape factor-high damping isolation bearings installed in the test facility should be replaced by a set of medium shape factor-low shear modulus - high damping isolation bearings to achieve improved response of the building over a wide range of earthquakes, including small earthquakes.

A new compound developed by the Malaysian Rubber Producers Research Association (MRPRA) has a shear modulus that is about half that of the shear modulus of the high damping rubber developed by LTV. Also, the value of the shear modulus is less dependent on shear strain. It would improve the response of the test building under small earthquakes, if the isolator bearings were made of low shear modulus compound. In July 1990, ANL requested Rubber

Consultants of the United Kingdom to manufacture ten bearings using the newly developed compound of MRPRA. The design and technical procurement specifications of bearings were performed and prepared by ANL. Eight bearings made by Rubber Consultants were shipped to Japan in October 1990: six for installation in the Sendai test building and two for laboratory tests in Japan. Two remaining bearings were sent to the Earthquake Engineering Research Center (EERC) at the University of California, Berkeley for laboratory testing in the USA. The design, procurement, installation, and testing of this set of new bearings are all part of the ANL/Shimizu Joint Program.

Unfortunately, one of the bearings sent to EERC for laboratory testing was accidentally destroyed during a calibration test. However, EERC has successfully completed all the necessary tests which were specified by ANL on the remaining bearing. This report describes the results of the tests by EERC of the medium shape factor-low modulus-high damping bearing made by Rubber Consultants.

A list of reports pertaining to the ANL/Shimizu Joint Program is given in Appendix B.

v

Mechanical Characteristics of Low Modulus High Damping
Natural Rubber Isolators for a Base Isolated Demonstration Building

by

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Abstract

This report describes the results of a test program of a seismic isolator bearing made recently by Rubber Consultants of the United Kingdom using a new high damping compound developed by the Malaysian Rubber Producers' Research Association (MRPRA). This bearing was one of a group of bearings procured by ANL for installation in an isolated building in the test facility at Tohoku University in Sendai, Japan. The high damping isolator has a shear modulus of 60 psi at 50% shear strain, about half that of the shear modulus of the high damping rubber used for most base isolated buildings in the United States.

The isolator bearing was tested to over 400% shear strain without debonding. It was not possible to test the isolator to failure due to the displacement limit of the actuator. The bearing has a diameter of 14.76 in. and was tested at a maximum horizontal displacement of 20 in. The isolator was carrying the design vertical load during the test. The results of this test show that it is possible to compound an elastomer with a low modulus, high damping, high strength and excellent bondability. This set of properties is desirable in a seismic isolator bearing as it is expected to result in a seismic isolation system which performs equally well during low to moderate earthquakes.

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Set #1 tests were conducted on 04/04/91

Set #2 tests were conducted on 03/25/91

1. INTRODUCTION

Since 1986, a test facility at Tohoku University in Sendai in northern Japan has been used to study the response of base isolation systems to earthquakes. The region is a highly seismic part of the country and the test facility has experienced a large number of earthquakes since its inception.

The test facility has two buildings, one is base isolated and the other conventionally founded. The buildings are full-sized three-story reinforced concrete structures and the dimensions and construction of the superstructures were identical. The construction of the test facility was carried out and funded by the Shimizu Corporation.

A number of different isolation systems have been installed in the isolated building in the test facility. The first was a system using low damping natural rubber bearings provided by Bridgestone Rubber Co., combined with hydraulic dampers. The second system consisted of a high damping rubber bearing system also provided by Bridgestone Rubber Co., but with no additional dampers, and the third a high damping natural rubber system manufactured by Fluorocarbon Corporation (now Furon) of the United States. These latter bearings were installed as part of a joint Argonne National Laboratory (ANL)/Shimizu Corp. project. ANL's work is funded by the National Science Foundation (NSF).

The Furon bearings were replaced in October 1990 by a set of low-modulus, high-damping natural rubber bearings manufactured by Rubber Consultants of the United Kingdom using a new compound developed by the Malaysian Rubber Producers Research Association (MRPRA). The design, manufacture and installation of these isolators was also part of the joint ANL/Shimizu Corp. joint program. The technical procurement specifications for these bearings were prepared by ANL with the assistance of Professor James M. Kelly.

Two of the ten bearings made by Rubber Consultants for use in the Sendai test facility were sent to the Earthquake Engineering Research Center (EERC) at the University of California, Berkeley, for testing. This report describes the results of that test program.

The new rubber compound has a shear modulus that is about half that of the shear modulus of the high damping rubber used for most base isolated buildings used in the United States. In comparison with the previously used compound, the damping in the new rubber compound is less dependent on strain and exceeds that in the previous compound at strains of 100% and above. The modulus is dependent on strain but less so than the earlier compound. These are all quite desirable characteristics.

The most important aspect of the strength of an elastomeric isolator is the ability of the elastomer to bond to the steel reinforcing plates. This elastomer was found to have excellent bonding characteristics. The isolator was tested to over 400% strain without any evidence of debonding. It was not possible to test the isolator to failure due to the displacement limit of the actuator in the EERC test facility. The maximum stroke at which the bearing was tested was 20 in. which exceeds the bearing diameter by 5 in. It is important to note that the isolator was carrying the full design vertical load during the test.

The results of the test show that it is possible to compound an elastomer with low shear modulus high damping, high strength and excellent bondability. This makes it possible to design isolators with a large plan dimension, which provides high stability under earthquakes much larger than the design earthquake. In addition, the margin of reserve strength will provide the designer with a high level of confidence that the isolation system will not be the weakest element in the seismic resistance of an isolated building.

2. SENDAI TEST BEARINGS

The dimensions of the Sendai test bearing is shown in Fig. 2.1. The bearing has 12 layers of 10 mm (.394 in.) thick rubber and 11 steel shims which are 3.25 mm (0.128 in.) thick. The shim diameter is 350 mm (13.78 in.) and there is a 12.5 mm (0.49 in.) of cover for a total diameter of 375 mm (14.76 in.). The bearing has oversize flange plates (not shown in Fig. 1) that permit it to be bolted to the test machine. With a nominal vertical load of 92.61 kips corresponding to 621 psi vertical pressure, these bearings provide a horizontal frequency of 0.40 Hz at 100% shear strain.

The vertical frequency of a rubber isolator is controlled by the ratio of the diameter to the thickness of the individual rubber layers. This is expressed by the shape factor S, defined for a circular bearing by

$$S = \frac{\Phi}{4t}$$

where Φ is the diameter and t is the thickness of an individual layer. Here, $S = 8.75$, which is a moderate shape factor. (It should be noted that the original bearings used in the joint ANL/Shimizu programs had a shape factor of about 24.)

3. TEST FACILITIES

The testing program was carried out on a base isolation test machine at the Earthquake Simulator Laboratory and is described in the following section.

Single Bearing Test Machine

Test Setup

The mechanical characteristic tests of the elastomeric bearings were performed in a test machine capable of subjecting single bearings to simultaneous generalized horizontal and vertical dynamic loadings. Fig. 3.1.1 shows the details of the test machine.

The test machine consists of two rigid reaction frames supporting one horizontal actuator and two vertical actuators. The test bearing is mounted on a force transducer which measures shear force, axial force and bending moment. The force transducer is located on a braced pedestal which is attached to a base block. The base block consists of a concrete block and a wide flange steel beam to provide secure anchorage to the test floor. Loads are applied to the test bearing by a beam; vertical loads are provided by two vertical actuators which simulate gravity loads on the bearings, while the horizontal loads are provided by a horizontal actuator which acts along the longitudinal axis of the load beam.

The rigid pedestal in the test setup is placed between the transducer and the test floor to maximize the length of the vertical actuators so that the change in the vertical load component due to horizontal displacements during testing is not significant. Two lateral struts are connected to opposite ends of the load beam to stabilize the setup in the transverse direction. For constant axial loads applied to the test bearing the vertical actuators are under force control, which means that the vertical load is maintained constant and is independent of the horizontal displacements of the load beam. In addition, the differential displacement between the two vertical actuators is maintained at zero, ensuring that the load beam is kept horizontal.

The horizontal hydraulic actuator can develop a maximum dynamic load of 538.21 kN (121 kips) at a hydraulic pressure of 20.67 MN/m² (3000 psi). The maximum travel of the horizontal actuator is ± 254 mm (± 10 in.) (i.e., 508 mm or 20 in. stroke). The servo-valve on

the actuator has a flow capacity of 170 gpm. If displacements in excess of the ± 254 mm (± 10 in.) limit are required, the setup can be modified to obtain a maximum displacement of 508 mm (20 in.) for loading in one horizontal direction only. A maximum load of 1334.4 kN (300 kips) can be applied by the two vertical actuators, each with a servo-valve capacity of 25 gpm. Signal control is performed by an IBM-AT 286 personal computer. Control signals for both components of loading can be completely general in nature.

Instrumentation

A total of 18 channels of data were recorded for the tests of the elastomeric bearings in the single bearing test machine. The following is a brief description of the components that were measured.

The loads applied by the hydraulic actuators were measured by precalibrated load cells. The compression load on the bearing is calculated by summation of the measured forces in the two vertical actuators. Shear force, axial force and bending moment are measured by a force transducer located underneath the bearing. A linear variable differential transformer (LVDT) built into the horizontal actuator measures the horizontal displacement of the load beam (this being the lateral displacement of the bearing). Linear potentiometers attached to both of the vertical actuators provide feedback signals for the control of the vertical load.

Four direct current differential transformers (DCDTs) measure the vertical displacements of the bearing. They are attached to the top and bottom steel plates that are directly bolted to the bearings. To observe any shortening of the pedestal assemblage and other components below the test bearing, two DCDTs were connected between the base block and the bottom bearing plate (situated between the bearing and the force transducer).

Sendai Test Facility

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

A general view of the building is shown in Fig. 3.2.1, the structure on the right being base isolated. Plan and elevation views are shown in Fig. 3.2.2. The first isolation system used in the test facility consisted of 6 laminated unfilled elastomeric bearings and 12 oil dampers. The oil dampers had an equivalent damping ratio of 15% for a wide range of shear strain including small strains.

Static tests can be performed on the isolated building to confirm the bearing stiffness and damping properties. In addition, forced-vibration tests can be performed to confirm the dynamic properties of the buildings using vibration exciters placed on the center of the roof slab to obtain the natural frequencies and damping ratios. The natural frequencies of the unisolated building were found to be 3.6 Hz in the x-direction and 4.4 Hz in the y-direction. The frequencies of the isolated building with the first system were 0.72 and 0.73 Hz, respectively, and the damping ratios were 16% in the x-direction and 15% in the y-direction. The damping of the bearings alone was less than 2%. The test results for the first system were reported in several publications, including Ref. [1].

The building had been instrumented with 11 strong motion accelerographs in the base slab, floors and roofs of the buildings while the first system was in place. In addition, there were four strong motion accelerographs installed in the base rock at elevation -27 m and near the surface in the free field. Thirty earthquakes were recorded with the first system, most of which were low-level events. The largest acceleration recorded on the ground surface was 98.1 gals (0.10 g) and the corresponding peak accelerations at the roofs were 272 gals for the unisolated building and 42 gals for the isolated structure. One of the strongest earthquakes recorded at the site was the earthquake of February 6, 1987 (magnitude 6.7, epicentral distance of 168 km).

The isolation system described above was replaced by 6 high damping rubber bearings manufactured by Bridgestone Corporation. No other damping devices were needed nor were they installed. The damping ratio at low strain levels was 19% and decreased to about 15% for higher strain. The buildings were monitored for earthquakes to study the performance of these high damping bearings. The review of the results for this system is given in Ref. [2].

The next system to be installed was the high damping isolators manufactured by Furon Corporation. These isolators were made from the same elastomer compound used in the Foothill Community Law and Justice Center (FCLJC) building in California. These Furon bearings were installed in the building in April 1989. Since then significant enhancements to the instrumentation was done as part of the joint ANL/Shimizu program.

The isolators were designed by ANL and EERC using the design parameters of the isolators for the GE-PRISM liquid metal nuclear reactor plant module. (Note: The GE-PRISM isolators were tested in an earlier program at EERC.) Static tests and forced vibration tests were performed on the isolated building with the U.S. system in place and the results are reported in Ref. [3]. These bearings remained in place until October 1990. The response of the building to the few earthquakes that occurred during the period of installation is reported in Refs. [3,4].

Detailed analysis of the response of the system to some of these earthquakes is being completed by Argonne and will be published in late 1991.

The system using the present low modulus high damping rubber was put in place in October 1990, but has experienced to date few earthquakes of any significance. This system was designed to make the natural frequency of the building equal to 0.4 Hz; the natural frequency was 0.75 Hz with the high modulus high damping isolation system. As a result, the measured response of this system, even for small earthquakes, is quite good and predictable.

4. TEST PROGRAM

(i) Horizontal Tests

The bearing was subjected to two sequences of horizontal displacement cycles. Each sequence included three cycles of displacement at each of the following strain levels:

Sequence 1: $\pm 5\%$, $\pm 10\%$, $\pm 25\%$, $\pm 50\%$, $\pm 75\%$, $\pm 100\%$

Sequence 2: $\pm 125\%$, $\pm 150\%$, $\pm 175\%$, $\pm 200\%$

(ii) Vertical Tests

The bearing was subjected to five cycles of vertical loading centered around 500, 625, 1000 and 1500 psi vertical pressure. The variation in the vertical load in each of these tests corresponded to about ± 100 psi. This particular test series is close to the limits of the test system capability at EERC.

The horizontal tests in Sequence 1 were carried out at frequencies of 0.1 Hz. and 0.5 Hz. The horizontal tests in sequence 2 and the vertical tests were carried out only at a frequency of 0.1 Hz. In addition to the cyclic vertical tests, the bearing was also subjected to a monotonic compression test.

(iii) Failure Test

After the dynamic test sequence was completed, the bearing was loaded monotonically in shear at a rate of 2.5 in/min. It was not possible to fail the isolator before reaching the maximum horizontal displacement limit of the actuator. The complete test program is given in Table 1.

5. TEST RESULTS

5.1 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 is computed from the secant measured from peak to peak in each hysteresis loop. The equivalent viscous damping is computed using the formula given in Appendix A. At each level of maximum strain there is a certain amount of softening between the first cycle and subsequent cycles. The effective stiffness and damping for the first cycle of the tests at frequency 0.1 Hz. are given in Table 2. The results of the tests at frequency 0.5 Hz. are given in Table 3.

The loads and displacements were very small in the tests at $\pm 5\%$ level; both quantities were close to the resolution limit of the transducers, and the accuracy of the results is less than that at the higher strains. The force displacement plots for Sequences I - II with zero vertical pressure at frequency of 0.1 Hz. are shown in Figs. 5.1.1 - 5.1.2, with 500 psi vertical pressure in Figs. 5.1.3 - 5.1.4, with 625 psi vertical pressure in Figs. 5.1.5 - 5.1.6, with 1000 psi vertical pressure in Figs. 5.1.7 - 5.1.8, and with 1500 psi vertical pressure in Figs. 5.1.9 - 5.1.10. The force displacement plots for Sequences I at frequency of 0.5 Hz. with zero, 500 psi, 625 psi, 1000 psi and 1500 psi vertical pressure are shown respectively in Figs. 5.1.11 - 5.1.15. The

change in the appearance of the hysteresis loops as the pressures increase is clear from these plots. The loops change from being quite elongated to being elliptical.

The damping factor quoted is based on modeling the bearing as an elastic and linear viscous element. This model predicts that the energy dissipation is quadratic in the displacement and that the effective stiffness is independent of the displacements. In the actual case the damping and stiffness depend on the displacement and the energy dissipation is not quite quadratic.

5.2 Vertical Load Tests

The bearing was subjected to vertical loading under zero horizontal displacement. Two types of tests were carried out. The first was monotonic loading to a level of vertical load beyond the design vertical load. It is a characteristic of laminated bearings that the mechanical response to this type of loading is quite non-linear. The force displacement curve for this test is shown in Fig. 5.2.1.

For the purposes of predicting the vertical dynamic response of the isolated structure, the monotonic loading is not the most important test, but rather the cyclic response for small excursions around the loaded value. Accordingly, several tests were carried out at various pressure levels and excursion levels of ± 100 psi. The results for five cycles around 500 psi, 625 psi, 1000 psi, and 1500 psi are shown in Figs. 5.2.2 through 5.2.9. The stiffnesses that were determined were measured from this are given in Table 4, and the hysteresis loops are shown in Fig. 5.2.10 where they are superimposed on the monotonic loading curve.

The results are quite surprising in that the stiffness varies with the pressure level around which the cycles are centered and they are much higher than the tangent stiffness of the monotonic loading curve. Since most data given in the literature for compression stiffness refers

to secant stiffness or tangent stiffness of the monotonic curve, it is clear that these results will have significance for the vertical response of isolation systems.

5.3 Influence of Vertical Pressure on the Dynamic Properties

Isolation bearings are generally used at pressure levels ranging from 700 to 1000 psi although for some very conservative designs, pressures as low as 250 psi have been proposed. Bearings with low shape factors and the range of height to width ratios that result from the selection of a low pressure tend to be somewhat sensitive to vertical load due to the fact that the vertical load is a significant fraction of the bearing buckling load.

The dynamic tests were carried out at five levels of vertical pressure, 0, 500, 625, 1000 and 1500 psi. The stiffnesses are given in Table 2 and shown in Fig. 5.3.1. For this isolator the pressure has a very definite effect on both the stiffness and the damping. It is obvious from the hysteresis loops that their area for fixed strain increases with increasing pressure, and the effective stiffness decreases, both of these factors leading to much larger damping factors.

The damping factors for each pressure level and each peak strain level are given in Table 2 and shown in Fig. 5.3.2. The damping values over the range of strains up to 100% varies with strain and averages 7.5% for 0 psi, 12.5% for 500 psi, 15% for 625 psi, 23% for 1000 psi and 31% for 1500 psi. The effect of the pressure is primarily an interaction with the buckling behavior of the bearing. This phenomenon has been described in Ref. [5]. Using the approach outlined in Ref. [5], the buckling pressure for this isolator is around 3500 psi, so that the modification of the stiffness and damping at 1500 psi should be expected.

5.4 Failure Test Results

The bearing was loaded monotonically at a rate of 2.5 in/min. The bearing could not be failed, in three trials, before the actuator reached its maximum displacement. The appearance of the bearing at moderate and at large horizontal deformations is shown in Figs. 5.4.1 - 5.4.3. The force displacement curve for the design pressure is shown in Fig. 5.4.4, and the deformation of the bearings in terms of stress versus strain is shown in Fig. 5.4.5. The maximum shear strain obtained was 415% with a maximum shear stress of 355 psi. This is equivalent to a maximum deflection of 19.6 in. with a maximum shear load of 52.9 Kips. It is interesting to note that the maximum displacement exceeds the diameter of the bearing. During the test, the bearing suffered vertical deformations due to the vertical load and the horizontal offset. Figures 5.4.6 - 5.4.11 show the vertical deformation time histories for the three trials. Note that the maximum vertical deformation was slightly over one inch.

The test curve shows that the stress-strain relation of the elastomer is tri-linear. An initial stiffness of around 3.50 kips/in applies for a strain of up to around 15%, the bearing has a stiffness of 1.00 kips/in up to a strain of around 200% and the third segment of the force displacement curve has a stiffness of 9.56 kips/in from 250% to maximum strain. Although it proved impossible to fail the isolator, at this large displacement it is quite likely that it was subjected to some internal damage. However, the test demonstrates that a large reserve of strength is available in properly designed and manufactured elastomer isolators.

6. CONCLUSION

The most important result of the failure test of these isolators is the fact that the bearing was able to sustain horizontal (shear) displacements, while under design vertical load, that exceeded the diameter of the bearing. The maximum shear strain developed at the limit of the

actuator displacement (412%) is, of course, very large and would be at least a factor of two beyond the most ambitious design level (and a factor of at least four beyond current isolator design practice). The significance, however, is the magnitude of shear stress at failure. If we accept that the design level is 200% shear strain, (twice what is usually the design level) then the corresponding shear stress is 50 psi. The vertical pressure on the bearing is 625 psi, so that a shear stress of 50 psi corresponds to a base shear coefficient of 0.08. The maximum shear stress achieved, namely 355 psi, corresponds to a base shear coefficient of 0.568. The implication of this result is that if the superstructure is designed to standard procedures for the design level base shear, there can be no circumstances under which the bearings will fail before the structure.

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TABLE 1
SENDAI - RUBBER BEARING TESTS

SENDAI BEARING				
FILENAME	SIGNAL	TIME	RATE	REMARKS
Compression Diagnostic Tests				
910325.01	zero	5	120	p=0 psi +-100 psi
910325.02	RKW5 dt=.05 0.1Hz	28	60	p=500 psi +-100psi, SPV=42
910325.03	RKW5 dt=.05 0.1Hz	28	60	p=1000 psi +-100psi, SPV=42
910325.04	RKW5 dt=.05 0.1Hz	28	60	p=1500 psi +-100psi, SPV=42
Shear Diagnostic Tests				
910325.05	sin5_100 dt=.02 0.1Hz	45	60	P=0 Kips, SPH=422
910325.06	sin5_100 dt=.02 0.1Hz	45	60	P=74.56 Kips, SPH=422 (p=500psi)
910325.07	sin5_100 dt=.02 0.1Hz	45	60	P=92.6 Kips, SPH=422 (p=629psi)
910325.08	sin5_100 dt=.02 "	45	60	P=149.13 Kips, SPH=422 (p=1000psi)
910325.09	sin5_100 dt=.02 "	45	60	P=223.7 Kips, SPH=422 (p=1500psi)
910325.10	sin5_100 dt=.02 0.5Hz	45	60	P=0 Kips, SPH=422
910325.11	sin5_100 dt=.02 0.5Hz	45	60	P=74.56 Kips, SPH=422 (p=500psi)
910325.12	sin5_100 dt=.02 0.5Hz	45	60	P=92.6 Kips, SPH=422 (p=629psi)
910325.13	sin5_100 dt=.02 "	45	60	P=149.13 Kips, SPH=422 (p=1000psi)
910325.14	sin5_100 dt=.02 "	45	60	P=223.7 Kips, SPH=422 (p=1500psi)
Compression Tests including the design load				
910325.15	RKW5 dt=.05 0.1Hz	28	60	p=500 psi +-100 psi, SPV=42
910325.16	RKW5dt=.05 0.1Hz	28	60	p=500 psi +-100 psi, SPV=42
910325.17	RKW5dt=.05 0.1Hz	28	60	p=1000 psi +-100 psi, SPV=42
910325.18	RKW5dt=.05 0.1Hz	28	60	p=1500 psi +-100 psi, SPV=42
Shear Test				
910325.19	sin5_100	45	60	P=74.56 Kips,SPH=422(p=500psi)
Compression Test				
910325.20	0-200 Kips			P=0 to 200 Kips
910402.02	zero	5	120	
910402.03	RKW1 dt=.1	25	60	P=0 to max. at 50% shear strain
910404.01	zero	5	120	P=0 Kips
910404.02	RKW1 dt=.1	25	60	P=0 to max. SPV=650 Pump at 2800 psi
910404.03	RKW1 dt=.1	25	60	P=0 to max. Pump at 3000 psi
Compression Diagnostic Test				
910404.04	RKW5	28	60	p=500 psi +- 100 psi SPV=42
910404.05	RKW5	28	60	p=621 psi +-100 psi SPV=42
910404.06	RKW5	28	60	p=1000 psi +-100 psi SPV=42
910404.07	RKW5	28	60	p=1500 psi +-100 psi SPV=42

TABLE 1 (CONTINUATION)

SENDAI - RUBBER BEARING TESTS

SENDAI BEARING				
FILENAME	SIGNAL	TIME	RATE	REMARKS
Shear Diagnostic Test				
910404.08	SIN5_100	225	5	p=500 psi SPH=197,=3.75"
910404.09	SIN5_100	200	5	p=500 psi SPH=248.6,=4.72"
910404.10	SIN125_2	130	5	p=500 psi SPH=248.6,=4.72"
910404.11	SIN125_2	130	5	p=500 psi SPH=497.2,=9.44"
910404.12	SIN125_2	125	5	p=621 psi SPH=497.2,=9.44"
910404.13	SIN125_2	125	5	p=621 psi SPH=497.2,=9.44"
Symmetrical bumps appeared at top and bottom of the bearing				
910404.14	SIN125_2	125	5	p=0 psi SPH=497.2,=9.44"
910404.15	SIN125_2	125	5	p=1500 psi SPH=497.2,=9.44"
Shear Failure Test				
910404.16	zero			
910404.17	BNEG	125	5	p=621psiSPH=1000 Bearing did not fail
4" plates spacers added				
910404.18	BNEG	125	5	p=621 psi SPH=1000 It reached 20" and did not fail
910404.19	BNEG	125	5	p=621 psi SPH=1000 test was repeated and still the bearing did not fail
A very interesting phenomenon happened with the shear failure test, when the bearing deflected about 10 inches from the center line, the bearing rolled over the base plate and started to gain strength again.				

SENDAI - BEARING TEST

TABLE 2

HORIZONTAL SHEAR TEST FREQUENCY : 0.1 Hz													
PRESSURE ¹ PSI	FILENAME (5-100%)	DYNAMIC PROPERTIES	SHEAR STRAIN										FILENAME (125-200%)
			5%	10%	25%	50%	75%	100%	125%	150%	175%	200%	
0	910325.05	K_{eff} (Kips in) β (%) G (psi)	4.08 12.68 129	3.36 10.84 107	2.79 8.50 88	2.36 7.40 75	2.14 7.02 68	1.98 6.68 63	1.49 6.34 47	1.44 6.60 46	1.41 6.87 45	1.43 6.61 45	910404.14
500	910325.06	K_{eff} (Kips in) β (%) G (psi)	4.18 18.33 133	3.27 17.82 103	2.47 14.83 78	2.07 12.51 66	1.86 11.29 59	1.74 10.67 55	1.4 10.69 44	1.37 10.73 43	1.32 10.59 42	1.27 10.75 40	910404.11
500	910325.09	K_{eff} (Kips in) β (%) G (psi)	3.75 15.93 119	2.90 19.64 92	2.10 15.23 67	1.76 12.15 56	1.56 11.66 49	1.49 10.79 47					
621	910325.07	K_{eff} (Kips in) β (%) G (psi)	4.68 16.49 148	3.44 19.41 109	2.46 17.54 78	1.94 15.4 61	1.73 13.38 55	1.66 12.35 52	1.11 14.81 35	1.06 14.56 33	1.01 15.33 32	0.98 16.66 31	910404.12
1000	910325.08	K_{eff} (Kips in) β (%) G (psi)	5.49 18.12 174	3.90 22.27 124	2.46 24.67 78	1.79 23.29 57	1.53 21.72 48	1.42 20.15 45	0.71 33.20 23	0.65 33.35 20	0.57 35.06 18	0.50 41.21 16	910404.13
1500	910325.09	K_{eff} (Kips in) β (%) G (psi)	6.76 12.94 214	4.77 21.20 151	2.72 27.62 86	1.76 31.23 56	1.41 31.60 45	1.24 31.16 39	0.74 40.25 23	0.76 35.57 24	0.85 31.23 27	0.99 26.71 31	910404.15

1. These pressures are the target pressures, actual values are indicated in the plots.

TABLE 3

HORIZONTAL SHEAR TEST FREQUENCY : 0.5 Hz								
PRESSURE ¹ PSI	FILENAME (5-100%)	DYNAMIC PROPERTIES	SHEAR STRAIN					
			5%	10%	25%	50%	75%	100%
0	910325.10	$K_{(eff)}$ (Kips in)	3.64	3.14	2.61	2.21	1.97	1.83
		β (%)	13.69	11.72	9.66	8.82	8.27	8.02
		G (psi)	115	100	83	70	62	58
500	910325.11	$K_{(eff)}$ (Kips in)	4.39	3.37	2.45	1.93	1.66	1.53
		β (%)	18.35	17.84	16.29	14.57	13.75	13.25
		G (psi)	139	107	78	61	53	48
621	910325.12	$K_{(eff)}$ (Kips in)	4.69	3.47	2.41	1.85	1.58	1.46
		β (%)	18.31	20.71	19.27	17.39	16.17	15.23
		G (psi)	149	110	76	59	50	46
1000	910325.13	$K_{(eff)}$ (Kips in)	5.39	3.82	2.29	1.56	1.28	1.16
		β (%)	17.81	23.03	27.77	27.49	26.68	25.54
		G (psi)	171	121	72	49	41	37
1500	910325.14	$K_{(eff)}$ (Kips in)	6.35	4.44	2.49	1.52	1.1	0.92
		β (%)	15.51	21.65	31.04	36.96	42.39	42.52
		G (psi)	201	141	79	48	35	29

TABLE 4

SENDAI - BEARING TEST COMPRESSION TEST					
FILENAME	PRESSURE ¹ PSI	ΔP PSI	STIFFNESS (Kips in)		f_v HERTZ
			1ST. CYCLE	5TH. CYCLE	
910325.15	500	± 100	1114.11	1114.12	12.10
910404.04	500	± 100	916.88	914.16	10.99
910325.16	621	± 100	1151.32	1149.26	10.84
910404.05	621	± 100	1059.80	1054.51	10.56
910325.17	1000	± 100	1697.81	1657.7	10.47
910404.06	1000	± 100	1621.35	1588.43	10.21
910325.18	1500	± 100	3878.29	3820.09	13.03
910404.07	1500	± 100	2545.18	2635.64	10.56

1. These pressures are the target pressures, actual values are indicated in the plots.

ARGONNE NATIONAL LABORATORY			
Title: HDNR-S BEARING PAD		Drwg. No.: NSF-RAS-001	
Project: ANL/NSF/SHIMIZU		Date: 07-17-90	Rev.: 01
Project Mgrs.: R.W. SEIDENSTICKER		Engineer: R.F. KULAK	
Y.W. CHANG		Drawn By: C. FIALA	

Fig. 2.1 Design Details for Test Bearings

Fig. 3.1.1 Single Bearing Test Machine

Fig. 3.2.1 General View of Test Facility

Fig. 3.2.2 Plan and Elevation Views of Test Facility

SEN...-HEARING TEST
SHEAR DIAGNOSTIC TEST 5% - 100% STRAIN

Strain	Stiffness	Shear Modulus	Damping
5%	4.08 Kips/in	129 psi	12.68 %
10%	3.36	107	10.84
25%	2.79	88	8.5
50%	2.36	75	7.4
75%	2.14	68	7.02
100%	1.98	63	6.68

FILENAME: 910325.05
VERT PRESSURE: 0 PSI
FREQUENCY: 0.1 HERTZ

Fig. 5.1.1 Force Displacement Plots for Sequence I - Vertical Zero Pressure at Frequency 0.1 Hz

SENDAI - BEARING TEST
SHEAR DIAGNOSTIC TEST 125% - 200% STRAIN

Strain	Stiffness	Shear Modulus	Damping
125%	1.49 Kips/in	47 psi	6.34 %
150%	1.44	46	6.6
175%	1.41	45	6.87
200%	1.43	45	6.61

FILENAME: 910404.14

VERT PRESSURE: 0 PSI

Horizontal Load vs Deflection

Fig. 5.1.2 Force Displacement Plots for Sequence II - Vertical Zero Pressure at Frequency 0.1 Hz

SENDAI-BEARING TEST
SHEAR DIAGNOSTIC TEST 5% - 100% STRAIN

Strain	Stiffness	Shear Modulus	Damping
5%	4.18 Kips/in	133 psi	18.33 %
10%	3.27	103	17.02
25%	2.47	78	14.83
50%	2.07	66	12.51
75%	1.86	59	11.29
100%	1.74	55	10.67

FILENAME: 910325.06
VERT PRESSURE: 500 PSI
FREQUENCY: 0.1 HERTZ

Fig. 5.1.3 Force Displacement Plots for Sequence I - 500 psi Vertical Pressure at Frequency 0.1 Hz

SENDAI -BEARING TEST
 SHEAR DIAGNOSTIC TEST 5% - 100% STRAIN

Strain	Stiffness	Shear Modulus	Damping
5%	3.75 Kips/in	119 psi	15.93 %
10%	2.9	92	19.64
25%	2.1	67	15.23
50%	1.76	56	12.15
75%	1.56	49	11.66
100%	1.49	47	10.79

FILENAME: 910404.09
 VERT PRESSURE: 500 PSI
 FREQUENCY: 0.1 HERTZ

Fig. 5.1.3B Force Displacement Plots for Sequence I - 500 psi Vertical Pressure at Frequency 0.1 Hz

SEND... - BEARING TEST
SHEAR DIAGNOSTIC TEST 125% - 200% STRAIN

FILENAME: 910404.11

VERT PRESSURE: 500 PSI

Strain	Stiffness	Shear Modulus	Damping
125%	1.4 Kips/in	44 psi	10.69 %
150%	1.37	43	10.73
175%	1.32	42	10.59
200%	1.27	40	10.75

Fig. 5.1.4 Force Displacement Plots for Sequence II - 500 psi Vertical Pressure at Frequency 0.1 Hz

SENDAL-BEARING TEST
 SHEAR DIAGNOSTIC TEST 5% - 100% STRAIN

Strain	Stiffness	Shear Modulus	Damping
5%	4.68 Kips/in	148 psi	16.49 %
10%	3.44	109	19.41
25%	2.46	78	17.54
50%	1.94	61	15.4
75%	1.73	55	13.30
100%	1.66	52	12.35

FILENAME: 910325.07
 VERT PRESSURE: 629 PSI
 FREQUENCY: 0.1 HERTZ

Fig. 5.1.5 Force Displacement Plots for Sequence I - 629 psi Vertical Pressure at Frequency 0.1 Hz

SENDAL - BEARING TEST
SHEAR DIAGNOSTIC TEST 125% - 200% STRAIN

Strain	Stiffness	Shear Modulus	Damping
125%	1.11 Kips/in	35 psi	14.81 %
150%	1.06	33	14.56
175%	1.01	32	15.33
200%	0.98	31	15.66

FILENAME: 910404.12

VERT PRESSURE: 621 PSI

Horizontal Load vs Deflection

Fig. 5.1.6 Force Displacement Plots for Sequence II - 621 psi Vertical Pressure at Frequency 0.1 Hz

SENDAI-BEARING TEST
 SHEAR DIAGNOSTIC TEST 5% - 100% STRAIN

Strain	Stiffness	Shear Modulus	Damping
5%	5.49 Kips/in	174 psi	18.12 %
10%	3.9	124	22.27
25%	2.46	78	24.67
50%	1.79	57	23.29
75%	1.53	48	21.72
100%	1.42	45	20.15

FILENAME: 910325.08
 VERT PRESSURE: 1000 PSI
 FREQUENCY: 0.1 HERTZ

Fig. 5.1.7 Force Displacement Plots for Sequence I -1000 psi Vertical Pressure at Frequency 0.1 Hz

SENDAL - BEARING TEST
SHEAR DIAGNOSTIC TEST 125% - 200% STRAIN

FILENAME: 910404.13

VERT PRESSURE: 1000 PSI

Strain	Stiffness	Shear Modulus	Damping
125%	0.71 Kips/in	23 psi	33.2 %
150%	0.65	20	33.35
175%	0.57	18	35.06
200%	0.5	16	41.21

Fig. 5.1.8 Force Displacement Plots for Sequence II -1000 psi Vertical Pressure at Frequency 0.1 Hz

SENDAL-BEARING TEST
 SHEAR DIAGNOSTIC TEST 5% - 100% STRAIN

Strain	Stiffness	Shear Modulus	Damping
5%	6.76 Kips/in	214 psi	12.94 %
10%	4.77	151	21.2
25%	2.72	86	27.62
50%	1.76	56	31.23
75%	1.41	45	31.6
100%	1.24	39	31.16

FILENAME: 910325.09
 VERT PRESSURE: 1500 PSI
 FREQUENCY: 0.1 HERTZ

Fig. 5.1.9 Force Displacement Plots for Sequence I - 1500 psi Vertical Pressure at Frequency 0.1 Hz

SENDAL - BEARING TEST
SHEAR DIAGNOSTIC TEST 125% - 200% STRAIN

FILENAME: 910404.15	Strain	Stiffness	Shear Modulus	Damping
VERT PRESSURE: 1500 PSI	125%	0.74 Kips/in	23 psi	40.25 %
	150%	0.76	24	35.57
	175%	0.85	27	31.23
	200%	0.99	31	26.71

Fig. 5.1.10 Force Displacement Plots for Sequence II - 1500 psi Vertical Pressure at Frequency 0.1 Hz

SENDAI-BEARING TEST
SHEAR DIAGNOSTIC TEST 5% - 100% STRAIN

Strain	Stiffness	Shear Modulus	Damping
5%	3.64 Kips/in	115 psi	13.69 %
10%	3.14	100	11.72
25%	2.61	83	9.66
50%	2.21	70	8.82
75%	1.97	62	8.27
100%	1.83	58	8.02

FILENAME: 910325.10
VERT PRESSURE: 0 PSI
FREQUENCY: 0.5 HERTZ

Fig. 5.1.11 Force Displacement Plots for Sequence I - Zero Vertical Pressure at Frequency 0.5 Hz

SENDA-BEARING TEST
 SHEAR DIAGNOSTIC TEST 5% - 100% STRAIN

Strain	Stiffness	Shear Modulus	Damping
5%	4.39 Kips/in	139 psi	18.35 %
10%	3.37	107	17.84
25%	2.45	78	16.29
50%	1.93	61	14.57
75%	1.66	53	13.75
100%	1.53	48	13.25

FILENAME: 910325.11
 VERT PRESSURE: 500 PSI
 FREQUENCY: 0.5 HERTZ

Fig. 5.1.12 Force Displacement Plots for Sequence I - 500 psi Vertical Pressure at Frequency 0.5 Hz

SENDAI-BEARING TEST
SHEAR DIAGNOSTIC TEST 5% - 100% STRAIN

Strain	Stiffness	Shear Modulus	Damping
5%	4.69 Kips/in	149 psi	18.31 %
10%	3.47	110	20.71
25%	2.41	76	19.27
50%	1.85	59	17.39
75%	1.58	50	16.17
100%	1.46	46	15.23

FILENAME: 910325.12
VERT PRESSURE: 629 PSI
FREQUENCY: 0.5 HERTZ

Fig. 5.1.13 Force Displacement Plots for Sequence I - 629 psi Vertical Pressure at Frequency 0.5 Hz

SEN. BEARING TEST
SHEAR DIAGNOSTIC TEST 5% - 100% STRAIN

Strain	Stiffness	Shear Modulus	Damping
5%	5.39 Kips/in	171 psi	17.81 %
10%	3.82	121	23.03
25%	2.29	72	27.77
50%	1.56	49	27.49
75%	1.28	41	26.68
100%	1.16	37	25.54

FILENAME: 910325.13
VERT PRESSURE: 1000 PSI
FREQUENCY: 0.5 HERTZ

Fig. 5.1.14 Force Displacement Plots for Sequence I - 1000 psi Vertical Pressure at Frequency 0.5 Hz

SENDAL-BEARING TEST
 SHEAR DIAGNOSTIC TEST 5% - 100% STRAIN

Strain	Stiffness	Shear Modulus	Damping
5%	6.35 Kip/in	201 psi	15.51 %
10%	4.44	141	21.65
25%	2.49	79	31.04
50%	1.52	48	36.96
75%	1.1	35	42.39
100%	0.92	29	42.52

FILENAME: 910325.14
 VERT PRESSURE: 1500 PSI
 FREQUENCY: 0.5 HERTZ

Fig. 5.1.15 Force Displacement Plots for Sequence I - 1500 psi Vertical Pressure at Frequency 0.5 Hz

Filename: 910404.03

Secant Vertical Stiffness : 647.57 Kips/in

Max.Vert.Load: 232.3652 Kips

Vertical Load vs Vertical Deflection

Fig. 5.2.1 Force Displacement Curve for Monotonic Loading

SENDAY -BEARING TEST
COMPRESSION TEST

Filename: 910404.04 Vertical Stiffness (first cycle): 916.88 kips/in
Average Pressure: 498 psi Vertical Stiffness (fifth cycle): 914.16 kips/in

Fig. 5.2.2 Force Displacement Plots under Vertical Cycles Centered at 500 psi for Set #1.

BEN. -BEARING TEST
COMPRESSION TEST

Filename: 910404.05 Vertical Stiffness (first cycle): 1059.8 kips/in
Average Pressure: 624 psi Vertical Stiffness (fifth cycle): 1054.51 kips/in

Fig. 5.2.3 Force Displacement Plots under Vertical Cycles Centered at 625 psi for Set #1.

SENDAI -BEARING TEST
COMPRESSION TEST

Filename: 910404.06 Vertical Stiffness (first cycle): 1621.35 kips/in
Average Pressure: 1021 psi Vertical Stiffness (fifth cycle): 1588.43 kips/in

Fig. 5.2.4 Force Displacement Plots under Vertical Cycles Centered at 1000 psi for Set #1.

SENDAI -BEARING TEST
COMPRESSION TEST

Filename: 910404.07 Vertical Stiffness (first cycle): 2545.18 kips/in
Average Pressure: 1499 psi Vertical Stiffness (fifth cycle): 2635.64 kips/in

Fig. 5.2.5 Force Displacement Plots under Vertical Cycles Centered at 1500 psi for Set #1.

SENDAI - BEARING TESTS
VERTICAL TEST

FILENAME: 910325.15
MEAN VERT. PRESSURE: 490 PSI

Cycle
first
fifth

Vertical Stiffness
1114.44 Kips/in
1114.12 Kips/in

Fig. 5.2.6 Force Displacement Plots under Vertical Cycles Centered at 500 psi for Set #2.

SENDA. BEARING TESTS
VERTICAL TEST

FILENAME: 910325.16

MEAN VERT.PRESSURE: 643 PSI

Vertical Stiffness
1151.32 Kips/in
1149.26 Kips/in

Cycle
first
fifth

Fig. 5.2.7 Force Displacement Plots under Vertical Cycles Centered at 625 psi for Set #2.

Fig. 5.2.8 Force Displacement Plots under Vertical Cycles Centered at 1000 psi for Set #2.

SENDA - BEARING TESTS

VERTICAL TEST

FILENAME: 910325.18
 MEAN VERT. PRESSURE: 1500 PSI
 Vertical load/displacement
 Vertical stiffness
 3878.29 Kips/in
 3020.09 Kips/in

Cycle

first
fifth

Fig. 5.2.9 Force Displacement Plots under Vertical Cycles Centered at 1500 psi for Set #2.

SENDAY -BEARING TEST
COMPRESSION TEST

Filename: 910404.03

Max.Vert.Load: 232.36 Kips

Fig. 5.2.10 Superimposition of Cyclic Vertical Tests on Monotonic Vertical Compression Test.

SENDA BEARING TEST

EFFECTIVE STIFFNESS - SHEAR STRAIN PLOTS AT FREQ.=0.1 Hz

pressure P=0 psi: □
 pressure P=500 psi: ○
 pressure P=629 psi: ×
 pressure P=1000 psi: △
 pressure P=1500 psi: ◇

Effective Stiffness vs Strain

Fig. 5.3.1 Influence of Vertical Load and Strain on Bearing Stiffness

SENDAI-BEARING TEST
DAMPING - SHEAR STRAIN PLOTS AT FREQ.=0.1 Hz

pressure P=0 psi: □
pressure P=500 psi: ○
pressure P=625 psi: △
pressure P=1000 psi: +
pressure P=1500 psi: ◇

Fig. 5.3.2 Influence of Vertical Load and Strain on Bearing Damping

Fig. 5.4.1 Configuration of Bearing before Failure Tests

Fig. 5.4.2 Bearing Appearance at 200% Strain

Fig. 5.4.3 Bearing Appearance at 300% Strain

SENDAI - BEARING TEST
SHEAR FAILURE TEST - BEARING DID NOT FAIL

Filename: 910404.18

Max Strain: 415 %

Horizontal Load vs Deflection

Fig. 5.4.4 Force Displacement Curves for Bearings in Failure Tests

SENDAL - BEARING TEST
SHEAR FAILURE TEST-BEARING DID NOT FAIL

Filename: 910404.18

Max Strain: 415 %

Fig. 5.4.5 Stress-Strain Curves for Bearings in Failure Tests

SENDAI-ARGONNE BEARING TEST
VERTICAL DEFLECTION HISTORY OF 4 DCDT[®] TRIAL No. 1

FILENAME: 910404.17
AXIAL PRESSURE: 660 PSI

Fig. 5.4.6 Vertical Displacement History of 4 DCDTs for Failure Test Trial No. 1

SENDAI - ARGONNE BEARING TEST
AVERAGE VERTICAL DEFLECTION HISTORY TRIAL No. 1

FILENAME: 910404.17

AXIAL PRESSURE: 660 psi

Fig. 5.4.7 Average Vertical Displacement History of Failure Test Trial No. 1

SENDAI-ARGONNE BEARING TEST
VERTICAL DEFLECTION HISTORY OF 4 DCDTs TRIAL No. 2

FILENAME: 910404.18

AXIAL PRESSURE: 507 PSI

Fig. 5.4.8 Vertical Displacement History of 4 DCDTs of Failure Test Trial No. 2

SENDAI - ARGONNE BEARING TEST
AVERAGE VERTICAL DEFLECTION HISTORY TRIAL No. 2

FILENAME: 910404.18

AXIAL PRESSURE: 507 psi

Fig. 5.4.9 Average Vertical Displacement History of Failure Test Trial No. 2

SENDAI-ARGONNE BEARING TEST
VERTICAL DEFLECTION HISTORY OF 4 DCDTs TRIAL No. 3

FILENAME: 910404.19

AXIAL PRESSURE: 621 PSI

Fig. 5.4.10 Vertical Displacement History of 4 DCDTs of Failure Test Trial No. 3

SENDAL - ARGONNE BEARING TEST
AVERAGE VERTICAL DEFLECTION HISTORY TRIAL No. 3

FILENAME: 910404.19

AXIAL PRESSURE: 621 psi

Fig. 5.4.11 Average Vertical Displacement History of Failure Test Trial No. 3

Appendix A

Estimation of Equivalent Viscous Damping

When the force displacement characteristic of an element or component is modeled by the standard structural model, the force is related to the displacement through

$$F = K\Delta + C\dot{\Delta}$$

If such an element is subjected to a sinusoidal displacement

$$\Delta = \Delta_0 \sin \omega t$$

the resulting force is

$$F = K\Delta_0 \sin \omega t + C\omega\Delta_0 \cos \omega t$$

The maximum force is given by the value of ωt for which $\dot{F} = 0$, namely,

$$\tan \omega t = \frac{K}{C} \omega$$

and the force

$$F_{\max} = K\Delta_0 \frac{K/C \omega + C \omega/K}{(1 + (K/C \omega)^2)^{1/2}}$$

or

$$F_{\max} = K\Delta_0 (1 + (\frac{C\omega}{K})^2)^{1/2}$$

The effective stiffness K_{eff} defined by

$$K_{eff} = \frac{F_{\max}}{\Delta_{\max}} = K (1 + (\frac{C\omega}{K})^2)^{1/2}$$

The total energy dissipated by the element in a complete cycle is given by

$$W_D = \int_0^{2\pi/\omega} F(t) \dot{\Delta}(t) dt$$

Thus, in a sinusoidal test we measure the area of the hysteresis loop and K_{eff} and from these we can determine K and C .

If the K and C were associated with a vibrating mass m in a single degree of freedom system

$$K/m = \omega_0^2 \quad \text{and} \quad C/m = 2\omega_0\beta$$

where ω_0 is the fundamental frequency and β is the fraction of critical damping. It follows that

$$C = \frac{2\beta}{\omega_0} K$$

and thus

$$W_D = 2\pi K \frac{\omega}{\omega_0} \Delta_o^2 \beta$$

leading to an estimate of β as

$$\beta = \frac{\omega_0}{\omega} \frac{W_D}{2\pi K \Delta_o^2}$$

This formula shows that the viscous model predicts that the energy dissipation should be linear in the frequency and quadratic in the displacement. In this paper, the equivalent viscous damping was calculated by using the following equation:

$$\beta = \frac{W_D}{2\pi K \Delta_o^2}$$

Appendix B

List of ANL and Shimizu Joint Study Reports

compiled by

Ralph W. Seidensticker and Yao-Wen Chang

Argonne National Laboratory

<u>Report No.</u>	<u>Title/Authors/Date</u>
ANL/Shimizu Shmz-001	Outline of Study and Site Experiment of Base-Isolated Building in Sendai T. Kuroda, M. Kobatake, Y. Nitta, H. Watanabe, M. Saruta, and K. Okada, July 1989
ANL/Shimizu Shmz-002	Outline of Study and Test on Characteristics of Individual Bearing T. Kuroda, M. Kobatake, Y. Nitta, H. Watanabe, M. Saruta, and K. Okada, November 1989
ANL/Shimizu Shmz-003	Recent Earthquake Observation Records at Sendai T. Kuroda, M. Kobatake, Y. Nitta, H. Watanabe, M. Saruta, and K. Okada, August 1990
ANL/Shimizu Shmz-004	Final Report on Joint US/Japanese Program on Seismic Isolation Study T. Kuroda, M. Kobatake, Y. Nitta, H. Watanabe, M. Saruta, and K. Okada, January 1991
ANL/Shimizu Shmz-005	Phase II Outline of Study and Site Experiment (UK Bearing) T. Kuroda, M. Kobatake, Y. Nitta, H. Watanabe, M. Saruta, and S. Hayashi, January 1991
ANL/Shimizu Shmz-006	Phase II Outline of Study and Test on Characteristics of Individual Bearing T. Kuroda, M. Kobatake, Y. Nitta, K. Ohta, H. Watanabe, M. Saruta, and S. Hayashi, June 1991
ANL/Shimizu ANL-001	Comparison of SISEC Code Simulation with Earthquake Data of Ordinary and Base-Isolated Buildings C. Y. Wang and J. Gvildys, August 1990
ANL/Shimizu ANL-002	EERC Lab Report 89-101, Performance Evaluation of ANL Sendai Bearing J. M. Kelly, I. D. Aiken, and D. Clyde, September 1989
ANL/Shimizu ANL-003	Dynamic and Failure Characteristics of Bridgestone Bearings J. M. Kelly, March 1991

ANL/Shimizu Mechanical Characteristics of Low Modulus High Damping Rubber Isolators for
ANL-004 a Base Isolated Demonstration Building
J. M. Kelly, June 1991

GUIDELINES FOR DESIGN CRITERIA FOR BASE ISOLATION RETROFIT OF EXISTING BUILDINGS

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ABSTRACT

This paper proposes tentative design criteria guidelines for seismic (base) isolation retrofit of existing buildings. The guidelines utilize to a great extent the design criteria and requirements developed in the late 1980's by the Structural Engineers Association of California (SEAOC) for the design of newly constructed isolated structures. These design provisions were published in the 1990 SEAOC Blue Book and the 1991 Uniform Building Code. Since the SEAOC/UBC provisions for new construction form the basis for the tentative retrofit guidelines, the first section of this paper is devoted to a review of these provisions. The balance of this paper summarizes design criteria of on-going or completed seismic-isolation retrofit projects, and identifies tentative guidelines for developing seismic-isolation design criteria for future retrofit projects.

The tentative seismic-isolation retrofit guidelines proposed in this paper are based on input provided by various members of SEAOC base isolation subcommittees, but do not represent a consensus opinion of these subcommittees. It is anticipated that these guidelines will provide a basis for further discussion and modification and eventually lead to a consensus set of guidelines that can be adopted by SEAOC.

INTRODUCTION

Seismic isolation, commonly referred to as base isolation, is a design concept based on the premise that a structure can be substantially decoupled from potentially damaging earthquake ground motions. By decoupling the structure from ground shaking, isolation reduces the level of response in the structure from the level which would otherwise occur in a conventional, fixed-base building or, conversely, offers the advantage of designing with a reduced level of earthquake load to achieve the same degree of seismic protection. It is assumed that decoupling will be accomplished using an isolation scheme which makes the fundamental period of the isolated structure several times greater than the period of the structure above the isolation system.

The potential advantages of seismic isolation and the recent advancements in isolation system products have been responsible for the design and construction of a number of isolated buildings and bridges in California and other states. This activity has, in turn, identified a need to supplement existing codes with design requirements developed

specifically for such structures. This need is shared by the public which requires assurance that isolated buildings are "safe" and by the engineering profession which requires a minimum standard upon which design and construction can be based. Accordingly, the Seismology Committee of the Structural Engineers Association of California (SEAOC) has developed design provisions for isolated structures that supplement existing design provisions for fixed-base structures.

The balance of this paper is divided into a background section on the history of various SEAOC isolation committees, a section that describes the design philosophy and criteria of the SEAOC/UBC provisions, a section that describes design methods and requirements, a section that summarizes the design criteria of on-going or completed retrofit projects and a section that identifies tentative guidelines for developing seismic-isolation design criteria for future retrofit projects.

BACKGROUND

The seismic isolation concept and criteria that would be appropriate for design and construction of isolated structures have been considered by various SEAOC committees, active since the early 1980's. Initially, a committee was formed by members of the Northern Section of SEAOC to develop guidelines for the use of elastomeric bearings as the base isolation system for hospitals. With input from members of this committee, the Building Safety Board (BSB) of the California Office of Statewide Health Planning and Development (OSHPD), adopted "guidelines" for regulation of isolated hospitals. These guidelines were based on the special hospital design criteria specified in Title 24 of the California Administrative Code [Ref. 1]. It is the intent of Title 24 that hospitals (and other vital facilities) be operational after a major earthquake.

At the request of the BSB, SEAOC reviewed the hospital guidelines in 1988 and proposed changes consistent with the prevailing design philosophy of SEAOC for isolated structures. These changes were incorporated by the BSB in a revision of the guidelines [Ref. 2] that represented consensus opinion of SEAOC and the BSB as of 1988. The BSB is currently updating the hospital guidelines to incorporate SEAOC/UBC design criteria developed since 1988.

In addition to providing input to the BSB for regulation of hospitals, committees within the Northern and Southern Sections of the SEAOC have been active in developing design provisions applicable to other types of isolated structures. This effort is consistent with one of SEAOC's primary responsibilities: the development of seismic provisions that establish minimum design and construction standards. Since 1959 the Seismology Committee of the SEAOC has been developing provisions for earthquake-resistant design of structures. These provisions are published periodically as the SEAOC document, *Recommended Lateral Force Requirements and Commentary* (commonly known as the SEAOC Blue Book), and serve as the role model for the seismic requirements of several building codes. For instance, most of the requirements of the most recent (fifth) edition of the SEAOC Blue Book [Ref. 3], have been adopted by the International Conference of Building Officials and published in the 1988 and 1991 editions of the *Uniform Building Code* [Ref. 4], the most widely used code for regulation of buildings constructed in zones of high seismicity.

Efforts directed at creating design provisions for isolated structures began with the Northern Section of the SEAOC in the mid-1980's. In 1986, this section of SEAOC published *Tentative Seismic Isolation Design Requirements* [Ref. 5], the first collection of design provisions for base-isolated structures. These provisions were based on the same seismic criteria being incorporated into draft revisions of the Blue Book, and used similar design concepts such as the prescription of minimum design forces and displacements by simple formulas. Recent isolation projects, such as the Los Angeles County Fire Command and Control Facility [Ref. 6], have used these provisions to help establish project-specific seismic design requirements.

In parallel with the Northern Section, the Southern Section of SEAOC studied base isolation theory and also developed design methods for isolated structures. In the late 1980's, members of the Southern Section published several papers that provide guidelines for the design of buildings with base isolators [Ref. 7]. In these papers, the authors emphasized the need to thoroughly understand and evaluate site seismicity, isolator characteristics and potential inelastic response of the isolated structure.

Recognizing the need for a document that would represent a consensus opinion of all sections of SEAOC, the Seismology Committee formed the Ad Hoc Base Isolation Subcommittee in early 1988 to synthesize a single seismic isolation design document from existing material. Accordingly, the representatives of the Northern, Southern and Central Sections of SEAOC developed design provisions titled: "General Requirements for the Design and Construction of Seismic-Isolated Structures." These provisions were approved by the Seismology Committee and published as Appendix 1L of the 1990 Blue Book [Ref. 3]. These provisions were also adopted (with minor, editorial changes) by the International Conference of Building Officials (ICBO) and published as a non-mandatory appendix to Chapter 23 of the 1991 UBC [Ref. 4]. The following two sections of this paper discuss the philosophy and criteria underlying the SEAOC/UBC provisions and the design methods and requirements specified by the SEAOC/UBC provisions.

DESIGN PHILOSOPHY AND CRITERIA

The underlying philosophy guiding the development of the SEAOC/UBC provisions for isolated structures may be characterized as a combination of SEAOC'S primary performance objective for fixed-base buildings, which is the protection of life safety for a major earthquake, and the additional performance objective of damage protection, an attribute inherent to isolated structures. The design criteria of the SEAOC/UBC provisions are based on this combination of life-safety and damage protection goals. These criteria are summarized in the following statements.

1. SEAOC/UBC provisions specify two levels of earthquake: the Design-Basis Earthquake and the Maximum-Credible Earthquake.

The Design-Basis Earthquake is the same earthquake level of ground shaking as that required by the Blue Book for design of fixed-base structures, a level of ground motion that has a 10 percent probability of being exceeded in a 50-year time period.

The Maximum-Credible Earthquake is an additional, higher level of earthquake

ground motion defined as the maximum level of ground shaking that may be expected at the building site within the known geological framework. This intensity may be taken as the level of earthquake ground motion that has a 10 percent probability of being exceeded in a 250-year time period.

2. The SEAOC/UBC provisions require the isolation system to be capable of sustaining loads corresponding to the Maximum-Credible Earthquake without failure (e.g., the isolation system is to be designed and tested for Maximum-Credible Earthquake displacement). Likewise, the provisions require building separations and utilities that cross the isolation interface to be designed to accommodate Maximum-Credible Earthquake displacement.
3. The SEAOC/UBC provisions require the structure (above the isolation system) to remain "essentially elastic" for the Design-Basis Earthquake (e.g., inelastic response of the lateral load-resisting system is limited to about one-quarter of that permitted by the SEAOC Blue Book for design of a comparable, fixed-base building).

The SEAOC/UBC provisions for fixed-base buildings provide reasonable protection against major structural failure and loss of life, but are not intended "to limit damage, maintain functions, or provide for easy repair" [page 1-C, Ref. 3]. Based on this philosophy, the lateral forces required for allowable-stress design of fixed-base structures are only one-fourth to one-twelfth of the force level that would occur in the building during a major earthquake, if the structure remained fully elastic. Life safety is protected by design provisions that require the structural system to have sufficient ductility and stability to displace significantly beyond the elastic limit without gross failure or collapse. However, damage to structural elements, nonstructural components and/or contents of a fixed-base building can occur during an earthquake and would be likely for a major event.

The SEAOC/UBC provisions for fixed-base buildings are based on earthquake forces corresponding to the Design-Basis Earthquake (reduced for design of elements, as discussed above). Survival for response beyond the Design-Basis Earthquake level is implicitly addressed by special ductility and detailing requirements. In contrast, the SEAOC/UBC provisions for isolated buildings explicitly consider response beyond the Design-Basis Earthquake by requiring the isolation system to be designed for displacement corresponding to the Maximum-Credible Earthquake, an event that represents "worst-case" earthquake demands on the isolation system. The intent of requiring the isolation system to be explicitly designed (and tested) for Maximum-Credible Earthquake displacement is to provide reasonable assurance that the isolation system will be at least as "safe" as a fixed-base structure. Explicit design (and testing) of the isolation system for "worst-case" earthquake displacement is necessary at this time since a sufficient base of experience does not exist that would justify less conservative criteria.

Ideally, lateral displacement of an isolated structure will result predominantly due to the deformations of the isolation system, rather than in distortion of the structure above. Accordingly, the lateral load-resisting system of the structure above the isolation system should be designed to have sufficient stiffness and strength to avoid large inelastic displacements. For this reason, the SEAOC/UBC provisions contain criteria that limit the inelastic response of the structure above the isolation system. Although damage control for the Design-Basis Earthquake is not an explicit objective of the SEAOC/UBC provisions, an

isolated structure designed to limit inelastic response of the structural system will also reduce the level of damage that would otherwise occur during an earthquake. Isolated structures designed in conformance with the SEAOC/UBC provisions should, in general, be able to:

1. resist minor and moderate levels of earthquake ground motion without damage to structural elements, nonstructural components or building contents, and
2. resist major levels of earthquake ground motion without failure of the isolation system, without significant damage to structural elements, without extensive damage to nonstructural components, and without major disruption to facility function.

The performance objectives for isolated structures, stated above, considerably exceed the performance anticipated for fixed-base structures during moderate and major earthquakes. Table 1 provides a tabular comparison of the performance expected for isolated and fixed-base structures designed in accordance with SEAOC/UBC provisions. Loss of function is not included in Table 1. For certain (fixed-base) facilities loss of function would not be expected to occur until there is significant structural damage causing closure or restricted access to the building. In other cases, the facility could have only limited or no structural damage, but would not be functional as a result of damage to vital nonstructural components and contents. Isolation would be expected to mitigate structural and nonstructural and to protect the facility against loss of function.

Table 1 Protection Provided by SEAOC/UBC Provisions for Minor, Moderate and Major Levels of Earthquake Ground Motion

Risk Category	Earthquake Ground Motion Level		
	Minor	Moderate	Major
Life Safety ¹	F/I	F/I	F/I
Structural Damage ²	F/I	F/I	I
Nonstructural Damage ³ (Contents Damage)	F/I	I	I

1. Loss of life or serious injury is not expected for fixed-base (F) or isolated (I) buildings.
2. Significant structural damage is not expected for fixed-base (F) or isolated (I) buildings.
3. Significant nonstructural (contents) damage is not expected for fixed-base (F) or isolated (I) buildings.

DESIGN METHODS AND REQUIREMENTS

Rather than addressing a specific method of base isolation, the SEAOC/UBC provisions include general design requirements applicable to a wide range of possible seismic isolation systems. In remaining general, the design provisions rely on mandatory testing of isolation system hardware to confirm the engineering parameters used in the design and to verify the overall adequacy of the isolation system. Some systems may not be capable of

demonstrating acceptability by test and, consequently, would not be permitted. In general, acceptable systems will:

1. remain stable for the required design displacement,
2. provide increasing resistance with increasing displacement,
3. not degrade under repeated cyclic load, and
4. have well-defined engineering parameters (e.g., established and repeatable force-deflection characteristics).

The SEAOC/UBC provisions prescribe the use of either one of two different methods for determining Design-Basis Earthquake load. The first method is based on equivalent static analysis and uses a formula (similar to the seismic-coefficient formula required by the 1988 Blue Book for design of fixed-base buildings) to prescribe peak lateral displacement of the isolation system. The static analysis method is intended primarily as a procedure for prescribing "minimum" design criteria and may be used for design of only a very limited class of isolated structures (without confirmatory dynamic analyses). The simple formulas of the static analysis method are useful for preliminary design and provide a means of expeditious review of more complex calculations.

The second method, which is required for most isolated structures (and recommended for all important isolation projects), relies on dynamic analysis to determine maximum response of the isolated structure. Either response spectrum or time history analysis is required for design of the following isolated structures:

1. isolated structures with a stiffness, weight or geometrical vertical irregularity or other irregularity (above the isolation interface), for which the SEAOC/UBC provisions require dynamic analysis of fixed-base structures,
2. isolated structures located within 15 km of an active fault,
3. isolated structures located on a soil profile with a site factor of S_3 or S_4 (i.e., a soft and very soft soil site),
4. isolated structures located in Seismic Zones 0, 1, 2A or 2B,
5. isolated structures greater than 4 stories or 65 feet in height (above the isolation system),
6. isolated structures with a period, T_1 , greater than 3.0 seconds, and
7. isolated structures with a period, T_1 , which is less than 3 times the elastic, fixed-base period of the structure above the isolation interface.

Additionally, time history analysis is required to determine the design displacement of the isolation system (and the structure above) for the following isolated structures:

1. isolated structures with a "nonlinear" isolation system, including but not limited to: isolation systems utilizing friction or sliding surfaces, isolation systems with effective damping values greater than about 30 percent of critical, isolation systems not capable of producing a significant restoring force and isolation systems that restrain

or limit extreme earthquake displacement,

2. isolated structures with a "nonlinear" structure (above the isolation system) including, but not limited to: structures designed for forces that are less than those specified by the SEAOC/UBC provisions for "essentially elastic" design, and
3. isolated structures located on a soil profile with a site factor of S_4 (i.e., very soft soil).

In the case of isolated structures located on a soil profile with a site factor of S_3 or S_4 , or located within 15 km of an active fault, site-specific design spectra, developed in accordance with the requirements of the 1988 Blue Book, are also required for design.

The restrictions placed on the use of static analysis effectively require dynamic analysis for virtually all isolated structures. However, lower-bound limits on isolation system design displacement and force are specified by the SEAOC/UBC provisions as a percentage of the values prescribed by the static-analysis formulas, even when dynamic analysis is used as the basis for design. These lower-bound limits on key design attributes ensure consistency in the design of isolated structures and serve as a "safety net" against gross underdesign. Table 2 provides a summary of the lower-bound limits on dynamic analysis specified by the SEAOC/UBC provisions.

Table 2 Lower-Bound Limits on Dynamic Analysis Specified as a Percentage of Static-Analysis Design Requirements

Design Parameter	Response Spectrum Analysis	Time History Analysis
Design-Basis Earthquake Displacement	90%	90%
Maximum-Credible Earthquake Displacement	80%	80%
Shear Force on Isolation System and Structure Below	90%	90%
Shear Force on Structure Above the Isolation System ("Regular" Configuration)	80%	60%
Shear Force on Structure Above the Isolation System ("Irregular" Configuration)	100%	80%

Design Displacement of the Isolation System

The SEAOC/UBC provisions specify methods for performing either response spectrum or time history analysis. When response spectrum analysis is used as the basis for design, the damping level of the Design-Basis Earthquake (or Maximum-Credible Earthquake) spectrum should be selected in consideration of the damping characteristics of the isolated structure. At long periods (e.g., 1 second, or greater), the damping level of the spectrum will be influenced predominantly by the effective damping of the isolation system.

At shorter periods, the damping level of the response spectrum will be influenced predominantly by the effective damping of the structure (above the isolation system). Response spectrum analysis can underestimate peak force for structures with "nonlinear" isolation systems. Therefore, structures with "nonlinear" isolation systems, including but not limited to isolation systems with effective damping of 30%, or greater, require time history analysis.

When time history analysis is used as the basis for design, the design displacement of the isolation system and design forces in elements of the structure above are to be based on the envelop of the results of not less than three separate analyses, each using a different pair of horizontal time histories. Each pair of horizontal time histories is to be of a duration consistent with the Design-Basis (or Maximum-Credible) Earthquake, incorporate near-field phenomena, as appropriate, and have response spectra whose square-root-sum-of-the-squares combination of the two horizontal components equals or exceeds 1.3 times the "target" spectrum at each spectral ordinate.

When static analysis is used as the basis for design (or used to determine lower-bound limits on dynamic analysis results), Design-Basis Earthquake displacement is prescribed by a formula which relates the shape of the earthquake spectrum, isolated-structure period and isolation system damping, as follows:

$$D = \frac{10ZNS_I T_I}{B} \quad (1)$$

- where:
- D = Design Displacement, in inches, at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Formula 1,
 - Z = numerical coefficient related to the seismicity of a region, as set forth in the SEAOC/UBC provisions (e.g., Z = 0.4 for Seismic Zone 4),
 - N = numerical coefficient based on the proximity of the structure to active faults (e.g., N = 1.0 for sites greater than 15 km from active faults),
 - S_I = numerical coefficient based on the soil profile (e.g., S_I = 1.5 for soil type S₂),
 - B = numerical coefficient related to the effective damping of the isolation system at the design displacement (e.g., B = 1.5 for isolation systems with effective damping, β , of 20% of critical), and
 - T_I = effective period of the isolated structure, in seconds, in the direction under consideration, as prescribed below by Formula 2.

The effective period, T_I, and effective damping β , of the isolated structure and the effective stiffness of an isolator unit, k_{eff}, are amplitude-dependent parameters, defined as follows:

$$T_I = 2\pi \sqrt{\frac{W}{k_{\min}}} g \quad (2)$$

$$\beta = \frac{1}{2\pi} \left(\frac{\text{Total Area}}{k_{\max} D^2} \right) \quad (3)$$

$$k_{\text{eff}} = \frac{F^+ - F^-}{\Delta^+ - \Delta^-} \quad (4)$$

- where:
- W = weight of the isolated structure, in kips,
 - g = acceleration constant due to gravity (i.e., 386.4 inches/second²),
 - $k_{\min}$ = minimum effective stiffness of the isolation system at the Design Displacement, D , in the horizontal direction under consideration (kips/in), based on lower-bound effective stiffness of isolator units,
 - $k_{\max}$ = maximum effective stiffness of the isolation system at the Design Displacement, D , in the horizontal direction under consideration (kips/in), based on upper-bound effective stiffness of isolator units,
 - k_{eff} = effective stiffness of an isolator unit in the horizontal direction under consideration as determined by cyclic-load testing of isolator prototypes,
 - F^+ (F^-) = maximum positive (negative) force in an isolator unit during a single cycle of prototype testing at a displacement amplitude of Δ^+ (Δ^-),
 - Δ^+ (Δ^-) = maximum positive (negative) displacement of an isolator unit during a single cycle of prototype testing, and
 - Total Area = sum of the areas of the hysteresis loops of all isolator units (kip-inches), where the area of the hysteresis loop of each isolator unit is the minimum area determined by cyclic-load testing at the Design Displacement, D .

The SEAOC/UBC provisions also prescribe design displacement, including torsional effects, as follows:

$$D_T = D \left[1 + y \left(\frac{12e}{(b^2 + d^2)} \right) \right] \quad (5)$$

- where:
- D_T = the Total Design Displacement, in inches, of the isolation system including both the translational displacement, D , at the center of rigidity and the component of torsional displacement in the direction

under consideration,

- y = the distance, in feet, between the center of isolation system rigidity and the point of interest, measured perpendicular to the direction under consideration,
- e = the actual eccentricity, in feet, between the center of mass of the structure and the center of rigidity of the isolation system, plus accidental eccentricity, in feet, taken as 5 percent of the maximum plan dimension of the structure in the direction under consideration,
- d = the maximum plan dimension of the structure, in feet,
- b = the minimum plan dimension of the structure, in feet, measured perpendicular to the direction of the plan dimension, d.

The additional component of displacement due to torsion, as prescribed by Formula 5, increases the design displacement at the corner of the structure by about 15% (for a perfectly square building in plan) to about 30% (for a very long, rectangular building), if the eccentricity is 5 percent of the maximum plan dimension. Such additional displacement due to torsion is appropriate for buildings with an isolation system whose stiffness is uniformly distributed in plan. Isolation systems that have stiffness concentrated toward the perimeter of the building or certain sliding systems that minimize the effects of mass eccentricity will have reduced displacements due to torsion. The SEAOC/UBC provisions permit values of D_T as small as $1.1D$, with proper justification.

When static analysis is used as the basis for design (or used to determine lower-bound limits on dynamic analysis), Maximum-Credible Earthquake displacement is prescribed by the Total Maximum Displacement, D_{TM} , as follows:

$$D_{TM} = 1.5 D_T \quad (6)$$

The Total Maximum Displacement is used to verify the strength and stability of the isolation system and to establish minimum separations around the isolated structure. At least two prototypes of each type of isolator unit must be tested for full vertical load at the Total Maximum Displacement to check isolation system stability for the Maximum-Credible Earthquake.

Design Force on the Structure Above the Isolation System

When dynamic analysis is used as the basis for design, the response of the structure above the isolation system is determined either by response spectrum or time history analysis, as previously described. The earthquake forces used for allowable-strength design of the isolation system and the structure below may be taken as two-thirds of the values calculated by dynamic analysis. The earthquake forces used for allowable-strength design of the structure above the isolation system may be taken as the values calculated by dynamic analysis divided by a response modification factor, R_{wi} . The response modification factor, R_{wi} , is a function of the type of structural system used to resist lateral force. Values of R_{wi} are provided in Table 3 for different types of lateral load-resisting systems. For comparison, Table 3 also includes values of the response modification factor, R_w , for fixed-base buildings

with the same type of lateral load-resisting system. Values of R_w range from 4 to 12 for fixed-base structures and values of R_{wI} range from 1.5 to 3.0 for isolated structures. The larger the R_w or R_{wI} value the greater the amount of inelastic response expected during a major earthquake. For fixed-base structures significant inelastic response would be expected while only limited inelastic response of the superstructure would be expected for an isolated structure.

Table 3 Comparison of Response Modification Factors for Fixed-Base and Isolated Structures

Basic Structural System	Lateral Load-Resisting System	Fixed-Base Structures R_w	Isolated Structures R_{wI}
Bearing Wall	Concrete or Masonry Shear Walls	6	2.6
	Light-Framed Walls with Shear Panels	6	2.2
Building Frame System	Concrete or Masonry Shear Walls	8	3.0
	Steel Concentric-Braced Frame	8	2.2
Moment Resisting Frame System	Special Moment-Resisting Frame (SMRF)	12	3.0
	Steel Ordinary Moment-Resisting Frame (OMRF)	6	1.8
Dual Systems	Masonry Shear Walls with SMRF	8	3.0
	Steel Concentric-Braced Frame with Steel SMRF	10	2.2

When static analysis is used as the basis for design (or used to determine lower-bound limits on dynamic analysis), Design-Basis Earthquake force on the isolated structure is prescribed as follows:

$$V_b = \frac{k_{\max} D}{1.5} \quad (7)$$

$$V_s = \frac{k_{\max} D}{R_{wI}} \quad (8)$$

where: V_b = total lateral seismic force or shear, in kips, on elements of the isolation system and the structure below, and
 V_s = total lateral seismic force or shear, in kips, on elements of the structure above the isolation system.

The static-analysis formulas use the same reduction factors as those prescribed for dynamic analysis. The only difference is that the reduction factors are used in dynamic analysis to directly determine design forces in elements, rather than to establish global design shears, V_b and V_s .

Whether determined by static analysis or by dynamic analysis, the SEAOC/UBC provisions specify that the total lateral shear used for design of the structure above the isolation system is not to be taken as less than each of the following:

1. the lateral seismic force required by governing building codes or applicable design documents for a fixed-base structure with an empirical period equal to the isolated period, T ,
2. the base shear corresponding to the design wind load, and
3. the lateral seismic force required to fully activate the isolation system (e.g., the yield level of a softening system, the ultimate capacity of a sacrificial wind-restraint system or the static friction level of a sliding system).

Vertical Distribution of Force

When dynamic analysis is used for design, the analysis will explicitly distribute lateral force to floors and elements of the structural model. When static analysis is used for design the distribution of force over the height of the structure is specified as follows:

$$F_x = \frac{V_s w_x}{\sum w_i} \quad (9)$$

where: F_x = the lateral force at level x , and

w_x (w_i) = the weight of the structure at level x (level i).

Equation (9) assumes a uniform distribution of lateral acceleration over the height of the structure. This assumption is valid for isolated structures that have very rigid superstructures. When the superstructure is flexible, lateral displacement will occur in the superstructure as well as in the isolation system and dynamic analysis is required to determine the distribution force over the height of the structure. Dynamic analysis is required for flexible structures since higher modes will contribute to lateral response of the superstructure. When the isolation system is "nonlinear," time history analysis is required to determine the distribution of force over the height of the structure. Time history analysis is required for "nonlinear" isolation systems since higher mode excitation by "nonlinear" systems is not accurately calculated by response spectrum analysis.

Drift Limits

The maximum interstory drift permitted for design of isolated structures varies depending on the method of analysis used, as summarized in Table 4. For comparison, the drift limits prescribed by the SEAOC/UBC provisions for fixed-base structures are also

summarized in Table 4.

Table 4 Comparison of Drift Limits for Fixed-Base and Isolated Structures

Structure	Static Analysis	Response Spectrum Analysis	Time History Analysis
Isolated Structures	$0.010/R_{wI}$	$0.015/R_{wI}$	$0.020/R_{wI}$
Fixed-Base Structures ($T < 0.7$ seconds)	$0.015/(3R_w/8)$ or $0.005 \times \text{story height}$	$0.015/(3R_w/8)$ or $0.005 \times \text{story height}$	$0.015/(3R_w/8)$ or $0.005 \times \text{story height}$
Fixed-Base Structures ($T \geq 0.7$ seconds)	$0.011/(3R_w/8)$ or $0.004 \times \text{story height}$	$0.011/(3R_w/8)$ or $0.004 \times \text{story height}$	$0.011/(3R_w/8)$ or $0.004 \times \text{story height}$

Drift limits in Table 4 are expressed as a numerical coefficient divided by either R_{wI} for isolated structures or by $3R_w/8$ for fixed-base structures. Drift limits are expressed in terms of $1/R_{wI}$ or $1/(3R_w/8)$ since drift limits are used to check displacement for "reduced" forces (i.e., lateral loads reduced either by R_{wI} for isolated structures or by R_w for fixed-base structures). The " $3R_w/8$ " term is used throughout the SEAOC/UBC provisions for fixed-base structures as an approximate ratio of actual earthquake response to response calculated for "reduced" forces.

The numerical coefficients given in Table 4 (e.g., 0.015 for a fixed-base structure with a period less than 0.7 seconds) represent the maximum drift intended by the SEAOC/UBC provisions for lateral forces corresponding to the Design-Basis earthquake. It may be seen that the drift limits for fixed-base structures are comparable to those prescribed for static or response spectrum analysis of isolated structures. Drift limits for isolated structures designed using time history analysis methods are slightly more liberal. More liberal drift limits are considered appropriate for buildings designed using time history analysis, since this method of analysis is expected to provide a more realistic estimate of lateral deflection.

DESIGN EXAMPLE

Figure 1 illustrates lateral design displacement and force as a function of isolated structure period for an example building with the following design conditions:

1. the superstructure is a steel, concentric-braced frame (i.e., $R_{wI} = 2.2$),
2. the isolation system has 20% effective damping (i.e., $B = 1.5$) and has been configured to resist torsion ($D_T = 1.1D$), and
3. the site is located in Seismic Zone 4 (i.e., $Z = 0.4$), has a site profile of soil type S_2 (i.e., $S_I = 1.5$) and is at least 15 km from all active faults (i.e., $N = 1.0$).

Figure 1 . Design Example: Lateral Design Displacement and Force as a Function of Isolated Structure Period

The example design conditions are typical of buildings located in Seismic Zone 4 away from faults. The displacement and force curves shown in Figure 1 are also typical of buildings located near faults, provided the site conditions are firm soil or rock (i.e., $N = 1.5$, $S_I = 1.0$).

Figure 1 illustrates the basic trend inherent to isolated-structure response: a decrease in force and an increase in displacement with increase in the effective period. Effective periods of 2.0 seconds to 3.0 seconds are typical for isolated structures. At 2.5 seconds, the Maximum-Credible Earthquake displacement, D_{TM} , is about 17 inches and the corresponding maximum lateral force felt by the superstructure, $k_{max}D$, is about $0.17W$. The lateral force required for design of the isolation system and structure below, V_b , is about $0.11W$ and the lateral force required for design of the superstructure, V_s , is about $0.07W$. For comparison, the lateral force, V , required for design of a 2.5 second, steel concentric-braced frame, fixed-base structure is about $0.04W$.

SEISMIC ISOLATION RETROFIT PROJECTS

To date, four buildings have been (or are in the process of being) retrofitted using seismic isolation. These buildings are the Salt Lake City and County Building [Ref. 8], the Rockwell Building in Seal Beach, California [Ref. 9], the Hawley Apartments in San Francisco, California [Ref. 10] and the Mackay School of Mines in Reno, Nevada [Ref. 11]. A number of other buildings have gone through conceptual retrofit design and are moving

toward development of final designs using seismic isolation. A summary of these projects is provided in Table 5. Table 5 also includes one project that is still in the conceptual design phase (i.e., San Francisco Asian Art Museum). Although the decision to proceed ahead with isolation has not yet been made for this project, the value of the art (estimated to be about \$3 billion) provides a compelling reason to isolate the building. At least two other large retrofit projects are developing isolation designs, but the owners have requested that these projects not be discussed at this time.

The retrofit projects summarized in Table 5 range from a 20,000 square foot building to buildings of up to 350,000 square feet. The original structural systems of these buildings include: wood bearing walls, "non-ductile" reinforced-concrete moment frames, steel moment frames with unreinforced-masonry (URM) in-fill and URM bearing walls. Most of the buildings are owned by a local, state or federal government agency and often have historical significance. The collective size of the buildings in Table 5 is about 2 million square feet and the combined value of these buildings is at least \$500 million.

Isolation systems of the four buildings that have been retrofitted to date include: lead-rubber bearing (LRB) isolators, friction-pendulum system (FPS) isolators, high-damping rubber (HDR) isolators and sliding polytetrafluoroethylene (PTFE) isolators. These types of isolators are representative of the range of products currently available in the United States. These projects have required as few as 31 isolators for the Hawley Apartments, a 4-story, 20,000 square foot residential building, and as many as 447 isolators for the Salt Lake City and County Building, a 5-story, 170,000 square foot historical structure. The extent of new structure added above the isolation system also varies greatly from one project to another. In some cases, such as the Mackay School of Mines, only minimal strengthening of the original structure is specified. In other cases, such as the Rockwell Building, a substantial amount of new structure is added to the original building.

As a starting point for developing design guidelines, this paper has compiled a summary of the seismic-isolation retrofit criteria used (or being used) for each of the projects listed in Table 5. This summary is included as Table 6 and contains information on seismic criteria, performance objectives, isolation system criteria and structural system criteria. This information was obtained from published references (i.e., Refs. 8-11) and from input provided by engineers familiar with these projects. As noted in Table 6, the design criteria for on-going projects should be considered preliminary and subject to change before the design is finalized.

Table 6 has attempted to compile the same type of data for each project in a consistent format. In certain cases, the cognizant engineer was requested to use judgement to establish values for items that are not included explicitly as design criteria. In particular, performance objectives (e.g., "minor" or "moderate" damage) are often implicit, rather than explicit, project criteria. A "life safety" performance objective in Table 6 indicates that limits on damage (or building function) are not project criteria, even though some degree of damage protection may be provided. Limits on response have been characterized throughout Table 6 in terms of "yield" or "ultimate" strength of lateral force system elements. Again, the cognizant engineer was requested to express response limits in these terms even though the criteria used for the project may have been different. For example, the response limits for the Rockwell Building were based on allowable concrete strains (rather than stresses).

Table 5 Summary of Seismic-Isolation Retrofit Projects in the United States

Building/Project Information			Structural Information ¹		
Name (Location)	Status	Size (Sq. Ft.)	Isolation System ²	Original Structure	New Structure
Salt Lake City and County Building	Retrofit Complete (1988)	170,000	447 LRB Isolators	1894 5-Story URM Bearing Wall w/Clock Tower (240' total height)	Steel Braced Frame (Clock Tower Only)
Rockwell Building (Seal Beach)	Retrofit Complete (1991)	300,000	78 LRB Isolators	1967 8-Story R/C Moment Frame	R/C Moment Frame at Perimeter, Floors 1-6
Hawley Apartments (San Francisco)	Retrofit Complete (1991)	20,000	31 FPS Isolators	1920 4-Story Wood Bearing Wall	Steel Moment Frame at 1st Floor
Mackay School of Mines (Reno, Nevada)	Retrofit in Progress	50,000	64 HDR + 42 PTFE Isolators	1908 3-Story URM Bearing Wall	Floor Ties/Wall Anchors (New Basement)
Long Beach VA Hospital	Final Design In Progress	350,000	144 LRB + 12 PTFE Isolators	1967 12-Story R/C Perforated Shear Wall	Basement Columns Strengthened
Hayward City Hall	Schematic Design In Progress	145,000	≈ 36 Isolators	1969 11-Story R/C Moment Frame	Steel Braced Frame at 1st/2nd Floors
Oakland City Hall	Conceptual Design Complete	153,000	≈ 100 Isolators	1914 18-Story Steel Frame/URM In-Fill w/Clock Tower (324' total height)	R/C Shear Walls at Cores, Steel Braced Frame at Clock Tower
State of California Justice Building (San Francisco)	Conceptual Design Complete	250,000	≈ 170 Isolators	1919 6-Story Steel Frame/URM In-Fill	R/C Shear Walls (and Building Expansion)
U.S. Court of Appeals (San Francisco)	Conceptual Design Complete	350,000	≈ 250 Isolators	1905 4-Story Steel Frame/URM In-Fill w/1933 Addition	R/C Shear Walls
U.S. Customs House (San Francisco)	Conceptual Design Complete	142,000	≈ 300 Isolators	1911 5-Story Steel Frame/URM In-Fill around 2-Story URM Bearing Wall	Center Core Strengthening of URM Bearing Walls and Main Building Parapets
San Francisco Asian Art Museum (currently the Main Public Library of San Francisco)	Conceptual Design In Progress	170,000	≈ 200 Isolators	1916 3-Story Steel Frame/URM in-Fill	R/C Shear Walls (and Building Expansion)

1. Structural information for on-going projects should be considered preliminary and subject to change.
2. Types of isolators include: lead-rubber bearing (LRB) isolators, friction-pendulum system (FPS) isolators, high-damping rubber (HDR) isolators and sliding polytetrafluoroethylene (PTFE) isolators.

Table 6 Summary of Seismic-Isolation Retrofit Design Criteria¹

Building Name (Location)	Seismic Criteria ²	Performance Objectives (Damage ³)	Isolation System Design Criteria ⁴	Structural System Design Criteria		
				Force Limits ⁵	Drift Limits	System
Salt Lake City and County Building	DBE	Minor	SEAONC ($F_{SD+LL} = 2$)	$\approx \text{Yield} \times 2$	None	Existing Elements (except Tower)
	MCE	Life Safety	SEAONC	None	None	
Rockwell Building (Seal Beach)	DBE	Minor (2%)	N/A	$< \text{Yield}$	$\Delta/h \leq 0.005$	New/ Existing Elements
	MCE = 7 (Newport)	Minor (5%)	SEAOC/UBC	$< \text{Yield}$	$\Delta/h \leq 0.005$	
Hawley Apartments (San Francisco)	DE = 7 (San Andreas)	Minor	SEAONC	$\approx \text{Yield} \times 2$	$\Delta/h \leq 0.005$	Existing Elements
Mackay School of Mines (Reno, Nevada)	DBE	Minor	SEAONC ($D_{DBE} \times 1.5$)	No Tension Force in URM	None	Existing Elements Only
	MCE = 8 (Owen's Valley)	Minor	SEAONC	No Tension Force in URM	None	
Long Beach VA Hospital	DBE	Minor	N/A	Yield	$\Delta/h < 0.001$	Existing Elements Only
	MCE	Minor	SEAOC/UBC	Yield	$\Delta/h < 0.001$	
Hayward City Hall	DE = 7.0 (Hayward)	Minor	N/A	Yield $\times 1.0-1.8$	TBD	Existing/ New Elements
	MCE = 7.5 (Hayward)	Life Safety	SEAOC/UBC	Ultimate	TBD	
Oakland City Hall	DBE	Minor	N/A	Ultimate	None	Existing/ New Elements
	MCE = 7 (Hayward)	Life Safety	SEAOC/UBC	None	None	
State of California Justice Building (San Francisco)	DBE	Minor	N/A	$\approx \text{Yield} \times 2$	$\Delta/h \leq 0.0033$	New Elements Only
	MCE	Life Safety	SEAOC/UBC	None	None	
U.S. Court of Appeals (San Francisco)	DBE	Minor	SEAOC/UBC ($D_{DBE} \times 1.25$)	TBD	TBD	Existing/ New Elements
U.S. Custom House (San Francisco)	DBE	Minor	N/A	TBD	TBD	Existing Elements
	MCE	Moderate	SEAOC/UBC	TBD	TBD	
San Francisco Asian Art Museum (currently the Main Public Library of San Francisco)	DE = 7 (San Andreas)	Minor	N/A	Yield $\times 1.5$	$\Delta/h < 0.0015$	New Elements Only
	MCE	Minor	SEAOC/UBC	Yield $\times 2.5$	$\Delta/h < 0.002$	

1. Design criteria for on-going projects should be considered preliminary and subject to change.
2. Design-Basis Earthquake (DBE) and Maximum-Credible Earthquake (MCE) are event levels defined by the SEAOC/UBC provisions.
3. "Minor" damage implies no, or only short-term, loss of building function; "moderate" indicates significant, but repairable damage; "life-safety" indicates no limits on damage or building function.
4. Isolation system criteria are specified by SEAOC/UBC provisions and SEAONC provisions [Ref. 5].
5. Force limits are specified in terms of "yield" and "ultimate" strength of lateral force system elements.

The design criteria for each project varies depending on the type of structure being retrofitted and the performance desired. Seismic criteria are usually consistent with the definitions of the Design-Basis Earthquake (DBE) and/or the Maximum-Credible Earthquake (MCE), as specified by the SEAOC/UBC provisions. Isolation system design criteria also tend to conform to the SEAOC/UBC provisions, except for older projects which relied on the SEAONC provisions [Ref. 5]. Structural system criteria varies the most from project to project, with limits on DBE response ranging from less than yield to as much as the ultimate capacity of elements. As a point of reference, the design criteria for the State of California Justice Building are based directly on the SEAOC/UBC provisions for new construction (e.g., DBE response of structural elements limited to about yield $\times$ 2). As Table 6 indicates, retrofit projects with stringent operational objectives, such as the Rockwell Building, or with special damage protection goals, such as the San Francisco Asian Art Museum, have specified design criteria that exceed SEAOC/UBC design requirements for new construction.

TENTATIVE SEISMIC-ISOLATION RETROFIT DESIGN GUIDELINES

The philosophy or basis for seismic-isolation retrofit design criteria is directly dependent on the owner's motivation to upgrade the building and expectations of upgraded-building performance during and following an earthquake. For this reason the design criteria are best developed on a case-by-case basis.

To date, there are five primary considerations that have motivated owners to choose isolation for retrofit of existing buildings:

1. Functionality. The facility should remain open and operational during and after an earthquake or be able to resume operation within a short period of time (e.g., Rockwell Building).
2. Contents Protection. Important contents are protected against damage due to earthquake shaking (e.g., San Francisco Asian Art Museum).
3. Investment Protection. Long-term economic loss due to earthquake damage is mitigated (e.g., State of California Justice Building).
4. Historical Building Preservation. Seismic-retrofit modification or demolition of historical building features is minimized (e.g., Salt Lake City and County Building).
5. Design Economy. The building is of a size and/or complexity that seismic isolation is the most economical construction alternative (e.g. Oakland City Hall).

Each retrofit project will have a different set of motivating factors and related performance objectives and therefore will likely require different design criteria. The first, essential, step in developing design criteria is to identify and rank the owner's seismic risk goals in terms of facility function, damage and investment protection, historical preservation and design economy. It will be these goals that guide the engineer's selection of performance objectives and design criteria appropriate for the building. Owners with

functionality, contents or investment protection goals high on their list of priorities, will require more stringent design criteria. Owners with historical preservation or design economy goals at the top of their list will require less stringent design criteria.

All retrofit projects require specification of seismic criteria (i.e., earthquake ground motion). In general, ground motion levels should be consistent with the Design-Basis Earthquake (DBE) and Maximum-Credible Earthquake (MCE), as defined by the SEAOC/UBC provisions for new construction. A case might be made for using less conservative seismic criteria if it can be shown that the isolated building provides life-safety protection comparable to conventionally-retrofitted buildings. However, such an argument would not be appropriate for projects with goals dominated by special damage protection or functionality objectives.

In general, isolation system design criteria for retrofit projects should be based on the SEAOC/UBC provisions for new construction. Basic requirements for stability of isolators, prototype testing, quality control, etc., are as valid for retrofit projects as for new construction. A case might be made for less conservative limits on clearances around the isolated building, provided life safety is not compromised. Again, such an argument would not be appropriate for projects with goals dominated by special damage protection or functionality objectives. Peer review of the isolation system should be performed for all isolation retrofit projects, as required by the SEAOC/UBC provisions for new construction. However, the extent of the review should be gaged to the size and importance of the project. Large, important projects require full design and construction review by a panel of seismic isolation, structural and geotechnical experts; while small projects may be adequately checked by building authorities with only limited oversight by an outside consultant.

Structural design criteria and related methods of analysis for retrofit projects may be grouped into one of three basic performance categories: performance greater than SEAOC/UBC provisions, performance comparable to SEAOC/UBC provisions and performance less than SEAOC/UBC provisions. The first category includes facilities with very stringent performance objectives in terms of either functionality or damage protection. The second category includes buildings with long-term investment protection goals consistent with those implicit in the SEAOC/UBC provisions for new construction. The last category includes buildings whose performance objectives are dominated by design economy or the need to preserve historical features of the building. A summary of tentative structure design criteria are summarized below for each of these three categories.

Performance Greater than SEAOC/UBC Provisions

Buildings that must function, with high confidence, during and after an earthquake or sustain virtually no damage should:

- limit DBE (MCE) response of elements to about yield, or less (e.g., $R_{wi} \leq 1.5$),
- limit DBE (MCE) drift to about one-half of the SEAOC/UBC provisions,
- limit DBE (MCE) floor acceleration as necessary to protect equipment function or fragile contents.

Whether the above criteria are applied to DBE or MCE response will depend on the specific seismic risk goals established by the owner for the building. Since the structural system will remain elastic (response less than yield), the design may be based solely on elastic models of the structure.

Performance Comparable to SEAOC/UBC Provisions

Buildings that have damage protection goals comparable to those of the SEAOC/UBC should:

- limit DBE response of elements to less than about two times yield (e.g., $R_{wi} \leq 3$),
- limit DBE drift to that permitted by the SEAOC/UBC provisions,
- verify post-yield capacity of existing (non-ductile) elements used as part of the lateral force resisting system.

The method of analysis appropriate for design of the structure will depend on the degree of nonlinear response considered acceptable and the type of materials used to resist lateral forces. If forces are resisted entirely by new (ductile) structural elements, then an elastic model may be used for design (as permitted by the SEAOC/UBC provisions). If significant post-yield response of existing (non-ductile) elements is permitted, then structure stability should be checked using a nonlinear model of the structure.

Performance Less than SEAOC/UBC Provisions (Life-Safety Minimum)

Buildings that need not be functional after an earthquake nor provide a specified level of damage protection should, as a minimum:

- limit DBE response of elements to less than ultimate capacity,
- limit DBE drift to about one-half of that which would cause structure instability,
- verify post-yield capacity of existing (non-ductile) elements used as part of the lateral force resisting system.

Since the structure criteria given above rely on significant post-yield response of elements, the design should be verified using a nonlinear model of the structure.

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Douglas Way (Mackay School of Mines), and Nabih Youssef (Long Beach VA Hospital). The assistance provided by these engineers is greatly appreciated.

The portion of this paper that describes SEAOC/UBC design provisions for seismic isolation of new construction is an update of previous work, first compiled for the ASCE Structures Congress in May 1989 [Ref. 12] and revised for presentation at a 1990 U.S./Japan workshop on the improvement of building structure design practices [Ref. 13].

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GERB BASE SEISMIC ISOLATORS FOR THE NEW EMERGENCY FACILITY IN NPP V1 JASLOVSKE BOHUNICE, CZECHOSLOVAKIA

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S u m m a r y

The NPP V1 Jaslovske Bohunice is the oldest operating Czechoslovak atomic power plant equipped with two Soviet PWR VVER 440 type reactors. This NPP was designed and constructed many years ago without any seismic analysis and qualification. Today the actual SSE is estimated about 8° MSK-64 with ZPGA equal to 0.25g at this site. This means that the seismic reinforcement of this NPP has now become a very urgent issue. After a careful evaluation of existing plant designs and their capacities to withstand the impact of earthquakes and other similar external or internal events, the Czechoslovak Atomic Commission is now asking the utility to build an additional bunkered safe shutdown facility capable of maintaining the plant in a safe shutdown condition during the earthquake and 72 hours after it. This building should be located near the reactor building and must have an independent I&C and power supply. The main idea is such that the new emergency reactor cooling system, diesel generators, I&C system, and their supporting systems will be housed in this new building, which should be designed to withstand the impact of any earthquake accident up to the SSE level. A base seismic isolation system is assumed to be used for this building in which the most important and sensitive safety-related emergency equipment will be placed (Figure 1).

Base seismic isolation is significant advanced development in earthquake engineering that is now rapidly gaining worldwide acceptance /1/, namely when used for safety-related facilities the importance of which far exceeds their construction costs. This approach introduces a damped flexible device between the building base and the ground to decouple the structure from the most important components of seismic motion, resulting in a significant reduction of seismic loads on the structure and equipment inside the building.

Several base seismic isolation systems have been proposed and have been used in a few cases for earthquake protection of such safety-related buildings. In this study the following three concepts have been analyzed and compared:

- conventional foundation of the building on the competent soil,
- seismic base isolation system developed by GERB, Germany, which uses helical springs and viscous dampers (Figure 2) /2/,
- French seismic base isolation system developed by EDF and by Spie Batignolles, France, using laminated neoprene bearings with lead bronze-stainless steel slip plates (Figure 3) /3/,

Spring and dashpot parameters of base isolators have been taken from available test data and estimated in regions outside of this data.

Generally, greater damping up to 15% in average is typical for the GERB base isolators, the spring parameters of which are almost linear, while their damping is rather velocity-dependent. On the other hand, parameters of the EDF base seismic isolators are almost linear up to the sliding limit which depends on the actual friction coefficient between the slip plates (usually about 0.2) and on the vertical loading of the bearing. However, no sliding was computed in this case due to only moderate seismic motion. It should be also noted that the GERB base seismic isolators as shown in Figure 2 may be loaded up to 1500 kN and their natural frequencies under the nominal load are 0.5 and 1.0 Hz in horizontal and vertical directions respectively.

The 2-D finite element model of this building has been subjected to both horizontal and vertical base accelerograms which are assumed representative for the site. The HYPERSAP computer program has been applied for seismic analysis of the model using the conventional time-history technique based on Wilson's numerical integration procedure. Then the floor response spectra have been extracted from the calculated response time histories at several elevations of the building. Those which have been determined for the elevation of 12 m above grade at which the most sensitive I&C systems will be housed are shown in Figures 4 and 5. It is possible to conclude on this basis that:

(1) As shown in Figure 4, there are substantial reductions in horizontal floor response accelerations when both GERB and also EDF base seismic isolation concepts are compared to the conventional building foundation on the soil.

(2) At the same time, as shown in Figure 5, there is a great difference in vertical floor response accelerations, because the base seismic isolators of EDF are relatively stiff in vertical direction while the GERB isolators actually are functional in any direction including vertically.

(3) In general, a large amount of rocking may result in buildings supported on base seismic isolators when they are flexible in the vertical direction. However, because the low center of gravity of this building and its wide and heavy base, no uplift due to rocking occurred in the corner bearings.

The GERB base seismic isolators are relatively expensive. However they will probably provide an excellent seismic protection for such important and safety-related facilities with sensitive equipment inside. In this case they may permit the use sensitive equipment inside the base-isolated building without additional expensive seismic qualification.

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Figure 1 New Emergency Facility, NPP V1 Jaslovske Bohunice, Czechoslovakia

Figure 2 Prototype of the GERB Base Seismic Isolator

Figure 3 Detail of the EDF Base Seismic Isolation Bearing

Fig. 4 Comparison of Horizontal FRS
Elevation 12.0 m above grade (I&C Room)

Fig. 5 Comparison of Vertical FRS
Elevation 12.0 m above grade (I&C Room)

OWNER / DESIGNER CHECKLIST

SEISMIC RETROFIT PROJECTS UTILIZING BASE ISOLATION. (On negotiated projects, the contractor can assist in answering many of the questions below. On bid projects, the Owner/Architect/Engineer team should deal with all of these issues.)

GENERAL INFORMATION

1. Owner should have tough pre-qualification requirements for general contractors.
2. It is suggested that a thorough field survey be done by the architect and engineer to understand/identify all existing elements that tie into or are affected by the new work.
3. It is desirable to schedule pre-construction reviews by an experienced general contractor with architect, engineer and owner to review practicality of details.
4. Suggest several pre-bid job walks on complex jobs. (Subs will likely go on 2nd and 3rd "walk".) Give adequate bid time.
5. How much of the building will be occupied during the work? What hours?
6. Will shift work be allowed (evenings, nights, weekends)?
7. Will shift work be mandated by owner?
8. Is there a staging area for construction trailer, equipment, etc.
9. With a staging plan, provide a barricade plan showing temporary exiting from building for occupants. Any phasing of exiting required to coordinate with construction?
10. What utilities will owner provide to contractors (power, water, telephone)?
11. Will propane operated forklifts be able to operate inside the building to help move material, equipment, and help in the erection of the jacking frames, and placement of the isolator assemblies?

STRUCTURAL INFORMATION

1. What will be the size and number of steel bearing plates used to assemble isolator?
2. What flatness tolerances of the steel bearing plates will be required?
3. How will the top and bottom bearing plates be grouted to the cut concrete column?
4. Can standard size flat jacks be used, or will special sizes be required? Other method of load transfer?
5. What is the calculated dead and live load in each column?
6. How many columns can be cut at one time? Tell bidders if there will be any restriction on sequence.
7. Will lateral restraint be required until all isolators are in place?
8. What is the rebar pattern in the existing column? This will determine cutting method.
9. How are the elevators affected by isolation (e.g. utilities, rails, equipment, etc.)?
10. How are the stairs affected by isolation?
11. Are the isolators/plates to be fireproofed? Must fireproofing survive a major seismic event?
12. Provide specification for doweling and grouting when adding concrete to the existing structure, if required.
 - a. Will pull tests be required?
 - b. If so, what percentage is to be tested? What are the acceptance criteria?
13. Do not dictate method of concrete placement (poured? pneumatic? other?), but do specify strength and soundness required.
14. Will any shoring/support of existing foundation be required? If so, provide criteria (contractor to decide method).
15. What construction loads may be carried by existing floors?

ARCHITECTURAL INFORMATION

1. Will there be access around each column for assembly of jacking frame and will it provide enough clearance for cutting and removal of the concrete column section?
2. Clearance will be needed to install the isolators (use 4 ft. around column as a rule of thumb).
3. Owner should provide a complete architectural demolition plan and a plan showing architectural reconstruction.
4. How does isolation affect architectural considerations (e.g. walls, expansion joints, floors, walls, ceilings)?
5. How will rated walls and other rated transitions be affected? Will walls need additional braces once cut free along the plane of isolation?
6. How will exterior Architectural items be affected at the seismic zone? (ie: canopies, balconies, louvers, exterior skin.)
7. Architectural requirements where/when tying into existing: Should contractor try to match existing or tear out and replace all? Direction should be on a complete architectural demo and reconstruction plan.
8. Is any site work affected? If so, bid documents should include an architectural demo and reconstruction plan, landscape plan.
9. Are there any code-required upgrades to balance of building (e.g. new fire sprinklers, handicapping of restrooms, generator, etc.)? What about handicap ramps at plane of isolation?
10. Do all specified products built into a particular assembly meet current pertinent codes and testing requirements? (Products may work and meet codes individually, but does it all work and satisfy codes built together?)
11. Will special code allowances be required for utility isolation and rated architectural isolation?
12. For occupied buildings, provide temporary exiting plans.

MECHANICAL INFORMATION

1. Clearance will be needed to erect the isolator assembly (as a rule of thumb, contractor needs 4 ft. around column, from the exterior face). They may affect mechanical equipment.
2. Utility demolition plans will be required. (i.e. What utilities need to be removed, isolated or relocated?) What existing underground utilities are affected?
3. How are the stairs affected by isolation? Ventilation? Pressurization?
4. Utility shutdowns/switch-overs: Regular shift? Over-time? Premium time? Contractor will need a scheduled sequence, or criteria for setting a schedule. Will fire protection need to be continuous during course of construction?
5. Mechanical coordination. When utilities pass through seismic zone, allow enough room for dual movement of adjacent utilities.
6. Consider a preliminary field survey to identify all buried utilities. If they are discovered during construction, then it will delay job and add to costs. Do as much field research as possible prior to completion of working documents.
7. Will special code allowances be required for utility isolation and rated architectural isolation?
8. Engineered supports are required immediately above and below all mechanical flex joints (flex-tends, flex-hoses, etc.).
9. If friable HVAC ducts are to be installed, then access doors or removable panels are required on shaft walls, for replacement of friable ducts.
10. All conduits to be flexed should be marked or identified by an engineer in the field.

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R&D ACTIVITIES AT ARGONNE NATIONAL LABORATORY FOR THE APPLICATION OF BASE SEISMIC ISOLATION IN NUCLEAR FACILITIES

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ABSTRACT

Argonne National Laboratory (ANL) has been deeply involved in the development of seismic isolation for use in nuclear facilities for the past decade. Initial focus of these efforts has been on the use of seismic isolation for advanced liquid metal reactors (LMR). This effort is directed toward providing an alternate seismic design strategy for LMR plants, in which low coolant pressures lead to thin-walled vessels and piping which lack the inherent seismic resistance provided in light-water reactor plants (LWR). Subsequent efforts in seismic isolation at ANL included a lead role in an accelerated development program for possible use of seismic isolation for the DOE's New Production reactors (NPR). Under funding provided by the National Science Foundation (NSF) Argonne is currently working with Shimizu in a joint United States-Japanese program on response of seismically-isolated buildings to actual earthquakes.

The seismic isolation program activities at ANL cover a wide spectrum, including the following six (6) elements: (1) development of computer codes to simulate the response of isolator bearings and to evaluate the seismic performance of isolated structures; (2) the development and implementation of an experimental test program for isolator bearings; (3) the performance of material testing on promising elastomer compounds for isolator bearings; (4) establishing a better understanding of the effects of long period ground motions on base isolated structures; (5) the determination of the effects of aging, temperature and other environmental factors on the long term performance of seismic isolation systems; and other environmental factors on the long term performance of seismic isolation systems; and (6) the development of nuclear-grade design and fabrication specifications for the isolators.

The results of recent work in these seismic isolation program elements cited above are described in this paper. The current Status of these programs is presented along with an assessment of work still needed to bring the benefits of this emerging technology to full potential in nuclear reactors and other nuclear facilities.

I. INTRODUCTION AND PURPOSE

Seismic isolation is growing rapidly throughout the world as a cost-effective design strategy for both new structures and in upgrading existing facilities to provide protection against the damaging effects of earthquakes. The use of seismic isolation in nuclear facilities, though more

limited at this time, is receiving increased attention, particularly for advanced nuclear reactor plants such as the liquid-metal reactor (LMR). This paper describes in an overall way, the research and development activities at Argonne which have been determined as necessary to use seismic isolation in nuclear facilities. The paper also includes a brief description of the historical development of seismic isolation in the US LMR program. In addition, certain selected elements of the ANL program are described in some detail to give the reader a flavor of the nature and depth of the ANL program.

II. BACKGROUND

Argonne National Laboratory (ANL) has been deeply involved in the development of seismic isolation for use in nuclear facilities for the past decade. Initial focus of these efforts has been on the use of seismic isolation for advanced liquid metal reactors (LMR). This effort is directed toward providing an alternate seismic design strategy for LMR plans, in which low coolant pressures lead to thin-walled vessels and piping which lack the inherent seismic resistance provided in light-water reactor plants (LWR). Subsequent efforts in seismic isolation at ANL included a lead role in an accelerated development program for possible use of seismic isolation for the DOE's New Production Reactors (NPR). Under funding provided by the National Science Foundation (NSF) Argonne is currently working with Shimizu in a joint United States-Japanese program on response of seismically isolated buildings to actual earthquakes. In carrying out many of these programs, ANL has worked closely with other DOE contractors (General Electric and the Energy Technology and Engineering Center-ETEC) and leading architect-engineering firms such as Bechtel and Burns and Roe. In addition, the staff and seismic isolation testing facilities at the Earthquake Engineering Research Center at the University of California at Berkeley have carried out extensive programs involving quasi-static and dynamic testing of isolator bearings. In addition, Argonne has managed the efforts of a number of experts in seismic isolation design, effects of long-period ground motion, materials and isolator fabrication. Fig. 1 summarizes the interrelationships between Argonne, its funding sponsors, major collaborators and major subcontractors (some of these are as well be discussed later in this reports). Fig,1 summarizes the interrelationships between Argonne, its funding sponsors, major collaborators and major subcontractors.

III. HISTORICAL PERSPECTIVE

Argonne National Laboratory has been a focal point in the United States for the development of several nuclear reactor concepts, including the boiling-water reactor (the Experimental Boiling Water Reactor-1956); and liquid-metal cooled reactors (the Experimental Breeder Reactor I--1951, and the Experimental Breeder Reactor II--1964). ANL has played an active role in other LMR plants, including the FFTF, CRBR, and the current GE reactor, the ALMR (PRISM) concept.

In this section we will present a brief overview of the historical development of seismic isolation R&D for nuclear facilities in the United States. No claim is made here for completeness. Our purpose is to give proper credit to the earlier researchers and engineers and to present an overview of how the program has developed. The author apologizes for any omissions of significant U.S. research both as reported in the open literature and that which is known widely in the R&D community.

In developing nuclear reactor design concepts, Argonne, along with many others has been exploring ways to cope with the high capital costs associated with these plants. For the LMR, especially, it became clear that a significant portion of these costs were due to meeting current seismic design requirements with facilities having thin-walled vessels and piping systems. In the early 1980s, Argonne met with engineers from France who were in the process of designing SuperPhenix 2 (now EFR) to discuss their ideas on lowering capital costs of LMRs and ways to enhance standardization of LMR plant designs. At that time the French had recently used seismic isolation in order to use their standard 900 MWe PWR plant in Koeberg, South Africa, for which site the seismic design requirement exceeded the design levels used in their standard plant. The French stated that it was their plan to use seismic isolation for the next LMR plant (i.e., the standard plant design) to be able to meet seismic requirements at most sites and to reduce capital costs. At that time the United States Department of Energy (DOE) initiated a scoping program at ANL to explore the feasibility of using seismic isolation in US LMR plant design concepts. At about the same time, DOE and the Electric Power Research Institute (EPRI) jointly sponsored work in seismic isolation for the then current US Large LMR plant concept. This latter work extended into joint testing program with CRIEPI in Japan.

Further US design developments in advanced LMR plants, namely the GE PRISM plant and the Rockwell-International SAFR plant, considered the use of seismic isolation in their reference design concepts. The current ALMR concept in the US, the PRISM modular plant design by General Electric, uses seismic isolation for the nuclear reactor module to avoid excessive material thicknesses in the reactor vessel and piping, and to enhance the economical standardization of their plant design. Argonne worked closely with all of these design activities to align its research and development program in seismic isolation with the specific needs of the plant designers.

Fig.2 Presents a graphical illustration of these historical activities. In addition, Fig.2 shows other important and related research which bears heavily on the development of seismic isolation for use in nuclear power plants in the United States. Details of many of these earlier and more recent works be found in Refs.[1] through [11]. Unfortunately, Fig.2 of this draft paper contains a major oversight, namely the omission of a pioneering paper by Kunar and Maini, EPRI Report NO-1220-SR, October, 1979. This paper evaluates just about every conceivable type of seismic isolation system and is still a very interesting document. It is included as Ref.(1) and will be added to the final version of Fig. 2.

IV. THE SCOPE OF ANL ACTIVITIES IN SEISMIC ISOLATION

The seismic isolation program activities at ANL cover a wide spectrum. A unique and distinguishing characteristic of the Argonne program is that it is fully integrated program, covering work from base material development to the determination of design guides, licensing guide lines and In-Service Inspection (ISI) concepts. Basically, the Argonne seismic isolation program consists of six (6) distinct but interrelated elements: (1) development of computer codes to simulate the response of isolator bearings and to evaluate the seismic performance of isolated structures; (2) the development and implementation of an experimental test program for isolator bearings; (3) the performance of material testing on promising elastomer compounds for isolator bearings; (4) establishing a better understanding of the effects of long period ground motions on base isolated structures; (5) the determination of the effects of aging, temperature and other environmental factors on the long term performance of seismic isolation systems; and (6) the development of nuclear-grade design and fabrication specifications for the isolators.

V. EARLY WORK UNDER DOE-NE ADVANCED NUCLEAR REACTOR PROGRAMS

The initial focus of the seismic isolation program at Argonne was concerned with development of satisfactory seismic solutions to the seismic problems associated with liquid metal reactors. Because LMRs operate at very low pressures, the piping, the reactor vessel, and other associated vessels have very thin walls, especially when compared with light water reactor plants. As a result, LMR plants do not have a reserve of intrinsic or inherent seismic resistance built into them. Generally, thermal stresses and other considerations of high temperature dictate the design of these vessel and piping systems. For moderate to strong earthquakes provision of adequate seismic resistance for LMR plants thus may become difficult and costly. Furthermore, the thin walls are an asset to the operation of the plant due to the excellent heat transfer properties of the sodium coolant. Conventional design for seismic resistance generally relies on provision of adequate stiffness, structural strength and ductility to cope with the seismic loads. For LMR plants, this approach is not only be costly, but may be very difficult to implement. When earthquake forces are involved, wall thickness of the large thin-walled reactor vessels, especially in areas of discontinuities (such as knuckle joints), may be of the order of several inches. Since the base material for these vessels is solid stainless steel, costs,(both material and fabrication), may go up quite rapidly. Also, it becomes quite difficult to provide sufficient seismic resistance and still maintain the inherent simplicity of these low operating pressure systems. The use of thicker walls, additional bracing, stiffeners, etc., not only complicate the design, but may in fact decrease overall safety of the system due to the added complexity of the design.

In about 1980, Argonne learned that France was seriously examining the potential use of seismic isolation for the SuperPhenix 2 reactor plant. This was part of a program to achieve large decreases of commodities in plants. SuperPhenix 2, at that time, was being designed to be a reactor plant marketable worldwide. It was required, therefore to meet considerably higher seismic standards than those which existed for SuperPhenix 1. The goal of SuperPhenix 2 was

to cut costs by as much as 40 to 50% (from SuperPhenix 1) while also increasing seismic resistance. The French mounted a strong effort in applying the seismic isolation technology which they had developed for use in locating their standard PWR plant in areas of higher seismicity. Initial studies by the French showed that not only horizontal seismic isolation was beneficial and feasible, but, in addition, vertical seismic isolation of at least the reactor block was not only feasible, but could in fact simplify considerably reactor vessel internal structures and equipment.

After the discussions with the French, Argonne proposed to DOE to initiate a modest study to explore seismic isolation for the U.S.LMR plant designs. This proposal was accepted by DOE - Nuclear Energy Programs -- and work began immediately. This first phase of the Argonne program was to establish engineering feasibility for providing base horizontal seismic isolation for at least of the nuclear island. The model for this engineering study was taken from the French use of seismic isolation at their PWR plant in Koeberg South Africa plant. The initial Argonne study was performed by Professor A. Ikonomou of the University of Patras Greece under the technical direction of the ANL Engineering Mechanics Program. (Dr.Ikonomou holds several patents in seismic isolation, primarily for devices employing teflon sliding bearings.) Dr.Ikonomou performed a complete scoping study of using seismic isolation in a typical 1000 megawatt electrical LMR plant. In this study the entire nuclear island (approximately 400 ft. square) was isolated. The study showed conclusively that direct extrapolation of the French experience was entirely feasible. The study further showed that reductions in seismic loads within the reactor plant containment and associated buildings located on the common foundation were very significant. Input loads were cut by as much as a factor of 4 or 5 over those associated with a non-isolated plant.

Following the Ikonomou study, Argonne (and, in a separate effort; for DOE/EPRI Burns and Roe, Inc.) performed studies to determine the state of technology development in seismic isolation, particularly with regard to its use in nuclear reactor facilities. Included in this proposed study was elastomeric bearings, sliding bearings, and other types of energy absorption devices and systems.

VI. Seismic isolation program for the DOE new Production Reactors(NPR)

Over a two to three year period (1988-1991) an accelerated and intensive program was carried out on the use of seismic isolation for DOE's New Production Reactors (NPR). While standardization was a primary driving force for using base isolation LMR nuclear power plants, the major incentive for considering base isolation for the NPR was the need to assure continuity of operation of this important and unique facility. A major seismic design requirement for the NPR was that the facility be returned to full production following a major seismic event with minimum downtime and at a reasonable cost. This made the use of seismic isolation a definite design alternative to standard seismic hardening.

The NPR seismic isolation program built upon the foundation of the on going work for LMR plants. During this 2-3 year period work was intensified along all fronts -- development of design and analytical tools; materials testing, bearing tests (both individual and on shake table test of scale models), technical procurement specifications, etc,. While neither the gas cooled NPR (MHTGR) nor the heavy water reactor NPR (HWRF) chose seismic isolation for their reference design, both designs teams recommended that the seismic isolation R&D program be continued. As a result of this funding support (by DOE-NPR) a wealth of analytical, testing, fabrication and design experience has been accumulated and is available for use in all types of critical facilities. Many tests were made on individual bearings and on scale model shake table tests (in particular for soft-soil conditions). Considerable improvement in procurement specifications has been achieved, along with large improvements in developing reliable processes to assure good bonding between rubber and steel layers in the bearings.

VII. Other Programs

In addition to the numerous DOE-funded programs in base seismic isolation, Argonne has also pursued a joint Japanese/U.S. program between Shimizu Corporation and Argonne on response of seismically isolated buildings to actual earthquakes.

In carrying out many of these programs, Argonne has worked closely with other DOE contractors (General Electric and the Energy Technology and Engineering Center-ETEC) and with leading architect -engineering firms such as Burns and Roe and Bechtel. In addition, the staff and testing facilities at the Earthquake Engineering Research Center (EERC) at the University of California at Berkeley have been deeply involved in these programs.

It is interesting to not to note that in the course of carrying out this rather diverse series of programs, rubber-steel laminated bearings have been procured from the following organizations;

*FURON Corp.,	Athens, Texas USA
*LTV Corp.,	Arlington, Texas USA
*Rubbers Consultants	United Kingdom
*Bridgestone Rubber Company	Japan

VIII. Discussion of Selected Portions of the ANL Seismic Isolation Program

The following sample items are presented here only to be illustrative as to level of detail, scope, etc., of some of the program elements.

A. Development of Computer Codes

A three-dimensional finite element code is under development for analyzing the laminated elastomer bearings that are being considered for use in seismic isolation systems for many critical facilities. The code can be used to optimize the design of isolators with respect to the thickness of the rubber and steel layers, the bearing mounting method, and aspect ratio. It can also be used to identify needed experiments to validate the bearing design to provide realistic force-displacement relations for use in global modeling of reactor plants. New elements, nonlinear material laws, and solution algorithms are being incorporated into the NEPTUNE code developed by ANL(35) to provide proper representation of the laminated elastomer bearing subjected to various seismic loadings. The code can be used to predict transient stress and strain within the elastomer, and to determine the ultimate strength of the elastomer and its safety margins.

In addition, a Seismic Isolation Evaluation Code (SISEC)(33) has been developed for analyzing the response of seismically isolated structures. SISEC can be used to study the effects of basemat flexibility, spatial variation of ground motion, effects of long-period excitation, and the vertical response of the structure. It can also be used to study ground settlement effects, overturning and liftoff effects, effects of seismic input at different elevations, effects of torsion, and effects of permanent displacements.

B. Experimental Test Program

1. Bearing Tests

An experimental test program has been developed to establish the viability of using laminated elastomer bearings for base isolation in nuclear facilities. The goal of the program is to determine the performance characteristics of laminated seismic isolators under a wide range of loadings. Tests were performed on scale-size laminated isolators and on small elastomer specimens. Tests were performed in the design range to determine the response of the bearing under expected earthquake loading conditions and beyond the design range to determine failure modes and to establish safety margins.

The bearings tested consisted of a solid cylinder composed of alternate layers of elastomer and thin steel shims. Thick plates cap the top and bottom ends of the bearing. The method of connecting the isolation bearing to the structures was investigated. Two types of connection were considered: dowel- and bolted-type. In the dowel-type connection, mounting plates containing dowel pins were bolted to the structure and the foundation. The dowel pins fit inside dowel holes located in the end plates of the bearing. This design tends to minimize the occurrence of tension in the elastomer, but allows the bearing to roll out under large displacements. In the bolted-type connection, the mounting plates were bolted to the bearing, the structure and the foundation. Tensile stresses can develop in certain regions of the bearing, but the bearing cannot roll out. Under large horizontal displacements the bearing may exhibit buckling-type instabilities.

Two very important points have become clear as a result of the extensive test program. First, it is extremely important to secure a highly reliable bond between the steel and rubber layers in the bearing matrix. Secondly, having assumed such bond quality exists, it is clear that very large margins to failure are inherent in this type of seismic isolation system. With design shear strain levels of 75 to 100%, most bearings were able to withstand shear strains (up to failure) of 250-350%. Some bearings, obviously of advanced design and quality control were able to resist over 600% shear strain before failure. (Even then, failure always occurred in the rubber layers not in the bond.)

2. Elastomer Specimen Tests

Testing of elastomeric specimens was performed to determine basic material properties: elastic constants, viscoelastic constants, damage parameters. The tests were conducted at strain ranges up to failure and at frequencies relevant to a seismic events. Shear tests on standard three-bar lap elastomer specimens were conducted using the INSTRON 8500 series testing machines. The computer program is written in Quick Basic language to control the INSTRON. The program was developed to allow sinusoidal loading and allows the user to specify the frequency, percent strain and number of load cycles as well as test identification data.

Three-bar lap shear specimens were subjected to a series of sinusoidal displacements cycles. The test results show that the shear modulus decreases and the damping increases during cycling.

The results of the bearing tests and the elastomer specimen tests have made sizeable contributions to the data base for reliable bearing design and validation of computer models.

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FIG. 1
ORGANIZATION FOR ANL SEISMIC
ISOLATION PROGRAM

FIG. 2 MAJOR EVENTS IN THE U.S. LMR SEISMIC ISOLATION PROGRAM

1979	AGBABIAN STUDY FOR DOE (1979)
1979	KENNEDY/RICHTER/VON RIESEMAN PAPER SMIRT 5 (1979)
1980	SEVERAL PERSONS/ORG. STUDY FEASIBILITY OF SEIS. ISOL.
1980	ANL, BURNS AND ROE, AGBABIAN
1981	MEETING BETWEEN IKONOMOU AND ANL IN PARIS & CHICAGO
1981	DISCUSS DESIGN OF SEIS. ISOL. FOR 400'X400' NUCL. ISLAND
1982	IKONOMOU PERFORMS STUDY OF SEISMIC ISOLATION FOR ANL
1982	BURNS AND ROE PERFORM SCOPING STUDY ON SEIS. ISOL.
1983	ANL STUDY ON SEIS. ISOL. FOR LARGE LMR PLANT
1983	AS PART OF JOINT STUDY WITH ROCKWELL-INTERNATIONAL
1984	ANL PERFORM SCOPING STUDY OF ISOL. FOR DOE
1984	BURNS AND ROE STUDY SEIS. ISOL. FOR LSPB PLANT (DOE)
1985	IKONOMOU ISSUES REPORT FOR ANL ON SEIS. ISOL.
1985	FOR 100 MWe LMR PLANT
1986	GE AND ROCKWELL-INTERNATIONAL STUDY SEIS. ISOL. FOR
1986	PRISM AND SAFR, RESPECTIVELY
1987	DOE ISSUES SEISMIC TECHNOLOGY PLAN FOR LMR PLANTS
1987	SEIS. ISOL. IS PROM. CANDIDATE FOR LMR SEISMIC DESIGN
1988	ANL/SHIMIZU JOINT STUDY BEGINS (INSITU EQ MEASUREMENTS)
1988	PRISM SELECTED FOR ALMR (USES SEIS. ISOL.)
1989	NRC REVIEWS PRISM DESIGN WITH SEIS. ISOL.
1989	ANL DEVELOPS SISEC CODE TO EVALUATE SEIS. ISOL. SYSTEMS
1990	ANL LEADS STUDY FOR SEISMIC ISOLATION ON THE NPR PLANTS
1991	GE PRISM DESIGN IS ALMR DESIGN-SEISMIC ISOLATION IS USED
1991	ANL COMPLETES NPR STUDY INCL. EXTENSIVE TESTS BY EERC AND ETEC

ANLSIPRO.DWG

FIG. 3 ANL SEISMIC ISOLATION PROGRAM

Summary of Experimental Tests of Elastomeric Seismic Isolation Bearings for Use in Nuclear Reactor Plants

by

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Abstract

This paper describes an experimental test program for isolator bearings which was developed to help establish the viability of using laminated elastomer bearings for base isolation of nuclear reactor plants. The goal of the test program is to determine the performance characteristics of laminated seismic isolation bearings under a wide range of loadings. Tests were performed on scale-size laminated seismic isolators both within the design shear strain range to determine the response of the bearing under expected earthquake loading conditions, and beyond the design range to determine failure modes and to establish safety margins. The test program was funded by the U.S. Department of Energy (DOE) and the National Science Foundation (NSF).

Three types of bearings, each procured from a different manufacturer, have been tested: (1) high shape factor-high damping-high shear modulus bearings; (2) medium shape factor-high damping-high shear modulus bearings; and (3) medium shape factor-high damping-low shear modulus bearings. These bearings were designed by ANL and made according to ANL specifications. All of these tests described in this report were performed at the Earthquake Engineering Research Center (EERC) at the University of California, Berkeley, with technical assistance from ANL. In addition, tests were performed by ETEC on two of the type (1) bearings cited above. Those tests will be reported by others at this workshop.

The tests performed on the three types of bearings have confirmed the high performance characteristics of the high damping-high and low shear modulus elastomeric bearings. The bearings have shown that they are capable of having extremely large shear strains before failure occurs. The most common failure mechanism was the debonding of the top steel plate from the isolators. This failure mechanism can be virtually eliminated by improved manufacturing quality control.

The most important result of the failure test of the isolators is the fact that bearings can sustain large horizontal displacement, several times larger than the design value, without failure. The results of these test programs should give the designer confidence that base isolated structures

can be designed and built with more than adequate safety margins. Their performance in moderate and strong earthquakes will be far superior to conventional structures.

I. Introduction and Scope

This paper presents an overview of the results of a number of tests of elastomeric seismic isolation bearings conducted over the last three years. These tests were conducted as part of the U.S. Department of Energy (DOE) program to develop seismic isolation technology for use in nuclear facilities, and also as part of a joint U.S./Japanese program to study seismic isolation. The joint program was conducted by Argonne National Laboratory (ANL) and the Shimizu Corporation of Japan. The ANL work on this joint program was funded by the National Science Foundation (NSF).

The goals of these tests are to determine the performance characteristics of isolator bearings under a wide range of loadings, to determine failure modes and to establish safety margins. During the course of conducting these tests, three types of elastomeric bearings were tested. The results of these tests are presented along with conclusions relating to bearing design and manufacture, acceptance testing and margins of safety against failure.

II. Background

ANL has been deeply involved in the development of seismic isolation for use in nuclear facilities for the past decade. Under funding and direction of DOE, ANL has participated in the development of the methodology needed to design and evaluate usefulness and effectiveness of seismic isolation for advanced liquid metal-cooled reactors (LMRs). As part of that program, tests were conducted on elastomeric bearings for DOE by the Earthquake Engineering Research Center (EERC) at the University of California, Berkeley, California and at the Energy Technology and Engineering Center (ETEC) near Los Angeles, California. The results of those tests are reported in other papers at this workshop and elsewhere.

More recently, ANL had a major role in evaluating the use of seismic isolation for the DOE New Production Reactors (NPR), and also worked with Shimizu Corporation on the joint U.S./Japanese program to study seismic isolation of a full size test building facility in Sendai, Japan.

As part of these programs, ANL procured over fifty elastomeric isolation bearings for use both in the Sendai test building facility and for individual bearing tests. The results of these individual bearing tests are summarized in this paper.

Three types of high-damping rubber bearings, each procured from a different manufacturer, have been tested: (1) high shape factor-high shear modulus bearings; (2) medium shape factor-high shear modulus bearings; and (3) medium shape factor-low shear modulus

bearings. The bearings were designed by ANL and made according to ANL specifications. All tests described in this report were performed at the EERC with technical oversight by ANL. The tests at EERC were conducted on either the Single Bearing Test Machine, or the Large Scale Test Machine. Descriptions of these test facilities are given in Refs. (1) and (2). The bearing configurations, test plans, and test results are described in the following sections. This material is based on detailed test reports prepared by EERC.

III. High Shape Factor-High Shear Modulus Bearings

A. Background Information

The high shape factor-high shear modulus bearings are made of a high damping-high shear modulus rubber compound. They have a high shape factor value of 24. The bearings were designed and constructed as "dowelled" bearings to avoid tension in the rubber.

ANL purchased sixteen (16) bearings for the joint ANL/Shimizu program. Eight bearings were shipped to Japan for installation at the Sendai test facility and for performance of laboratory tests at Shimizu Corporation; four bearings were sent to EERC for testing to determine their static and dynamic characteristics; two bearings were sent to ETEC for dynamic tests; and two bearings were retained by ANL for archival purposes.

B. Description of Bearings

Figure 1 shows the configuration of the Sendai bearing. The overall diameter of the bearing, including a 3/4 inch protective cover layer, is 20 inches. The bearing has 1 inch thick end plates located at its top and bottom with four drilled holes in the top plate used for dowel pins.

C. Test Plan

The ANL test program at EERC was specifically designed to replicate insitu field tests conducted in Japan on the full size building after the bearings were installed. The bearings were tested in the Large Scale Bearing Test Machine at EERC.

The Sendai seismically-isolated test building weighs a total of 255.4 metric tons and is supported by six bearings. It is estimated that the four corner bearings carry a load of 37.8 metric tons each, and the two middle bearings carry 52.3 tons each. These loads correspond to 83.2 kips and 115.1 kips, respectively.

The four bearings were placed in the test machine with the vertical load set to 83.2 kips. Each horizontal displacement cycle ($\pm 3.15\%$, $\pm 6.25\%$, $\pm 12.5\%$, $\pm 25\%$, $\pm 50\%$, $+75\%$ - 50% shear strain) was repeated three times and data collected for all three cycles. The vertical load was increased to 115.1 kips and the test sequence repeated. The bearings were then deformed to

displacement cycles of $\pm 100\%$ and $\pm 125\%$ at the larger vertical load. It was observed that one bearing was showing signs of distress at this point. This bearing and its partner on that side of the test machine were immobilized by a strong-back, and the testing was continued with displacement cycles corresponding to $\pm 150\%$, $\pm 200\%$ shear strain. No further evidence of damage was visible, and it was concluded that the two tested bearings performed satisfactorily to $\pm 200\%$ strain. (The distressed bearing was cut in half after the test and it was found that the bond between the rubber and the top end plate had failed.)

D. Test Results

Test results are given in detail in ANL/Shimizu report, ANL-002 [Ref. 1]. Figures in this section are taken directly from Ref. 1. Major test results are force-displacement loops and damping values. The force-displacement (hysteresis) loops were recorded for the bearings at the two vertical loads, 83.2 kips and 115.1 kips. The results for the 115.1 kip vertical load is shown in Fig. 2. Figure 3 shows shear modulus as a function of strain obtained from the third cycle of the bearing tests and from the 0.75 Hz coupon tests provided by the elastomer compound supplier. The damping values obtained from the bearing and coupon tests are given in Fig. 4.

As can be seen from Fig. 3, at 50% shear strain the shear modulus of the rubber is about 120 psi. With a building weight of 255.4 metric tons, the bearing's frequency at 50% shear strain is 0.79 Hz. This is higher than the design frequency of 0.75 Hz. In fact, the frequency of 0.75 Hz would not be achieved for the Sendai bearings until the shear strain reached around 100%.

E. Conclusions

The results indicate that the bearings are somewhat stiffer than anticipated in the design. The system frequency at 50% strain is 0.79 Hz. For small displacements generated by minor earthquakes, the frequency will be even higher. For example, at around 10% strain, the response will be of the order of 1.2 Hz. The damping in the bearings is also consistently high. Over the range of strains, from 3% to 50%, the average damping is 15.5%. Thus, for small to moderate earthquakes, structures using these high-shape factor bearings will not experience significant reductions in horizontal seismic loads.

The bearing that showed early signs of distress experienced a bond failure that occurred at the top end plate. While observations of the type of failure show that the failure is not sudden, they also highlighted the difficulty of detecting a poor bond in bearings. The testing procedure of 50% shear strain (at 100% of design vertical compressive load) as an acceptance test did not reveal the presence of a poor bond. The test procedure was modified for later procurements to require the test to subject the bearings to large horizontal displacements while the vertical load on the bearing is reduced to essentially zero, thus eliminating the frictional effect which may mask defects in the bonding.

IV. Medium Shape Factor-High Shear Modulus Bearings

A. Background Information

The motivation to design and test medium (or moderate) shape factor bearings include a desire to achieve a bearing design that facilitates fabrication to high quality standards and has the highly predictable behavior needed for nuclear applications. Also, this particular test program was very important to the DOE's seismic isolation program, because it provided data on a range of shape factors lying between the earlier bearings of GE's PRISM and Rockwell International's SAFR reactors tested at EERC, which were high and low shape factor bearings, respectively.

Four types of rubber bearings were manufactured. The bearings were designed as "bolted"-type bearings (i.e., the bearing is able to develop tensile stresses in the rubber during horizontal deformations). The four types of isolators have two different rubber compounds and two different shape factors, 9 and 18.

Each type of bearing was subjected to seven different types of tests, making a total of 28 bearings tested. Each type of isolator was subjected to five different failure tests in which combinations of the axial and the shear loads were varied in order to obtain an insight into effects of axial-shear load interaction. Fatigue tests were carried out at two different shear strain levels: one at 100% and the other at 150%.

B. Description of Bearings

The four types of bearings used in this test series were designated as types 1, 2, 5 and 6. The dimensions and overall arrangement of bearing type 1 is shown in Fig. 5. (Details of the other bearing types are similar.) A summary of the dimensions and properties of the four types of bearings is given in Table I. As shown in Table I, two different rubber compounds were used (designated here as A and B), and two different shape factors were used, 9 and 18. The overall height of the bearings varied slightly to maintain the overall rubber thickness at 3.75 inches.

The compound used in types 1 and 2 bearings, designated A, has a nominal shear modulus of 120 psi at 100% shear strain; the compound used in types 5 and 6 bearings, designated as B, has a shear modulus of 150 psi at 100% shear strain. The nominal design vertical load for the bearings was 62.5 tons. These isolators were designed to provide a horizontal frequency of 0.50 Hz at 100% shear strain.

C. Test Plan

The seven tests performed on each type of bearing were fatigue and combination failure tests. The purpose of the fatigue tests was to study (1) the resistance of the isolators to a long time duration of cyclic dynamic loads, (2) the loss of stiffness under cyclic loading, and

(3) the failure mechanisms. The purpose of the combination failure tests was to gain insight into an axial-shear load interactions.

There were five combination shear-axial load failure tests performed on each type of bearing for a total of 20 tests. The five tests are: (1) pure compression failure, (2) shear failure with a compressive design vertical load, (3) shear failure with no axial load, (4) shear failure with a tensile design vertical load, and (5) pure tension failure.

The original test program for the fatigue tests was to test each type of bearing to a cyclic displacement corresponding to 50% and 100% shear strains until failure while carrying the design load of 62.5 tons (125 kips). However, after the first fatigue test was performed, bearing type 5 resisted 2880 cycles without a sign of failure. A decision was made to modify the test program to shear strain levels of 100% and 150% and only bearing types 1 and 5 were tested. Thus, there were a total of five fatigue tests.

D. Test Results

Test results important to establish dynamic characteristics and failure mechanisms of the bearings are summarized below.

1. Fatigue Tests

During the five tests, only the two bearings subjected to cycles at 150% shear strain failed. One bearing tested in fatigue at 150% shear strain sustained up to 1120 cycles before failing. Another bearing sustained 515 cycles due to a fissure caused by the expulsion of an air bubble entrapped in the bearing. Figures 6 and 7 are photographs of the two bearings after failure. (Note that Fig. 7 shows only the lower portion of the failed bearing after removal from the test machine.)

The shape factor has no apparent influence on the mechanical behavior of the isolators when subjected to cyclic loading. Similarly, the type of rubber compound had no apparent influence on the mechanical behavior of the isolators when subjected to a fatigue test.

2. Combination Failure Tests

The combination failure tests gave a variety of results which depended to a large extent on the quality of the bond between the steel plates and shims and the rubber layers. For example, some of the isolators carrying high compressive load and tested in shear did not fail, but when the axial load was removed and tested again in shear, they did fail. This suggests that weak bonding was the cause of the failure.

In general, for tests of shear failure with compressive vertical load, the maximum shear strain reached by the isolators surpassed 250% shear strain, and all tests surpassed 175% shear strain.

As expected, bearings with higher shape factor resisted higher compression loads than those with low shape factor bearings, since bearings with higher shape factor require more layers and consequently more steel shims which resist most of the vertical load. The compression tests gave consistent results.

It was possible to test all bearings to failure during the test of shear failure with no axial load. This test was useful in determining the quality of the bonding between the steel shims and plates with the rubber layers.

The results of the tension failure tests were varied, with the tensile strength ranging from 111 psi up to 586 psi. The variability in tensile strength is due to variability in bond strength between the steel plates and shims and rubber layers.

E. Conclusions

These bearings performed well during the fatigue tests. The bearings tested in fatigue at 150% shear strain sustained up to 1120 cycles before failing (equivalent to 30 minutes of very severe ground shaking). These results are more than satisfactory when considering that no earthquake will generate so many cycles all with large displacements. For example, an earthquake of two (2) minutes duration (which is very high) would have about 200 cycles, only a fraction of which cycles would be of the higher displacement motion.

The compression tests revealed that the mechanism of failure is yielding and tearing of the reinforcing steel plates. The plates are loaded by surface shear stresses as they act to prevent barrelling of the bearing under the vertical load. It was demonstrated that the elastomer and the rubber steel bond are able to sustain shear stresses that are large enough to yield the steel in a type of all around tension. The vertical pressure on the bearing, at which the steel reinforcing plates failed, depended on the shape factor but was in every case many times larger than the design vertical pressure.

The program has shown that adequate bond can be achieved if correct quality assurance procedures are followed during the molding and vulcanizing of the bearings. It has also been shown that bonding cannot be determined by tests with a horizontal shear strain of 100% under the presence of any vertical compressive load. One must test under zero axial load or under a slight tension load (to minimize pseudo-beneficial effects of friction).

V. Medium Shape Factor-Low Shear Modulus Bearings

A. Background Information

After experiencing 37 minor to moderate earthquakes in 1-1/2 years, ANL and Shimizu Corporation decided that the high shape factor-high damping isolation bearings installed in the Sendai test facility should be replaced by a set of medium shape factor-low shear modulus

- high damping isolation bearings. This was done to achieve improved response of the building over a wide range of small to moderate earthquakes.

A new compound had been developed which had a shear modulus about half that of the shear modulus of the high damping rubber used in the original test bearings. In July 1990, ANL purchased ten bearings using the newly developed compound. Eight bearings were shipped to Japan in October 1990: six for installation in the Sendai test building and two for laboratory tests in Japan. The two remaining were sent to EERC for laboratory testing.

B. Bearing Configuration

These bearings are characterized as having medium shape factor-high damping-low shear modulus. These bearings have a diameter of 14.75 inches and are 8.1 inches in height. They have 12 layers of 0.394 inch thick rubber and 11 steel shims which are 0.128 inches thick. Details of these bearings are described elsewhere at this workshop (see paper by R. F. Kulak, "Technical Specifications for the Successful Fabrication of Laminated Seismic Isolation Bearings"). With a nominal vertical load of 92.6 kips corresponding to 621 psi vertical pressure, these bearings provide a horizontal frequency of 0.40 Hz at 100% shear strain.

C. Test Plan

The test plan consists of three types of tests: (1) horizontal tests, (2) vertical tests, and (3) failure tests. All tests were carried out on the Single Bearing Test Machine at the Earthquake Simulator Laboratory of EERC.

In the horizontal tests, the bearing was subjected to two sequences of horizontal displacement cycles. Each sequence included three cycles of displacement of the strain levels:

Sequence 1: $\pm 5\%$, $\pm 10\%$, $\pm 25\%$, $\pm 50\%$, $\pm 75\%$, $\pm 100\%$

Sequence 2: $\pm 125\%$, $\pm 150\%$, $\pm 175\%$, $\pm 200\%$

In the vertical tests, the bearing was subjected to five cycles of vertical loading centered around 500, 625, 1000, and 1500 psi of vertical pressure. The variation in the vertical load in each of these tests corresponded to about ± 100 psi. This particular test series is close to the limits of the test system capability at EERC.

The horizontal tests in Sequence 1 were carried out at frequencies of 0.1 Hz and 0.5 Hz. The horizontal tests in Sequence 2 and the vertical tests were carried out only at a frequency of 0.1 Hz.

After the completion of the horizontal and vertical tests a failure test was performed. In the failure test, the bearing was loaded monotonically in shear at a rate of 2.5 in/min.

D. Test Results

Test results are discussed in detail in Ref. [2]. Tables II gives the effective stiffness and equivalent viscous damping for tests performed at a frequency of 0.5 Hz. The force displacement plot for Sequence 1 tests at frequency of 0.5 Hz with 1000 psi vertical pressure is shown in Fig. 8.

Table III gives the stiffness values of the bearing for five cycles around 500 psi, 621 psi, 1000 psi, and 1500 psi. The variation of pressures is ± 100 psi. The hysteresis loops are shown in Fig. 9 where they are superimposed on the monotonic loading curve. As can be seen from Table III, the stiffness value varies with the pressure level around which the cycles are centered. The stiffness values are higher at high pressure levels. Figure 9 shows that the slopes of the hysteresis loops due to pressure variation are much higher than the tangent stiffness of the monotonic loading curve.

In the failure test the bearing was loaded monotonically at a rate of 2.5 in/min. In all three trials, the bearing did not fail when the actuator reached its maximum displacement. Figures 10 and 11 show the deformation of the bearing at shear strain of 200% and 300%, respectively. The maximum shear strain obtained in the test was 415%, with a maximum shear stress of 355 psi. This is equivalent to a maximum displacement of 19.6 inches with a maximum shear load of 52.9 kips.

E. Conclusions

The results of horizontal shear tests show that the shape of hysteresis loops change with the magnitude of vertical pressure loads. The loops change from narrow-elongated to wide-elliptical as vertical pressure increases. The results of vertical tests show that the bearing stiffness varies with the pressure level around which the cycles are centered and they are much higher than the tangent stiffness of the monotonic loading curve. The most important results of the failure test is the fact that the bearing was able to sustain very high horizontal shear displacements while under the design vertical load.

V. Summary

The three test programs summarized in this paper have confirmed the high performance characteristics of the high damping-high and low shear modulus elastomeric bearings. The tests have shown that these bearings are capable of experiencing extremely large shear strains (several times the design shear strain level) before failure occurs. Failure mechanisms of these bearings are strongly influenced by the quality of the bond between the steel plates and shims and rubber layers. Bearings tested in fatigue at 150% shear strain sustained up to 1120 cycles before failing. This is roughly equivalent to about 30 minutes of extremely strong ground shaking. These results are more than satisfactory when considering that no earthquake will generate so many cycles with such large displacements.

VI. Acknowledgments

The authors wish to thank Professor James M. Kelly, Dr. Ian D. Aiken, and Messrs. Wes Neighbors, Donald Clyde, Edgardo Quiroz, and Ivo Van Asten of the Earthquake Engineering Research Center (EERC) for performing and analyzing the bearing tests and preparing test reports. These tests were performed as part of work in the Engineering Mechanics Program of the Reactor Engineering Division of Argonne National Laboratory under the auspices of the U.S. Department of Energy, Contract No. W-31-109-Eng-38. The funding of the experimental tests was partially provided by the National Science Foundation under NSF Agreement No. CES-8800871.

VII. References

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Table I. Dimensions and Properties of Medium Shape Factor Bearings

MEDIUM SHAPE FACTOR BEARING TEST BEARINGS DIMENSIONS AND COMPOUNDS							
TOTAL RUBBER THICKNESS: 3.75" SINGLE STEEL SHIM THICKNESS: 1/16"							
BEARING TYPE	STEEL SHIM			RUBBER		SHAPE FACTOR	COMPOUND TYPE
	DIAMETER	AREA (IN ²)	LAYERS	THICKNESS	LAYERS		
1	14"	153.94	9	3/8"	10	9	A
2	14"	153.94	19	3/16"	20	18	A
5	12"	113.10	11	5/16"	12	9	B
6	12"	113.10	23	5/32"	24	18	B

Table II. Equivalent Stiffness and Viscous Damping of Low Shear Modulus Bearing for Tests Performed at 0.5 Hz Frequency

HORIZONTAL SHEAR TEST FREQUENCY: 0.5 Hz									
PRESSURE ¹ PSI	FILENAME (5-100%)	DYNAMIC PROPERTIES	SHEAR STRAIN						
			5%	10%	25%	50%	75%	100%	
0	910325.10	K _{eff} (Kips in) β (%) G (psi)	3.64 13.69 115	3.14 11.72 100	2.61 9.66 83	2.21 8.82 70	1.97 8.27 62	1.83 8.02 58	
500	910325.11	K _{eff} (Kips in) β (%) G (psi)	4.39 18.35 139	3.37 17.84 107	2.45 16.29 78	1.93 14.57 61	1.66 13.75 53	1.53 13.25 48	
621	910325.12	K _{eff} (Kips in) β (%) G (psi)	4.69 18.31 149	3.47 20.71 110	2.41 19.27 76	1.85 17.39 59	1.58 16.17 50	1.46 15.23 46	
1000	910325.13	K _{eff} (Kips in) β (%) G (psi)	5.39 17.81 171	3.82 23.03 121	2.29 27.77 72	1.56 27.49 49	1.28 26.68 41	1.16 25.54 37	
1500	910325.14	K _{eff} (Kips in) β (%) G (psi)	6.35 15.51 201	4.44 21.65 141	2.49 31.04 79	1.52 36.96 48	1.1 42.39 35	0.92 42.52 29	

¹These pressures are the target pressures, actual values are indicated in the plots.

Table III. Stiffness of Low Shear Modulus Bearing at Various Pressure Loadings

SENDAI - BEARING TEST COMPRESSION TEST					
FILENAME	PRESSURE ¹ PSI	ΔP PSI	STIFFNESS (Kips in)		f_v HERTZ
			1ST CYCLE	5TH CYCLE	
910325.15	500	± 100	1114.11	1114.12	12.10
910404.04	500	± 100	916.88	914.16	10.99
910325.16	621	± 100	1151.32	1149.26	10.84
910404.05	621	± 100	1059.80	1054.51	10.56
910325.17	1000	± 100	1697.81	1657.7	10.47
910404.06	1000	± 100	1621.35	1588.43	10.21
910325.18	1500	± 100	3878.29	3820.09	13.03
910404.07	1500	± 100	2545.18	2635.64	10.56

¹These pressures are the target pressures, actual values are indicated in the plots.

Fig. 1. Design Detail for Sendai Bearing

static vertical load = 115.31 kips/bearing

Fig. 2. Force-Displacement Loops for Bearing, Vertical Load = 115.1 kips

ANL-SENDAI BEARING TESTS

SHEAR MODULUS vs. SHEAR STRAIN
Bearing (3rd Cycle) and Coupon Tests

Fig. 3. Shear Modulus vs. Shear Strain for Bearing and Coupon Tests

ANL-SENDAL BEARING TESTS

DAMPING FACTOR vs. SHEAR STRAIN

Bearing and Coupon Tests

Fig. 4. Damping Factor vs. Shear Strain for Bearing and Coupon Tests

ARGONNE NATIONAL LABORATORY
 DATE: February 22, 1990
 DWG. NO.: NPR-RAS-001-B

Fig. 5. Design Details for Test Bearing Type 1

Fig. 6. Bearing Type 1.07 - Fatigue Test at 150% Shear Strain -
Test No. 1

Fig. 7. Bearing Type 5.07 - Fatigue Test at 150% Shear Strain -
Test No. 2

SENDAY-DEARING TEST
 SKEAR DIAGNOSTIC TEST 5% - 100% STRAIN

Strain	Stiffness	Shoar Modulus	Damping
5%	5.39 Kips/in	171 psi	17.81 %
10%	3.82	121	23.03
25%	2.29	72	27.77
50%	1.56	49	27.49
75%	1.28	41	26.68
100%	1.16	37	25.54

FILENAME: 910325.13
 VERT PRESSURE: 1000 PSI
 FREQUENCY: 0.5 HERTZ

Fig. 8. Force Displacement Plots for Sequence I - 1000 psi Vertical Pressure at Frequency 0.5 Hz

SENDAY -BEARING TEST
COMPRESSION TEST

Filename: 910404.03

Max.Verb.Load: 232.36 Kipn

Fig. 9. Superimposition of Cyclic Vertical Tests on Monotonic Vertical Compression Test - for Low Shear Modulus Bearings

Fig. 10. Bearing Appearance at 200% Strain

Fig. 11. Bearing Appearance at 300% Strain

Isolator System Design Problem #2

The Martin Luther King Jr./Charles R. Drew Medical Center

for

WORKSHOP ON SEISMIC BASE ISOLATION FOR DEPARTMENT OF ENERGY FACILITIES

presentation by:

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1.0 INTRODUCTION:

Base Isolation is a strategy for reducing the effects of earthquakes on buildings. Any one of a number of mechanisms may be used both to uncouple the building from the horizontal components of the earthquake ground motion, and simultaneously to support the vertical weight of the structure. While many Base Isolation systems have been proposed over the years, none has been considered sufficiently practical to be implemented. With the development of multilayer elastomeric bearings, application of the concept has become a practical possibility.

Considerable research, supported by the National Science Foundation and the Malaysian Rubber Producers Research Association, has been carried out on Base Isolation at the Earthquake Simulator Laboratory of the Earthquake Engineering Research Center at the University of California, Berkeley, on the 20 ft. shaking table at that facility. This research has established that Base Isolation of structures is technically feasible.

The bearings for use in aseismic isolation systems for buildings are a development of elastomeric bridge bearings. The vertical stiffness of the bearings is high and the horizontal stiffness low. Many years of experience with bridge bearings have shown that they are equivalently as strong and reliable as the reinforced concrete components in bridges. Recognition of the engineering qualities of elastomeric bearings has led to their application in numerous aseismic Base Isolation systems built abroad and in the United States.

At the request of the Design Team for The Martin Luther King Medical Center, Base Isolation Consultants (BIC) has performed a preliminary assessment of the possible application of Base Isolation for the proposed Martin Luther King/Charles R. Drew Medical Center. The assessment focuses on the feasibility of Base Isolation in consideration of the preliminary configuration developed by the architects, Langdon Wilson (LW), and Structural Engineers, John Martin and Associates (JMA), the geotectonic setting, the costs associated with both initial construction costs and life-cycle costs, and the impact on construction schedule. Site specific Response Spectra were developed by LeRoy Crandall and Associates (LCA).

2.0 FEASIBILITY:

BIC'S feasibility study focuses on the geotectonic setting of the referenced facility. It can be seen from TABLE A that the Newport-Inglewood Fault poses the highest risk for the referenced facility. Although the San Andreas Fault could potentially produce a larger earthquake, it would not be felt as strongly due to its proximity (45 miles) to the hospital. The Newport-Inglewood Fault is only 2 miles from the referenced site, thereby capable of producing higher ground accelerations.

The Maximum Probable Earthquake is defined by Title 24 of the California Code of Regulations as the 500 year return earthquake. The Maximum Credible Earthquake with a less likely probability of occurrence has a return period of approximately 2500 years, and usually is representative of a larger event. LCA's Response Spectra for the Newport-Inglewood events show little difference between the Maximum Probable and Maximum Credible earthquakes, indicating that the risk associated with the larger event is high for the referenced site.

SUMMARY OF EARTHQUAKE RISKS

Fault	Distance (mi)	Maximum Earthquake Magnitudes		Maximum Ground Acceleration	
		Probable	Credible	Probable	Credible
Newport Inglewood	2.0	6.8	7.0	.48g	.50g
San Andreas	45.0	-	8.3	-	.16g

TABLE A

The most damaging aspects of an earthquake, besides surface faulting, are those associated

with acceleration, duration and the frequency content of the earthquake as it relates to the structure. The main purpose of Base Isolation is to de-couple the building from the damaging frequencies of the earthquake, so that reduced force transmission is a by product of this reconfiguration. There is no indication of any possibility of surface faulting at the referenced site.

LW's drawings for the proposed addition indicate an almost square (160'x184') building with 5 stories and a basement. Total area planned adds up to 176,640 s.f. with each floor at 29,440 s.f.

Initially the Design Team had chosen a steel moment frame detailed in accordance with ductile moment resisting space frame requirements of Title 24 as the lateral resisting elements for the hospital. For the purpose of the feasibility analysis, we reconfigured the structure to a Base Isolated structure with a stiff moment resisting space frame superstructure. Since at the isolation interface the structure will be relatively flexible, the desire would be to keep the super structure relatively rigid to prevent upper mode response at the top portion of the structure, thereby taking full advantage of the features of a Base Isolated system. A steel braced frame is the most economical method of achieving this rigidity, but was not evaluated at the time because of its incompatibility with the exterior architectural effect of the building, and the required circulation at the first level.

We estimate the predominate site period to be approximately .6 seconds. With a predicted peak ground acceleration of .48g from the Newport-Inglewood Fault, one can then calculate the difference in response between a fixed based code moment frame and a Base Isolated frame configured with a predominate period of around 2.0 seconds. An additional feature of Base Isolated systems is the inherent damping in most isolation systems, which can be between 15% and 20% (equivalent viscous damping). We have summarized in TABLE B the seismic performance of the two types of structures. We have based this comparison on dynamic analyses we have performed using the SAP80 [1] program. The building is modeled as 5 or 6 lumped masses connected by one column, the horizontal stiffness of which is mainly in flexure. One of the purposes of the model is to examine the effects of stiffening the code moment frame for the purpose of adapting it to the Base Isolation scheme.

DYNAMIC ANALYSES RESULTS
BASED ON .48G GROUND ACCELERATION

Structure Type	Period (sec)	Displ. @Roof@Base (in) (in)		Base Shear (g)	Accel. @Roof (g)
Fixed Base Code Frame (5% damped)	1.04	11.4	00.0	0.62	1.22
Base-Isolated Code Frame (10% damped)	2.37	13.8	10.6	0.23	0.27
Base-Isolated Stiffened Frame (10% damped)	2.30	12.6	10.6	0.22	0.25

TABLE B

Our analysis is based on a response spectra analysis using Response Spectra provided by LCA. The fixed-base structure with a period of 1.04 seconds will undergo displacements in excess of 11 inches, which means that interstory drift will be more than 2 inches. Such large displacements indicate that the steel in the frames will be stretched beyond the yield point with permanent deformations taking place. Additionally these large excursions would result in significant damage to the architectural, mechanical, electrical elements and contents of the facility. With the high degree of force transmission at the top part of the structure (1.22g @ roof), personnel would be at risk.

The feasibility of Base Isolation as a strategy for earthquake protection becomes apparent for sites similar to the referenced facility, where structures are located near faults which can produce high accelerations. With high frequency content expected from these sources, Base Isolated structures would perform better than fixed-based structures.

The advantage in using an isolation system under the referenced structure is that the displacements the building will experience during an earthquake will occur mainly at the isolation interface and not in the steel frames. The isolators can be made elastic, and are capable of restoring the building to its original position, whereas the deformations in a fixed-base code frame would become permanent. Interstory drift also becomes minimal, with significantly reduced damage to the architectural and mechanical elements within the building.

In the Base-Isolated design, the contents and occupants within the facility are protected, since the transmission of ground acceleration is filtered by the isolator as opposed to amplified by a steel frame. The fixed-base code frame, having a period of 1.04 seconds, can experience for the Maximum Credible event base shear of .62g and an amplification of almost 2.0 to produce an acceleration of 1.22g at the top of structure. If subjected to the same ground acceleration, the stiffened Base-Isolated design, with a fundamental period of 2.30 seconds, similarly can experience a base shear of .22g, almost 1/3 that of the fixed-base design, with practically no amplification of forces up the structure; therefore, at the top level, the isolated structure would experience only 1/5 (1.22g/.25g) of the force levels to which the conventionally designed steel frames would be subjected.

3.0 CONSTRUCTION COSTS:

We have developed schematic costs for the referenced facility based on 176,640 S.F. of construction. Cost opinions are stated for both a conventional fixed-base design and an isolated design. Based on our experience and recent studies BIC performed for Kaiser Permanente of Northern California, we have proportioned the building elements for a code-designed hospital as follows:

Architectural	\$78/S.F.	26%
Mechanical & Electrical	\$87/S.F.	29%
Structural	\$54/S.F.	18%
Contents	\$81/S.F.	27%
<hr/>		
Total	\$300/S.F.	100%

TABLE C

The basis of the structural cost is a fixed-base steel moment-resisting space frame, detailed for ductile requirements.

ADDITIONAL CONSTRUCTION COSTS
ASSOCIATED WITH BASE ISOLATION

	<u>FIXED-BASE</u> <u>DESIGN</u>	<u>ISOLATED</u> <u>DESIGN</u>	
A. Building Costs			
1. Architectural	\$13,177,900	\$13,507,300	+2.5%
2. Structural	\$ 9,538,500	\$10,183,500	+6.8%
3. Mechanical	\$11,489,200	\$11,604,000	+1.0%
4. Electrical	\$ 3,868,400	\$ 3,868,400	0%
5. Conveying Sys.	\$ 600,000	\$ 630,000	+5.0%
Total Building Costs	\$38,674,000	\$39,793,200	+2.9%
Contingency (10%)	\$ 3,867,400	\$ 3,979,320	
<hr/>			
Total	\$42,541,400	\$43,772,520	+2.9%
Additional for Isolation:		<u>(+\$1,231,120)</u>	

TABLE D

The additional structural costs associated with the isolation scheme involve the addition of a crawl space under the basement slab and the costs of isolators and testing. This crawl space can be used as a pipe basement. Stiffening the moment frame is also additional. The outer retaining wall can be of tilt-up concrete construction with tie-backs for the sake of economy, reducing the cost associated with a cast-in-place concrete retaining wall. The overall estimated structural increase of 6.8% (\$645,000) associated with the isolation scheme was arrived at as follows:

ADDITIONAL STRUCTURAL COSTS

1. Creating a crawl space	\$ +300,000
2. Isolation Bearings	\$ +350,000
3. Prototype Bearings & Testing	\$ +35,000
4. Stiffening of frames	\$ +100,000
5. Change retaining wall to tilt-up constr.	\$ -140,000

Total	\$ +645,000
-------	-------------

TABLE E

Other additional building costs were extrapolated from construction cost data from the Foothill project [2]. The additional 2.9% increase of construction cost for Base Isolation is based on the building components being installed under one construction contract. If the project is phased over time and portions of the architectural, mechanical and electrical infill is constructed at a later date, the escalation factor associated with costlier construction becomes more dominant than the additional construction cost associated with current implementation of Base Isolation.

4.0 COSTS ASSOCIATED w/EARTHQUAKE DAMAGE

A damage cost comparison is shown in TABLE F. We have made these calculations associated with estimating earthquake damage at the referenced project based on the Navy's methodology as described in a paper entitled "Economics of Seismic Design for New Buildings", ASCE Journal of Structural Engineering, Vol. 110, No. 12, December 1984, by John Ferritto [3].

The Navy's methodology focuses on earthquake damage associated with two primary structural response characteristics - interstory drift and floor accelerations.

Earthquake damages is calculated based on a formula-

$$[\text{Element Damage} = \text{Cost} \times \text{Repair Multiplier} \times \text{Damage Ratio}]$$

Element represents that portion of the building -

architectural, mechanical and electrical, structural and contents.

Repair Multiplier represents the additional costs associated

with replacement, and comes from GSA information.

Damage Ratio for different drift conditions and floor
acceleration characteristics associated with various building
elements is shown on Tables 1 and 2 [3].

Based on \$/S.F. values in TABLE C, we proportioned the costs of the building elements. Total damage then is the summation of the element damages associated with both drift and acceleration effects. Damage ratios associated with acceleration has been divided by 3 to account for hospital construction, which, being reviewed by OSA and OSHPD, is of a higher structural quality than the Navy's examples.

A summary of the earthquake performance of the 2 structural schemes as they relate to project site's predicted seismicity and associated estimated damage based on Navy's methodology [3] is shown on TABLE F.

DAMAGE ESTIMATE COMPARISONS

PERIOD (SEC)	MAX.CREDIBLE E.Q.(.48g) DISPL. DRIFT/BASE ACCEL. FLOOR SHEAR @ TOP	SUM OF D.R. DAMAGE AS DAM. RATIO X OF BLDG. \$/s.f. COST
FIXED BASE		
MOMENT FRAME 1.04	11.4in 2.3in .62g 1.22g	
ARCH. \$78/sf	.49* .30**	.79 \$62/sf
M&E 87	.09 .26	.35 30
STRUCT.54	.03 .15	.18 10
CONT. 81	.09 .37	.46 37
TOTALS\$300/sf		\$139/sf (46%)[\$19,569,600]
ISOLATED-STIFFENED		
MOMENT FRAME 2.30	19.0in 0.2in .37g .38g	
ARCH. \$78/sf	.0* .10**	.10 \$ 8/sf
M&E 87	.0 .08	.08 7
STRUCT.54	.0 .05	.05 3
CONT. 81	.0 .12	.12 10
TOTALS\$300/sf		\$28/sf (09%)[\$3,920,100]

* damage ratios due to drift [3]

**damage ratios due to acc. [3]

TABLE F

What can be seen from TABLE F is that Base Isolation can significantly reduce the cost of earthquake damage at the referenced facility. One can challenge the NAVY'S Methodology used to assess damage but to our knowledge it is the only objective Methodology that is published which is base on building drift and face acceleration, two key components that cause damage in buildings.

An important intangible which has not been factored into the analysis is the issue of the safety of the personnel and patients. With the potential of experiencing over 1.0g of force for the fixed base design, people will be at risk not only from the forces, but also from overturned partitions, ceilings and equipment. With isolation, force transmission for the maximum credible earthquake (the worst scenario) will be within the elastic capacity of the structural system so there will be no permanent deformation in the steel members, and also the elastic capacity of bracing elements so the architectural and mechanical elements along with contents are protected. The security of the building elements along with reduced force transmission translates to higher degree of safety for personnel and patients.

Another intangible issue is that associated with the disruption of the Hospital. If ground accelerations reach .48g as postulated, significant damage will occur to a fixed-base design. The hospital will be in havoc when it is needed most - during the aftermath of an earthquake. In the event the referenced site will experience .48g of acceleration, the damage would be extensive, requiring months to correct. With Base Isolation, damage can be kept to a minimum and the facility can be made operational shortly after the earthquake.

5.0 SCHEDULE:

Normally the project would take 12 months to design, 5 months for OSHPD review and another 26 months to bid, award and construct (A total of 43 months). To allow for additional time required for both OSHPD review and prototype bearing fabrication and testing, the structural isolation package can be delivered early to OSHPD. Upon completion of Design Development (2 months), the emphasis can be placed on the completion of the structural isolation package for OSHPD review (within 6 months). By this process we would gain 4 months on the schedule to accommodate this additional time required by OSHPD. The isolation bearings need to be pre-purchased and supplied to the general contractor for installation. The time required to fabricate and test bearings is 4 months. This can be accomplished during the time the contractor is mobilizing, excavating and installing the foundations.

6.0 IMPLEMENTATION

The Martin Luther King Jr./Charles R. Drew Medical Center will be a 5 level Trauma Care and Diagnostics Imaging Center located in the County of Los Angeles. Because of the critical nature of the Medical Center and because of it is within a seismically active area of Southern California, the decision was made to incorporate Base Isolation as a strategy for earthquake protection.

The isolation system consists of a hybrid system with high damping rubber (HDR) bearings in combination with friction dampers.

In accordance with OSHPD requirements, time history analysis has been performed. Input records are those which were furnished by LeRoy Crandall and Associates (LCA) as described in their report to the County of Los Angeles, Dated 3-13-91 and 7-15-91. The design time histories, as outlined in the LCA reports, have been approved by the California division of Mines and Geology.

7.0 DESCRIPTION OF STRUCTURE

The referenced structure is a 5 story building with a mechanical penthouse located at the upper roof and a full basement. It is a steel structure with braced frames configured at the perimeter to resist lateral loads. Concrete over metal deck is the floor construction. A 2 story concrete wall exists at the perimeter from the basement level to 2nd floor level to create rigid base for the braced frames. Essentially what has been developed is a 2-story concrete box with 3 intersecting concrete diaphragms. The steel braced frames cantilever from this box.

HDR isolators are located below the basement floor. Friction dampers are located at the perimeter to reduce both translational and torsional response of the structure.

8.0 DYNAMIC ANALYSIS

BIC's program N-PAD [4] was used to perform non-linear time history analysis. a total of 24 runs (6 runs in each direction for each of the two events, MCE and MPE) were executed. N-PAD, which was first used successfully for the analysis of the Foothill Communities Law & Justice Center (FCLJC) [2]. Using a "strain-space plasticity" theory, the program represents the state of the art in modelling of isolation devices.

N-PAD is designed to work in conjunction with SAP81 and introduces non-linear degrees of freedom at the base of a 3-D elastic superstructure. The SAP-81 program is used to model the elastic superstructure; the model is then combined with the non-linear isolator base modeled on N-PAD. Before execution of N-PAD, some of the SAP81 programs - SAP, FRAME, PLANE, SOLVE, TABS - need to be executed to establish the stiffness matrix of the superstructure. After completion, N-PAD will generate the complete stiffness, mass, and damping matrix of the base isolation system and combine it with the superstructure matrix. Each isolator is considered as having two translational and one vertical degree of freedom (d.o.f.). The vertical d.o.f. is elastic and independent; the two translational d.o.f.'s are coupled and can be non-linear.

For each time step, the stress and strain of every pad is calculated and the program establishes the assembled stiffness of the pads at the rigid base center. The building is assumed to have a rigid diaphragm at the base and at every floor level and the horizontal d.o.f.'s are reduced to three (two

translational and one rotational) at each story. The stiffness of the superstructure is elastic and generated by TABS. The base system (three horizontal d.o.f.'s) is non-linear and iterations are carried out to calculate the instantaneous stiffness for every time step. Newmark's method is used for time history analysis.

The program has been checked for accuracy by Professor Ed Wilson of the University of California at Berkeley. Recent comparisons between N-PAD results and actual shake table test results show good correlation.

HDR bearings are modelled as bi-linear elastic elements, friction dampers as two dimensional non-linear element, (elastic plastic). Damping in the HDR bearings is assigned as a constant equivalent viscous damping coefficient on a local basis (for every pad). Damping in the friction dampers are taken into consideration as cyclic non-linear hysteretic damping. N-PAD also performs a modal analysis of the superstructure for every time step. 5% damping was assigned to the structure for the MPE runs, 8% for the MCE runs.

9.0 N-PAD ANALYTICAL RESULTS

Horizontal displacements at the base, base shear (%G) and roof acceleration (%G) plots for each of the 24 runs were plotted. A sample set of N-PAD plots along with a computer generated model of the isolated structure are in the Appendix. A summary of these results follows:

SUMMARY OF N-PAD ANALYTICAL RESULTS

<u>MCE</u>				
<u>Record</u>	<u>Dir</u>	<u>Displ.</u>	<u>Base Shear</u>	<u>Roof Acc.</u>
BC230	-Y-	14"	24%	26%
BC140	-Y-	(18")	(30%)	(34%)
LOMAP090	-Y-	14"	24%	29%
LOMAP000	-Y-	17"	29%	33%
TAFT69	-Y-	15"	25%	29%
TAFT21	-Y-	15"	27%	31%
BC230	-X-	14"	25%	27%
BC140	-X-	(18")	(32%)	(34%)
LOMAP090	-X-	14"	24%	28%

LOMAP000	-X-	17"	29%	33%
TAFT69	-X-	15"	25%	28%
TAFT21	-X-	15"	26%	31%

MPE

ARR8140	-Y-	11"	20%	23%
ARR8230	-Y-	11"	20%	24%
ARR10050	-Y-	(13")	(23%)	(25%)
ARR10320	-Y-	12"	21%	25%
CORR90	-Y-	9"	17%	22%
CORR00	-Y-	10"	19%	23%
ARR8140	-X-	11"	20%	23%
ARR8230	-X-	11"	20%	24%
ARR10050	-X-	(13")	(23%)	(25%)
ARR10320	-X-	12"	21%	27%
CORR90	-X-	10"	18%	21%
CORR00	-X-	10"	19%	22%

() Indicates most critical case for each direction.

10.0 REFERENCES

- [1] Wilson & Habibullah, "SAP 80 Structural Analysis Programs"; 1984.
- [2] A. G. Tarics, D. Way, and J. M. Kelly, "The Implementation of Base Isolation for the Foothill Communities Law and Justice Center", A Report to the National Science Foundation (CEE-821758C) 1984.
- [3] J. Ferritto, "Economics of Seismic Design for New Buildings", ASCE Journal of Structural Engineering, Vol. 110, No. 12, December 1984.
- [4] D. Way, V. Jeng, "N-Pad, A Three-Dimensional Program for the Analysis of Base-Isolated Structures," Proceedings of American Society of Civil Engineers, Structural Congress, 1989.

APPENDIX

bic6\llnl.rep

N-PAD Model of Structure

North Frame Vertical Reactions at Bearings/Sliders

Due to MPE

Maximum Credible Earthquake

Time History Record - BC140
scaled 100% in X direction, 30% in Y direction

Maximum Credible Earthquake

Time History Record - BC140
scaled 100% in X direction, 30% in Y direction

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