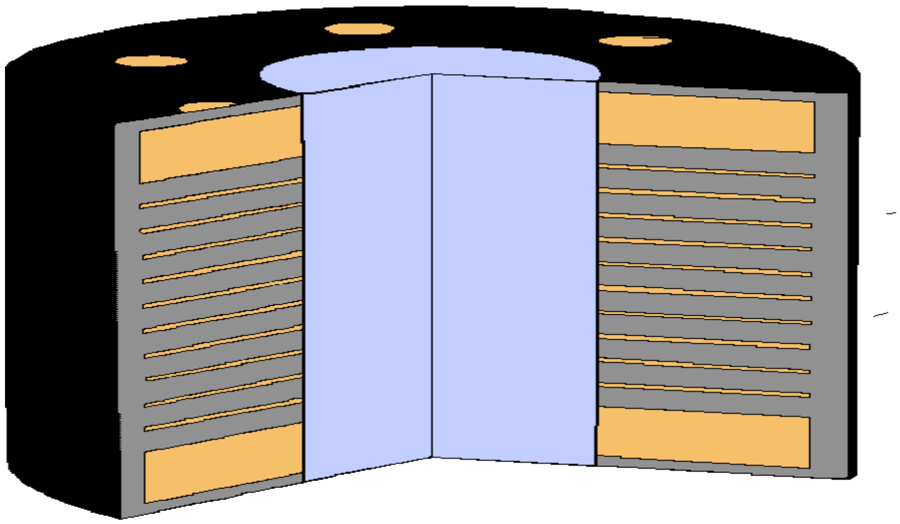
BASE ISOLATION OF STRUCTURES



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DESIGN GUIDELINES

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Holmes Consulting Group	New Zealand (Auckland, Wellington, Christchurch, Queenstown)	Structural Engineering
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2001

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COMPANY CREDENTIALS IN BASE ISOLATION

THE COMPANY

Holmes Consulting Group, part of the Holmes Group, is New Zealand's largest specialist structural engineering company, with over 90 staff in three main offices in NZ plus 25 in the San Francisco, CA, office. Since 1954 the company has designed a wide range of structures in the commercial and industrial fields.

HCG has been progressive in applications of seismic isolation and since its first isolated project, Union House, in 1982, has completed six isolated structures. On these projects HCG provided full structural engineering services. In addition, for the last 8 years we have provided design and analysis services to Skellerup Industries of New Zealand and later Skellerup Oiles Seismic Protection (SOSP), a San Diego based manufacturer of seismic isolation hardware. Isolation hardware which we have used on our projects include Lead-Rubber Bearings (LRBs), High Damping Rubber Bearings (HDR), Teflon on stainless steel sliding bearings, sleeved piles and steel cantilever energy dissipators.

SEISMIC ISOLATION EXPERTISE

The company has developed design and analysis software to ensure effective and economical implementation of seismic isolation for buildings, bridges and industrial equipment. Expertise encompasses the areas of isolation system design, analysis, specifications and evaluation of performance.

System Design

- Special purpose spreadsheets and design programs
- British Standards (BS 5400)
- Uniform Building Code (UBC)
- U.S. Bridge Design (AASHTO)

Analysis Software

- ETABS Linear and nonlinear analysis of buildings
- SAP2000 General purpose linear and nonlinear analysis
- DRAIN-2D Two dimensional nonlinear analysis
- 3D-BASIS Analysis of base isolated buildings
- ANSR-II Three dimensional nonlinear analysis

Specifications

- Codes
- Materials
- Fabrication
- Tolerances
- Material Tests
- Prototype Tests
- Quality Control Tests

Evaluation

- Isolation system stiffness
- Isolation system damping
- Effect of variations on performance

Services Provided

- Design of base isolation systems
- Analysis of isolated structures
- Evaluation of prototype and production test results

PERSONNEL

Trevor Kelly, Technical Director, heads the seismic isolation division of HCG in the Auckland office. He has over 15 years experience in the design and evaluation of seismic isolation systems in the United States, New Zealand and other countries and is a licensed Structural Engineer in California.

PROJECT EXPERIENCE

Project	Isolation System
Structural Engineers of Record	
Union House, New Zealand	Sleeved piles + steel cantilevers
Parliament Buildings Strengthening, New Zealand	Lead rubber + high damping rubber
Museum of New Zealand	Lead rubber bearings + Teflon sliders
Whareroa Boiler, New Zealand	Teflon sliders + steel cantilevers
Bank of New Zealand Arcade	Lead rubber + elastomeric bearings
Maritime Museum	Lead rubber + elastomeric bearings
Completed Projects as Advisers to Skellerup	
Missouri Botanical Garden, MI	High damping rubber bearings
Hutt Valley Hospital, NZ	Lead rubber bearings
3 Mile Slough Bridge, CA	Lead rubber bearings
Road No. 87, Arik Bridge, Israel	Lead rubber bearings
Taiwan Freeway Contracts C347, C358	Lead rubber bearings
St John's Hospital, CA	Lead rubber bearings
Benecia-Martinez Bridge, CA	Lead rubber bearings
Berkeley Civic Center, CA	Lead rubber bearings
Princess Wharf, New Zealand	Lead rubber bearings
Big Tujunga Canyon Bridge, CA	Lead rubber bearings
Plus 26 other projects where we have prepared isolation system design as part of bid document submittals.	

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1 INTRODUCTION

Most structural engineers have at least a little knowledge of what base isolation is – a system of springs installed at the base of a structure to protect against earthquake damage. They know less about the when and why – when to use base isolation and why use it? When it comes to how, they either have too little knowledge or too much knowledge. Conflicting claims from promoters and manufacturers are confusing, contradictory and difficult to fully assess. Then, if a system can be selected from all the choices, there is the final set of *hows* – how to design the system, how to connect it to the structure, how to evaluate its performance and how to specify, test and build it. And, of course, the big *how*, how much does it cost?

These notes attempts to answer these questions, in sufficient detail for our practicing structural engineers, with little prior knowledge of base isolation, to evaluate whether isolation is suitable for their projects; decide what is the best system; design and detail the system; and document the process for construction.

The emphasis here is on design practice. The principles and mathematics of base isolation have been dealt with in detail in textbooks which contain rigorous treatments of the structural dynamics (see the two textbooks listed in the *Bibliography*, by Skinner, Robinson and McVerry [1993] and by Naiem and Kelly [1999]). These notes provide sufficient depth for our engineers to understand how the dynamics effect response but do not provide instructions as to how to solve the non-linear equations of motion governing the system response. As for much else in structural engineering, we have computer programs to do this part of the work for us.

1.1 THE CONCEPT OF BASE ISOLATION

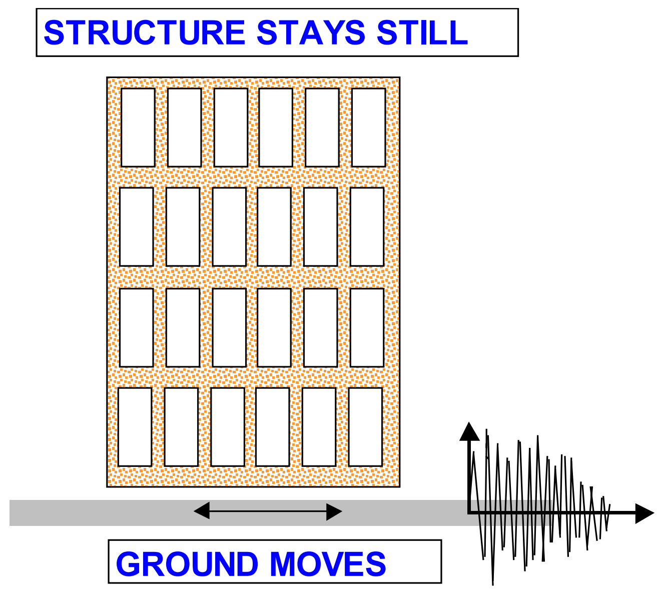
The term base isolation uses the word isolation in its meaning of *the state of being separated* and base as *a part that supports from beneath or serves as a foundation for an object or structure* (Concise Oxford Dictionary). As suggested in the literal sense, the structure (a building, bridge or piece of equipment) is separated from its foundation. The original terminology of *base isolation* is more commonly replaced with *seismic isolation* nowadays, reflecting that in some cases the separation is somewhere above the base – for example, in a bridge the superstructure may be separated from substructure columns. In another sense, the term seismic isolation is more accurate anyway in that the structure is separated from the effects of the seism, or earthquake.

Intuitively, the concept of separating the structure from the ground to avoid earthquake damage is quite simple to grasp. After all, in an earthquake the ground moves and it is this ground movement which causes most of the damage to structures. An airplane flying over an earthquake is not affected. So, the principle is simple. Separate the structure from the ground. The ground will move but the building will not move. As in so many things, the devil is in the detail. The only way a structure can be supported under gravity is to rest on the ground. Isolation conflicts with this fundamental structural engineering requirement. How can the structure be separated from the ground for earthquake loads but still resist gravity?

Ideal separation would be total. Perhaps an air gap, frictionless rollers, a well-oiled sliding surface, sky hooks, magnetic levitation. These all have practical restraints. An air gap would not provide vertical support; a sky-hook needs to hang from something; frictionless rollers, sliders or magnetic levitation would allow the building to move for blocks under a gust of wind.

So far, no one has solved the problems associated with ideal isolation systems and they are unlikely to be solved in the near future. In the meantime, earthquakes are causing damage to structures and their contents, even for well designed buildings. So, these notes do not deal with ideals but rather with practical isolation systems, systems that provide a compromise between attachment to the ground to resist gravity and separation from the ground to resist earthquakes.

FIGURE 1-1 BASE ISOLATION



When we define a new concept, it is often helpful to compare it with known concepts. Seismic isolation is a means of reducing the seismic demand on the structure:

Rolling with the punch is an analogy first used by Arnold [1983?]. A boxer can stand still and take the full force of a punch but a boxer with any sense will roll back so that the power of the punch is dissipated before it reaches its target. A structure without isolation is like the upright boxer, taking the full force of the earthquake; the isolated building rolls back to reduce the impact of the earthquake.

Automobile suspension. A vehicle with no suspension system would transmit shocks from every bump and pothole in the road directly to the occupants. The suspension system has springs and dampers which modify the forces so the occupants feel very little of the motion as the wheels move over an uneven surface. As we'll see later, this is a good analogy as springs and dampers are essential components of any practical isolation system

The party trick with the tablecloth. You've probably seen the party trick where the tablecloth on a fully laden table is pulled out sideways very fast. If it's done right, everything on the table will remain in place and even unstable objects such as full glasses will not overturn. The cloth forms a sliding isolation system so that the motion of the cloth is not transmitted into the objects above.

Base Isolation falls into general category of *Passive Energy Dissipation*, which also includes In-Structure Damping. In-structure damping adds damping devices within the structure to dissipate energy but does not permit base movement. This technique for reducing earthquake demand is covered in separate HCG Design Guidelines. The other category of earthquake demand reduction is termed *Active Control*, where isolation and/or energy dissipation devices are powered to provide optimum performance. This category is the topic of active research but there are no widely available practical systems and our company has no plans to implement this strategy in the short term.

1.2 THE PURPOSE OF BASE ISOLATION

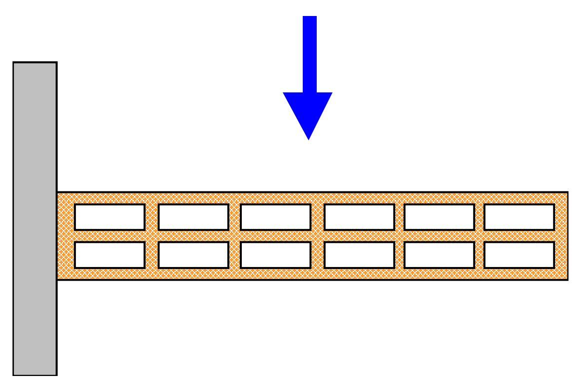
A high proportion of the world is subjected to earthquakes and society expects that structural engineers will design our buildings so that they can survive the effects of these earthquakes. As for all the load cases we encounter in the design process, such as gravity and wind, we work to meet a single basic equation:

CAPACITY > DEMAND

We know that earthquakes happen and are uncontrollable. So, in that sense, we have to accept the demand and make sure that the capacity exceeds it. The earthquake causes inertia forces proportional to the product of the building mass and the earthquake ground accelerations. As the ground accelerations increases, the strength of the building, the capacity, must be increased to avoid structural damage.

It is not practical to continue to increase the strength of the building indefinitely. In high seismic zones the accelerations causing forces in the building may exceed one or even two times the acceleration due to gravity, g. It is easy to visualize the strength needed for this level of load – strength to resist 1g means than the building could resist gravity applied sideways, which means that the building could be tipped on its side and held horizontal without damage.

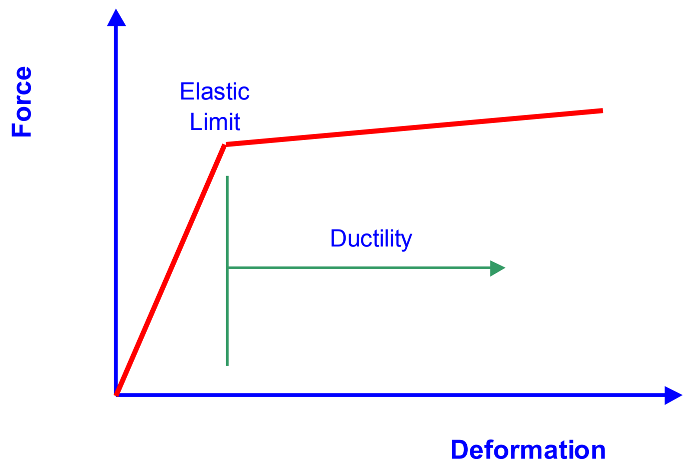
FIGURE 1-2 DESIGN FOR 1G EARTHQUAKE LOADS



Designing for this level of strength is not easy, nor cheap. So most codes allow engineers to use ductility to achieve the capacity. Ductility is a concept of allowing the structural elements to deform beyond their elastic limit in a controlled manner. Beyond this limit, the structural elements soften and the displacements increase with only a small increase in force.

The elastic limit is the load point up to which the effects of loads are non-permanent; that is, when the load is removed the material returns to its initial condition. Once this elastic limit is exceeded changes occur. These changes are permanent and non-reversible when the load is removed. These changes may be dramatic – when concrete exceeds its elastic limit in tension a crack forms – or subtle, such as when the flange of a steel girder yields.

FIGURE 1-3 DUCTILITY



For most structural materials, ductility equals structural damage, in that the effect of both is the same in terms of the definition of damage as that which impairs the usefulness of the object. Ductility will generally cause visible damage. The capacity of a structure to continue to resist loads will be impaired.

A design philosophy focused on capacity leads to a choice of two evils:

1. Continue to increase the elastic strength. This is expensive and for buildings leads to higher floor accelerations. Mitigation of structural damage by further strengthening may cause more damage to the contents than would occur in a building with less strength.

2. Limit the elastic strength and detail for ductility. This approach accepts damage to structural components, which may not be repairable.

Base isolation takes the opposite approach, it attempts to reduce the demand rather than increase the capacity. We cannot control the earthquake itself but we can modify the demand it makes on the structure by preventing the motions being transmitted from the foundation into the structure above.

So, the primary reason to use isolation is to mitigate earthquake effects. Naturally, there is a cost associated with isolation and so it only makes sense to use it when the benefits exceed this cost. And, of course, the cost benefit ratio must be more attractive than that available from alternative measures of providing earthquake resistance.

1.3 A Brief History of Base Isolation

Although the first patents for base isolation were in the 1800's, and examples of base isolation were claimed during the early 1900's (e.g. Tokyo Imperial Hotel) it was the 1970's before base isolation moved into the mainstream of structural engineering. Isolation was used on bridges from the early 1970's and buildings from the late 1970's. Bridges are a more natural candidate for isolation than buildings because they are often built with bearings separating the superstructure from the substructure.

The first bridge applications added energy dissipation to the flexibility already there. The lead rubber bearing (LRB) was invented in the 1970's and this allowed the flexibility and damping to be included in a single unit. About the same time the first applications using rubber bearings for isolation were constructed. However, these had the drawback of little inherent damping and were not rigid enough to resist service loads such as wind.

In the early 1980's developments in rubber technology lead to new rubber compounds which were termed "high damping rubber" (HDR). These compounds produced bearings that had a high stiffness at low shear strains but a reduced stiffness at higher strain levels. On unloading, these bearings formed a hysteresis loop that had a significant amount of damping. The first building and bridge applications in the U.S. in the early 1980's used either LRBs or HDR bearings.

Some early projects used sliding bearings in parallel with LRBs or HDR bearings, typically to support light components such as stairs. Sliding bearings were not used alone as the isolation system because, although they have high levels of damping, they do not have a restoring force. A structure on sliding bearings would likely end up in a different location after an earthquake and continue to dislocate under aftershocks.

The development of the friction pendulum system (FPS) shaped the sliding bearing into a spherical surface, overcoming this major disadvantage of sliding bearings. As the bearing moved laterally it was lifted vertically. This provided a restoring force.

Although many other systems have been promulgated, based on rollers, cables etc., the market for base isolation now is mainly distributed among variations of LRBs, HDR bearings, flat sliding bearings and FPS.

In terms of supply, the LRB is now out of patent and so there are competing suppliers in most parts of the world. Although specific HDR compounds may be protected, a number of manufacturers have proprietary compounds that provide the same general level of performance. The FPS system is patented but there are licensees in most parts of the world.

1.4 THE HOLMES ISOLATION TOOLBOX

The main differences between design of an isolated structure compared to non-isolated structures are that the isolation devices need to be designed and the level of analysis required is usually higher. We have developed tools to assist in each of these areas.

Isolation System Design

The isolator design is governed by a relatively small number of equations and does not require extensive numerical computation and so can be performed using spreadsheet tools. We have template spreadsheets developed for the codes we will normally encounter (UBC, NZS4203 and AASHTO) as listed in Table 1-1. These spreadsheets are described further later but are able to design most commonly used isolators. The performance is estimated based on a single mass approximation, effectively assuming a rigid building above the isolators.

The other spreadsheet listed in Table 1-1, *Bridge*, incorporates the AASHTO bearing design procedures but has the analysis built in. This is because bridge models are generally simpler than building models. The *Bridge* spreadsheet can perform a single mode analysis, including the effects of flexible piers and eccentricity under transverse loads, and also shells to a non-linear time history analysis using a modified version of the Drain-2D computer program. Dynamic analysis results are imported to the spreadsheet for comparison with single mode results.

Isolation System Evaluation

Evaluation of base isolated structures usually requires a dynamic analysis, either response spectrum or time history. Often we do both. For buildings, the ETABS program can be used for both types of analysis provided a linear elastic structure is appropriate. The DUCTILEIN and DUCTILEOUT pre- and post-processors can be used with isolated buildings. If the structure is not suited to ETABS but non-linearity is restricted to the isolation system then SAP2000 has similar capabilities to ETABS and is more suited to general-purpose finite element modeling.

If non-linearity of the structural system needs to be modeled then use the ANSR-L program. This is general purpose and is suited to both buildings and other types of structure. Use the MODELA and PROCESSA pre- and post-processors with this program. This program is also suited to multiple analyses, for example, to examine a large number of options for the isolation system parameters.

3D-BASIS is a special purpose program for analysis of base isolation buildings. It uses the structural model developed for ETABS as a super-element. We do not use this program very often now that ETABS has non-linear isolation elements but it may be more efficient for multiple batched analyses, similar to ANSR-L.

TABLE 1-1 ISOLATION DESIGN AND EVALUATION TOOLS

Name	Type	Purpose
UBCTemplate	.xls	Design isolators to UBC provisions.
NZS4203Template	.xls	As above but modified for NZS4203 seismic loads.
AASHTO Template	.xls	Design isolators to 1999 AASHTO provisions.
BRIDGE	.xls	as above with Analysis Modules
ETABS	.exe	Analysis of linear buildings with non-linear isolators
SAP2000	.exe	Analysis of linear structures with non-linear isolators.
3D-BASIS	.exe	Analysis of linear buildings with non-linear isolators
ANSR-L	.exe	Analysis of non-linear structures with nonlinear isolators.
DUCTILEIN	.xls	Prepare models for ETABS
DUCTILEOUT	.xls	Process results from ETABS
MODELA	.xls	Prepare models for ANSR-L
PROCESSA	.xls	Process results from ANSR-L
ACCEL	.xls	5% Damped spectrum of earthquake records Far fault has 36 records distant from faults Near fault has 10 near fault records Contains procedure for UBC scaling

1.5 Isolation System Suppliers

There are continual changes in the list of isolation system suppliers as new entrants commence supply and existing suppliers extend their product range. The system suppliers listed in Table 1-2 are companies which we have used in our isolation projects, who have supplied to major projects for other engineers or who have qualified in the HITEC program. HITEC is a program operated in the U.S. by the Highway Technology Innovation Center for qualification of isolation and energy dissipation systems for bridges.

Given the changes in the industry, this list may be outdated quickly. You can find out current information on these suppliers from the web and may also identify suppliers not listed below.

The project specifications should ensure that potential suppliers have the quality of product and resources to supply in a timely fashion. This may require a pre-qualification process.

There are a large number of manufacturers of elastomeric bearings worldwide as these bearings are widely used for bridge pads and bearings for non-isolation purposes. These manufacturers may offer to supply isolation systems such as lead-rubber and high damping rubber bearings. However, standard bridge bearings are designed to operate at relatively low strain levels of about 25%. Isolation bearings in high seismic zones may be required to operate at strain levels ten times this level, up to 250%. The manufacturing processes required to achieve this level of performance are much more stringent than for the lower strain levels. In particular, the bonding techniques are critical and the facilities must be of clean-room standard to ensure no contamination of components during assembly. Manufacturers not included in Table 1-2 should be required to provide evidence that their product can achieve the performance levels required of seismic isolators.

Table 1-2 Isolation System Suppliers

Company	Product
Bridgestone (Japan) BTR Andre (UK) Scougal Rubber Corporation (US)	High damping rubber
Robinson Seismic (NZ)	Lead rubber
Earthquake Protection Systems, Inc (US)	Friction pendulum system
Dynamic Isolation Systems, Inc (US) Skellerup Industries (NZ) Seismic Energy Products (US)	Lead rubber, high damping rubber
Hercules Engineering (Australia)	Pot (sliding) bearings
R J Watson, Inc (US) FIP-Energy Absorption Systems (US)	Sliding Bearings
Taylor Devices, Inc. (US) Enidine, Inc. (US)	Viscous Dampers

1.6 Isolation System Durability

Many isolation systems use materials which are not traditionally used in structural engineering, such as natural or synthetic rubber or polytetrafluoroethylene (PTFE, which is used for sliding bearings, usually known as Teflon ©, which is DuPont's trade name for PTFE). An often expressed concern of structural engineers considering the use of isolation is that these components may not have a design life as long as other structural components, usually considered to be 50 years or more.

Natural rubber has been used as an engineering material since the 1840's and some of these early components remained in service for nearly a century in spite of their manufacturers lacking any knowledge of protecting elastomers against degradation. Natural rubber bearings used for applications such as gun mountings from the 1940's remain in service today.

Elastomeric (layered rubber and steel) bearings have been in use for about 40 years for bridges and have proved satisfactory over this period. Shear testing on 37 year old bridge bearings showed an average increase in stiffness of only 7% and also showed that oxidation was restricted to distances from 10 mm to 20 mm from the surface. Since these early bearings were manufactured technology for providing resistance to oxygen and ozone degradation has improved and so it is expected that modern isolation bearings would easily exceed a 50 year design life.

Some early bridge bearings were cold bonded (glued, rather than vulcanized) and these bearings had premature failures, damping the reputation of isolation bearings. The manufacture of all elastomeric bearings isolation bearings is by vulcanization; the steel plates are sand blasted and de-greased, stacked in a mold in parallel with the rubber layers and the assembly is then cured under heat and pressure. Curing may take 24 hours or more for very large isolators.

Some bridge bearings are manufactured from synthetic rubbers, usually neoprene. There are reports that neoprene will stiffen with age to a far greater extent than natural rubber and this material does not appear to have been used for isolation bearings for this reason. If a manufacturer suggests a synthetic elastomer, be sure to request extensive data on the effects of age on the properties.

PTFE was invented in 1938 and has been used extensively for all types of applications since the 1940's. It is virtually inert to all chemicals and is about the best material known to man for corrosion resistance, which is why there is difficulty in etching and bonding it. Given these properties, it should last almost indefinitely. In base isolation applications the PTFE slides on a stainless steel surface under high pressure and velocity and there is some flaking of the PTFE and these flakes are deposited on the stainless steel surface. Eventually the bearing will wear out but indications are that this will occur after travel of between 10 km and 20 km. For buildings this is not a concern as sliding occurs only during earthquake and the total travel is measured in meters rather than kilometers. For bridges the PTFE is often lubricated with silicone grease contained by dimples in the PTFE.

2 PRINCIPLES OF BASE ISOLATION

2.1 FLEXIBILITY – THE PERIOD SHIFT EFFECT

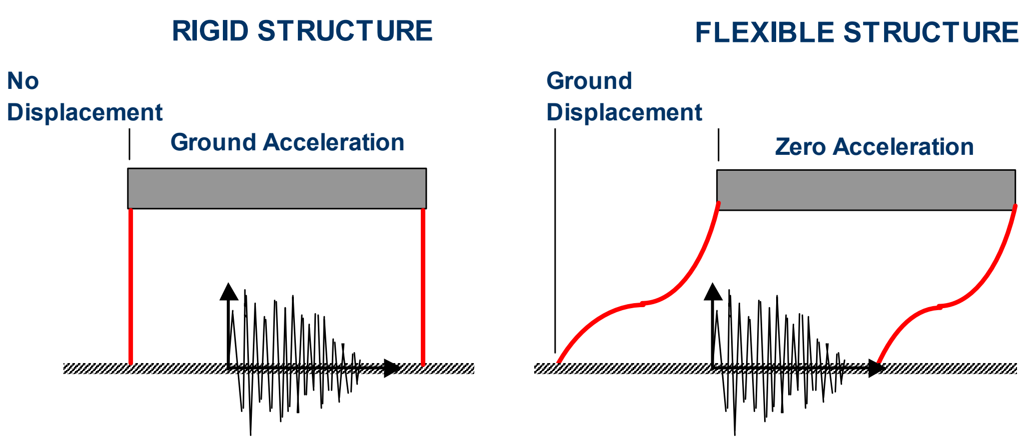
2.1.1 THE PRINCIPLE

The fundamental principle of base isolation is to modify the response of the building so that the ground can move below the building without transmitting these motions into the building. In an ideal system this separation would be total. In the real world, there needs to be some contact between the structure and the ground.

A building that is perfectly rigid will have a zero period. When the ground moves the acceleration induced in the structure will be equal to the ground acceleration and there will be zero relative displacement between the structure and the ground. The structure and ground move the same amount.

A building that is perfectly flexible will have an infinite period. For this type of structure, when the ground beneath the structure moves there will be zero acceleration induced in the structure and the relative displacement between the structure and ground will be equal to the ground displacement. The structure will not move, the ground will.

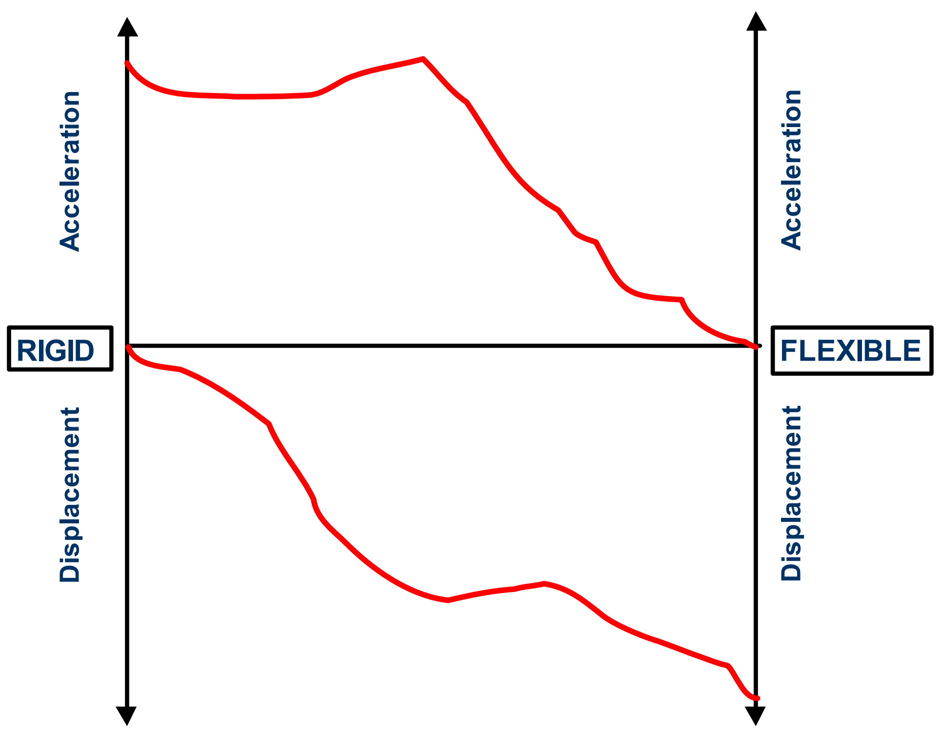
FIGURE 2-1 TRANSMISSION OF GROUND MOTIONS



All real structures are neither perfectly rigid nor perfectly flexible and so the response to ground motions is between these two extremes, as shown in Figure 2-2. For periods between zero and infinity, the maximum accelerations and displacements relative to the ground are a function of the earthquake, as shown conceptually in Figure 2-2.

For most earthquakes there be a range of periods at which the acceleration in the structure will be amplified beyond the maximum ground acceleration. The relative displacements will generally not exceed the peak ground displacement, that is the infinite period displacement, but there are some exceptions to this, particularly for soft soil sites and site which are located close to the fault generating the earthquake.

FIGURE 2-2 STRUCTURE ACCELERATION AND DISPLACEMENT



2.1.2 EARTHQUAKE CHARACTERISTICS

The reduction in acceleration response when the period is lengthened is a result of the characteristics of earthquake motions. Although we generally approach structural design using earthquake accelerations or displacements, it is the velocity that gives the best illustration of the effects of isolation. The energy input from an earthquake is proportional to the velocity squared.

Implementation of base isolation in codes is based on the assumption that over the mid-frequency range, for periods of about 0.5 seconds to 4 seconds, the energy input is a constant, that is, the velocity is constant. Design codes such as the UBC and NZS4203 assume this. For a constant velocity, the displacement is proportional to the period, T, and the acceleration is inversely proportional to T. If the period is doubled, the displacement will double but the acceleration will be halved.

2.1.3 CODE EARTHQUAKE LOADS

The period shift effect can be calculated directly from code specified earthquake loads. Code specifications generally provide a base shear coefficient, C, as a function of period. This coefficient is a representation of the spectral acceleration such that C times the acceleration due to gravity, g, provides an acceleration in units of time/length^2.

Figures 2-3 and 2-4 show the period shift effect on accelerations and displacements. The curves on these figures are for a high seismic zone and are based on the coefficients defined by FEMA-273 and UBC. They show that the period shift effect is most effectively if short period structures (T < 1 second) are isolated to 2 seconds or more.

FIGURE 2-3 PERIOD SHIFT EFFECT ON ACCELERATIONS

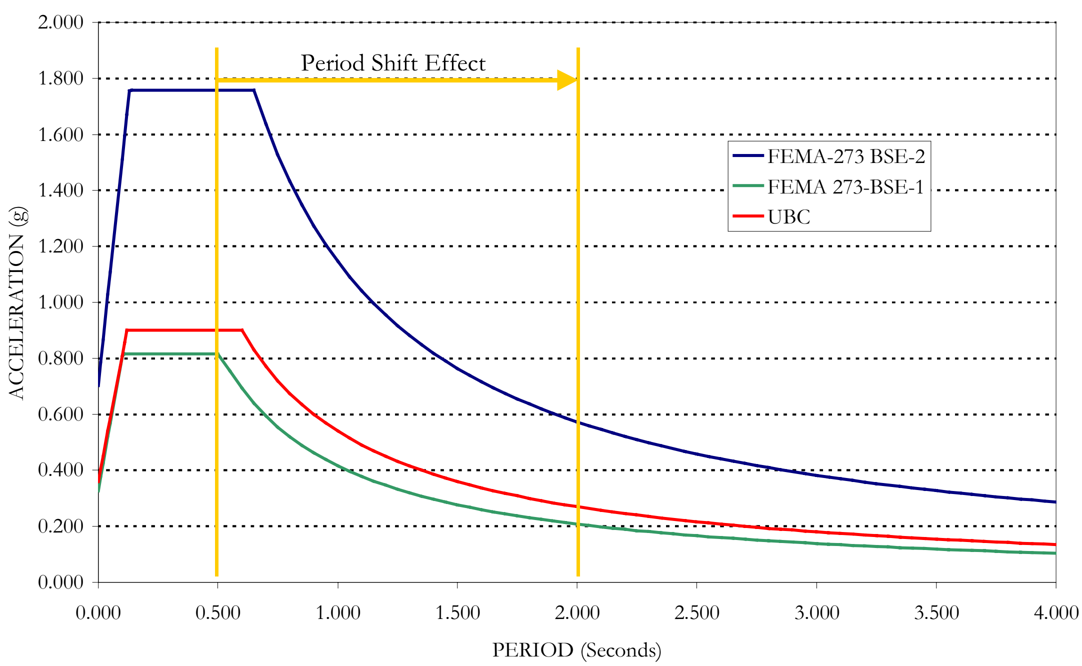
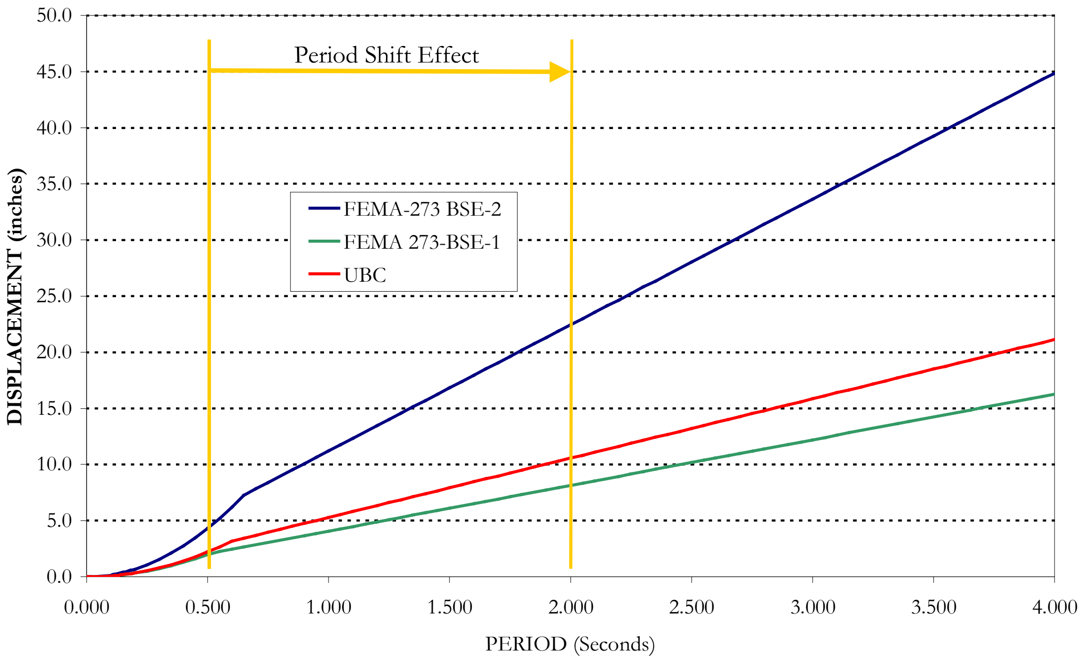


Figure 2-4 Period Shift Effect on Displacement



The displacements in Figure 2-4 are calculated from the code shear coefficient. For any code that specifies a design coefficient, C, the acceleration represented by this coefficient can be converted to a pseudo-spectral displacement, S_d, using the relationship:

$$S_d = \frac{gC}{\omega^2}$$

Where ω is the circular frequency, in radians per cycle. This is related to the period as

$$\omega = \frac{2\pi}{T}$$

From which the displacement, Δ, can be calculated as

$$\Delta = S_d = \frac{gCT^2}{4\pi^2}$$

For most codes, beyond a minimum period up to which the base shear coefficient is a constant, the velocity is assumed constant and the base shear coefficient is inversely proportional to T, that is,

$$C = \frac{C_v}{T}$$

Where C_v is a constant related to factors such as soil type, seismicity, near fault effects etc. In this zone, the product of displacement and shear coefficient is a constant:

$$\Delta C = C_v^2 \left(\frac{g}{4\pi^2} \right)$$

In this equation, C_v is code specific. The constant related to units of length, $g/4\pi^2$, has a numerical value of 248.5 for mm units and 9.788 for inch units.

The fact that this product is constant provides a clear illustration of the trade off between base shear coefficient, C, and the displacement, Δ. If you want to reduce the base shear coefficient by a factor of 2 then the displacements will double. If you want to reduce the coefficient by 4, the displacements will increase by a factor of 4.

As an example, consider a UBC design in Zone 4 with a near fault factors of $N_a = 1.2$ and $N_v = 1.6$, Soil Profile S_B. The seismic coefficients are $C_a = 0.48$ and $C_v = 0.64$. The period beyond which the velocity is assumed constant is calculated as

$$T_s = \frac{C_v}{2.5C_a} = 0.533$$

For periods beyond 0.533 seconds, the product $\Delta C = 248.5 \times 0.64^2 = 101.8$ in mm units (4.01 in inch units).

Table 2-1 shows the relationship between base shear coefficient and displacement for this example. At the transition period, 0.53 seconds, the coefficient is 1.2. If you want to reduce this by a factor of 4, to 0.30, the displacement will be 339 mm (13.4 inches). At this displacement the period can be calculated as $C_v / C = 0.64/0.30 = 2.13$ seconds.

Most codes specify coefficients based on 5% damping and the values in Table 2-1 are based on this. As discussed later, the displacements associated with adding damping to the system can reduce the period shift effect.

Although the example above is based on the UBC, a similar function can be derived for any code that specifies the coefficient as an inverse function of period:

- Calculate the coefficient, C at any period, T, beyond the transition period.
- Calculate the displacement, Δ, at this period as $248.5CT^2$ mm ($9.788CT^2$ inches).
- The product of CΔ can now be used as a constant to calculate the displacement at any other base shear coefficient C_1 as $\Delta_1 = C\Delta/C_1$.

Table 2-1 Base Shear Coefficient versus Displacement

Base Shear Coefficient	Maximum Displacement		Period, T
C	mm	inch	(seconds)
1.20	85	3.3	0.53
1.10	93	3.6	0.58
1.00	102	4.0	0.64
0.90	113	4.5	0.71
0.80	127	5.0	0.80
0.70	145	5.7	0.91
0.60	170	6.7	1.07
0.50	204	8.0	1.28
0.40	254	10.0	1.60
0.35	291	11.5	1.83
0.30	339	13.4	2.13
0.25	407	16.0	2.56
0.20	509	20.0	3.20
0.15	679	26.7	4.27

2.2 Energy Dissipation – Adding Damping

Damping is the characteristic of a structural system that opposes motion and tends to return the system to rest when it is disturbed. Damping arises from a multitude of sources. For isolation systems, damping is generally categorized as viscous (velocity dependent) or hysteretic (displacement dependent). For equivalent linear analysis, hysteretic damping is converted to equivalent viscous damping.

Whereas the period shift effect usually decreases acceleration but increases displacements, damping almost always decreases both accelerations and displacement. A warning here, increased damping reduces accelerations in respect to the base shear, which is dominated by first mode response. However, high damping may increase accelerations in higher modes of the structure. For multi-story buildings, the statement that "the more damping the better" may not hold true.

Response spectra provided in codes are almost invariably for 5% damping. There are several procedures available for modifying spectra for damping ratios other than 5%.

The Eurocode EC8 provides a formula for the acceleration at damping ξ relative to the acceleration at 5% damping as:

$$\Delta_{(T,\xi)} = \Delta_{(t,5)} \sqrt{\frac{7}{2+\xi}}$$

Where ξ is expressed as a percent of critical damping. UBC and AASHTO provide tabulated B coefficients, as listed in Table 2-2. FEMA-273 provides a different factor for short and long periods, as listed in Table 2-4. Generally the factor B_l would apply for all isolated structures. This has the same values as the UBC and AASHTO values in Table 2-2.

TABLE 2-2 UBC AND AASHTO DAMPING COEFFICIENTS

	Equivalent Viscous Damping						
	<2%	5%	10%	20%	30%	40%	>50%
B	0.8	1.0	1.2	1.5	1.7	1.9	2.0

In Table 2-3 the reciprocal of the EC8 value is listed alongside the equivalent factors from FEMA-273. EC8 provides for a greater reduction due to damping than the other codes and seem to relate to the short period values, B_s, from FEMA-273.

TABLE 2-3 FEMA-273 DAMPING COEFFICIENTS

Effective Damping β % of Critical	B_s Periods ≤ T_o	B_l Periods > T_o	EC8
< 2	0.8	0.8	0.75
5	1.0	1.0	1.00
10	1.3	1.2	1.31
20	1.8	1.5	1.77
30	2.3	1.7	2.14
40	2.7	1.9	2.45
> 50	3.0	2.0	2.73

Figures 2-5 and 2-6 plot the effect of damping on isolated accelerations and displacements respectively using UBC / AASHTO values of B. Both quantities are inversely proportional to the damping coefficient, B, and so the damping reduces both by the same relative amount.

Figure 2-5 Effect of Damping on Accelerations

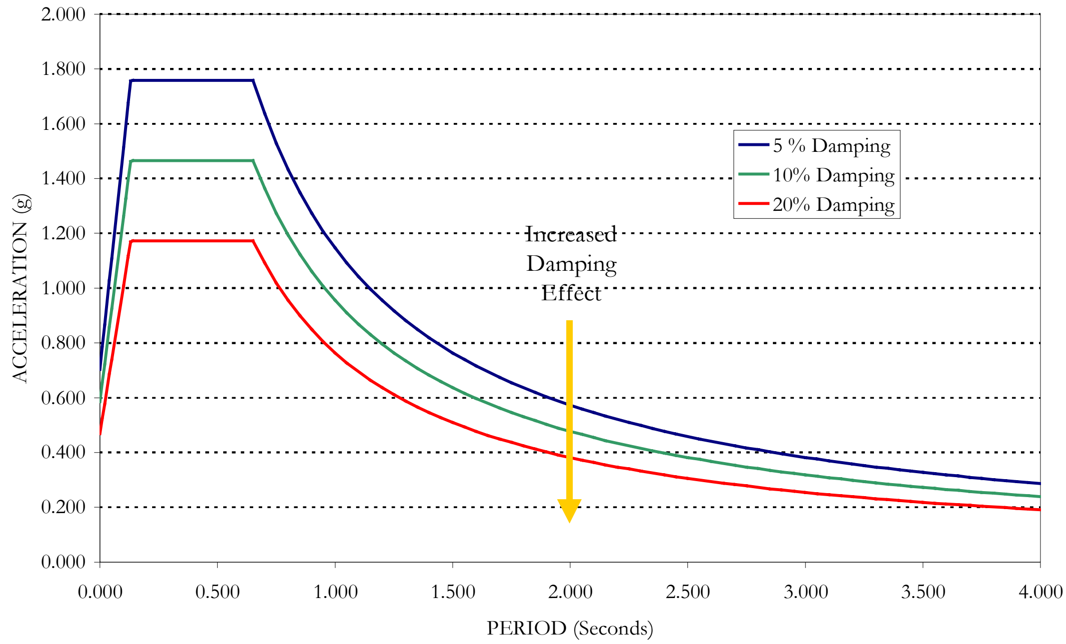
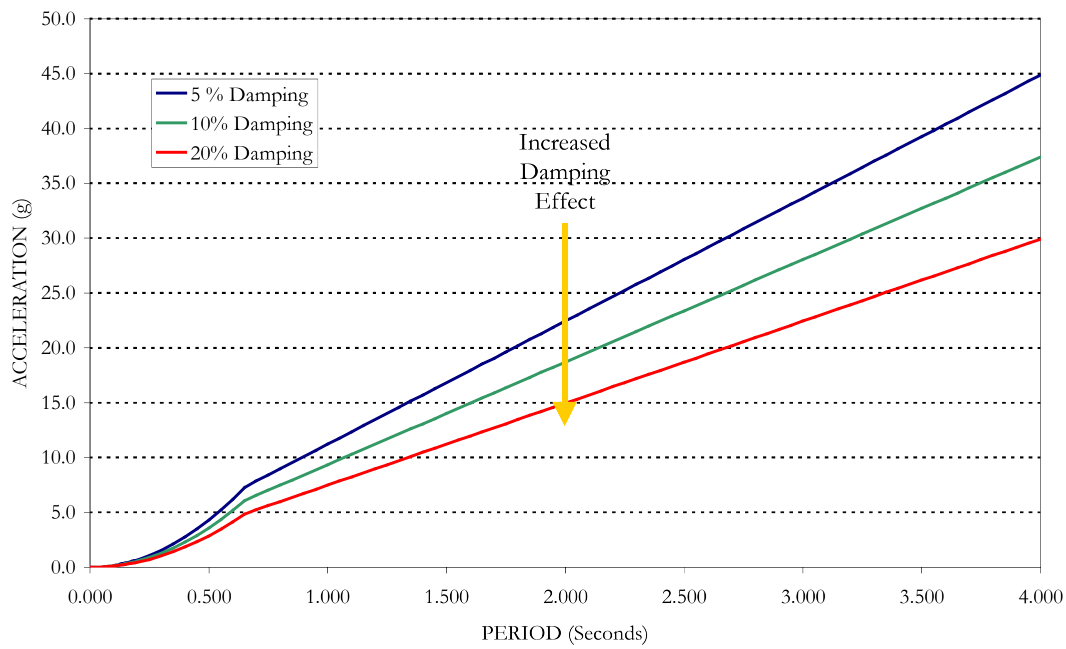


Figure 2-6 Effect of Damping on Displacements



2.2.1 HOW ACCURATE IS THE B FACTOR?

The accuracy of the B factor can be assessed by generating response spectra for various damping ratios and comparing these with the spectra reduced by the B factor. Two records were chosen for this comparison:

1. The N-S component of the 1940 El Centro earthquake. This was one of the first strong motion records of high amplitude recorded and has been the basis for many studies. Figure 2-7 shows the acceleration spectra for this record and Figure 2-8 the equivalent displacement spectra, each for damping ratios from 5% to 40% of critical.

2. The Los Angeles, Sepulveda V.A. Hospital, 360 degree component of the record from the 1994 Northridge earthquake. This record was only 8 km (5 miles) from the epicenter of the 1994 earthquake and so exhibits near fault effects. Figures 2.9 and 2.10 show the respective acceleration and displacement spectra for the same damping values as for the El Centro record.

The near fault Sepulveda record produces a much greater response than the more distance El Centro record, a peak spectral acceleration of 2.8g compared to 0.9g and peak spectral displacement of 550 mm (22") versus 275 mm (11") for 5% damping. The impact of this on isolation design is discussed later.

The actual B factors from these records can be calculated at each period by dividing the spectral acceleration or displacement at a particular damping by the equivalent value at the 5% damping. Figures 2-11 to 2-14 plot these B factors for 10% and 30% damping and compare them with the B values from the UBC and EC.

The first point to note is that the use of a constant B factor is very much an approximation. The actual reduction in response due to viscous damping in a function of both the period and the earthquake record. As the B factor is a single value function, Table 2-4 compares it with the mean values calculated from the spectra for periods from 0.5 to 4 seconds, the period range for isolation systems.

TABLE 2-4 COMPARISON OF DAMPING FACTORS

	10 % Damping		**30% Damping**	
	Acceleration	**Displacement**	**Acceleration**	**Displacement**
Average From Spectra				
El Centro	0.84	0.81	0.65	0.49
Sepulveda	0.83	0.80	0.70	0.47
UBC	0.83	0.83	0.59	0.59
EC	0.76	0.76	0.47	0.47

The UBC B factor for 10% damping is a good representation of the mean for the two examples considered here, for both acceleration and displacement response. For 30% damping the higher damping tends to reduce displacements by a greater proportion than accelerations. For this level, the UBC factor is non-conservative for accelerations and conservative for displacements compared to the mean value. The EC factor tends to overestimate the effects of damping across the board.

Given the uncertainties in the earthquake motions, the UBC B factor appears to be a reasonable approximation but you need to be aware that is just than, an approximation. To get a more accurate response to a particular earthquake you will need to generate damped spectra or run a time history analysis.

FIGURE 2-7 EL CENTRO 1940: ACCELERATION SPECTRA

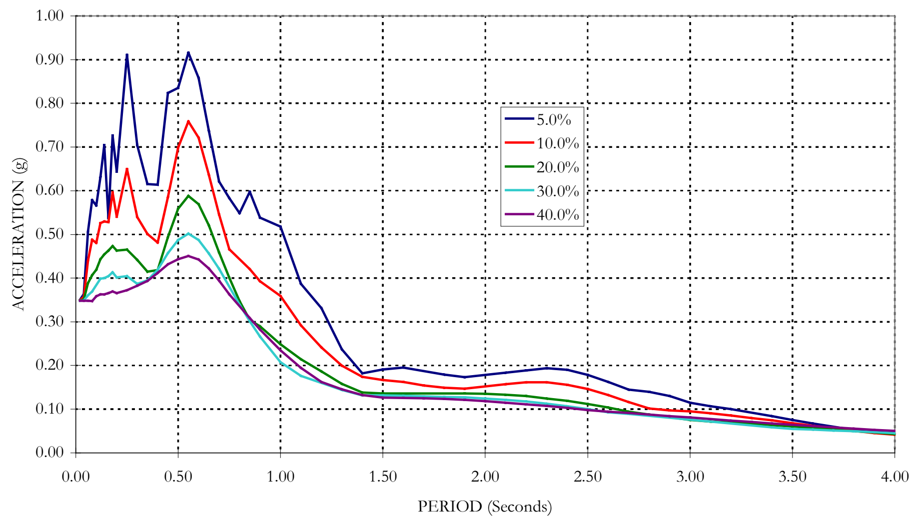


FIGURE 2-8 EL CENTRO 1940 N-S DISPLACEMENT SPECTRA

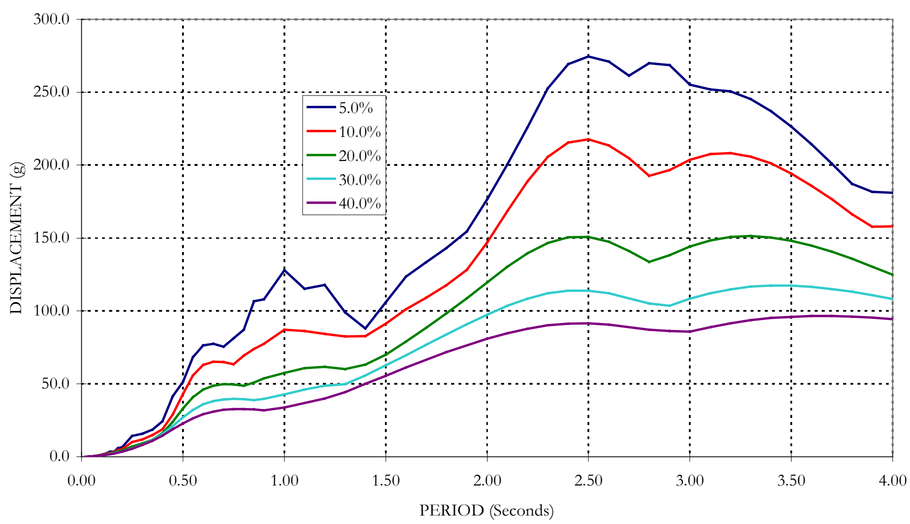


FIGURE 2-9 NORTHRIDGE SEPULVEDA: ACCELERATION SPECTRA

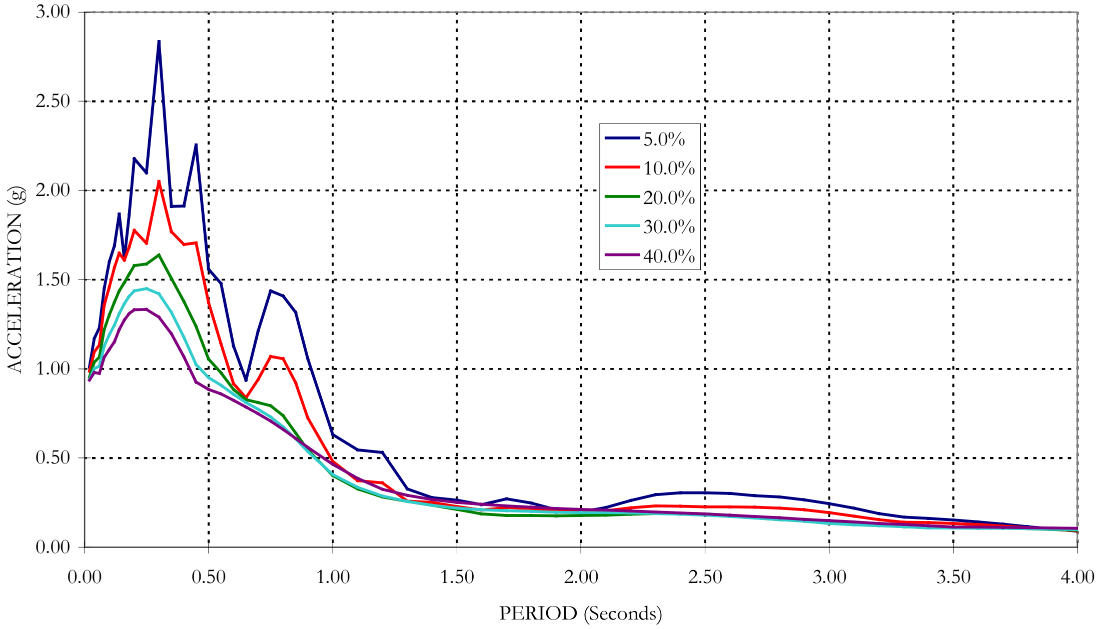


FIGURE 2-10 NORTHRIDGE SEPULVEDA: DISPLACEMENT SPECTRA

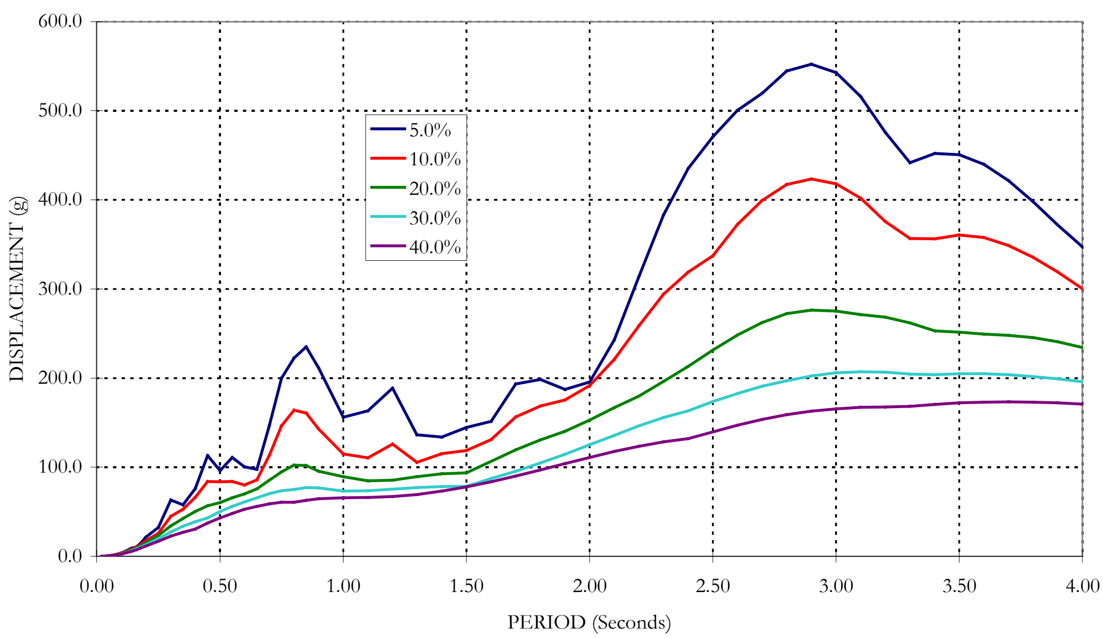


Figure 2-11 B Factor for 10% damping: acceleration

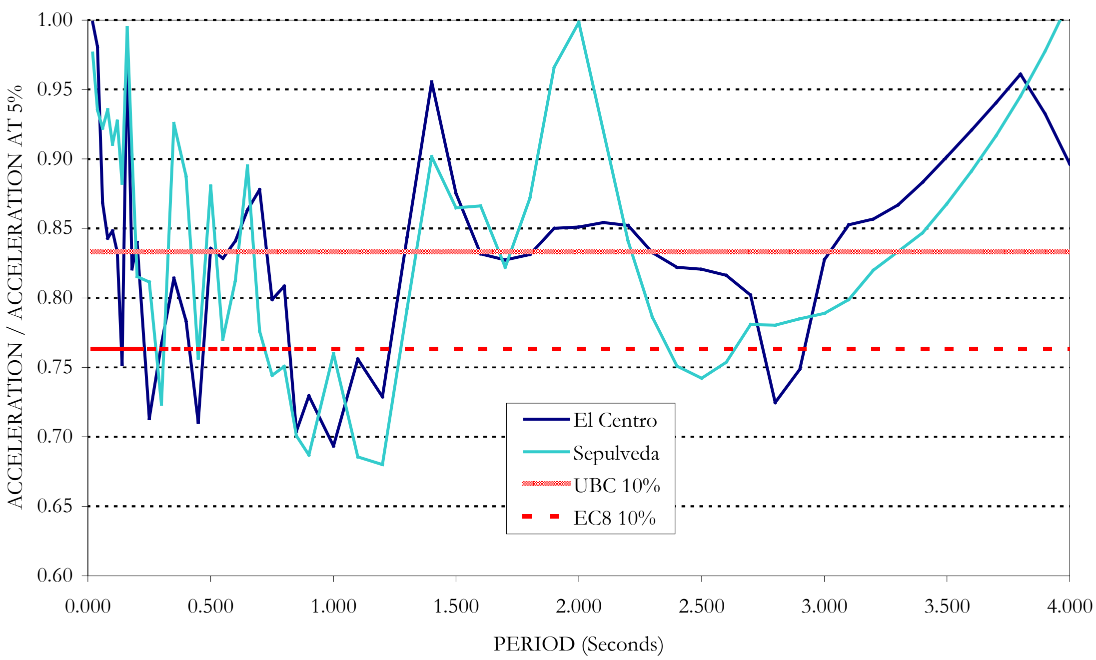


Figure 2-12 B Factor for 10% damping: Displacement

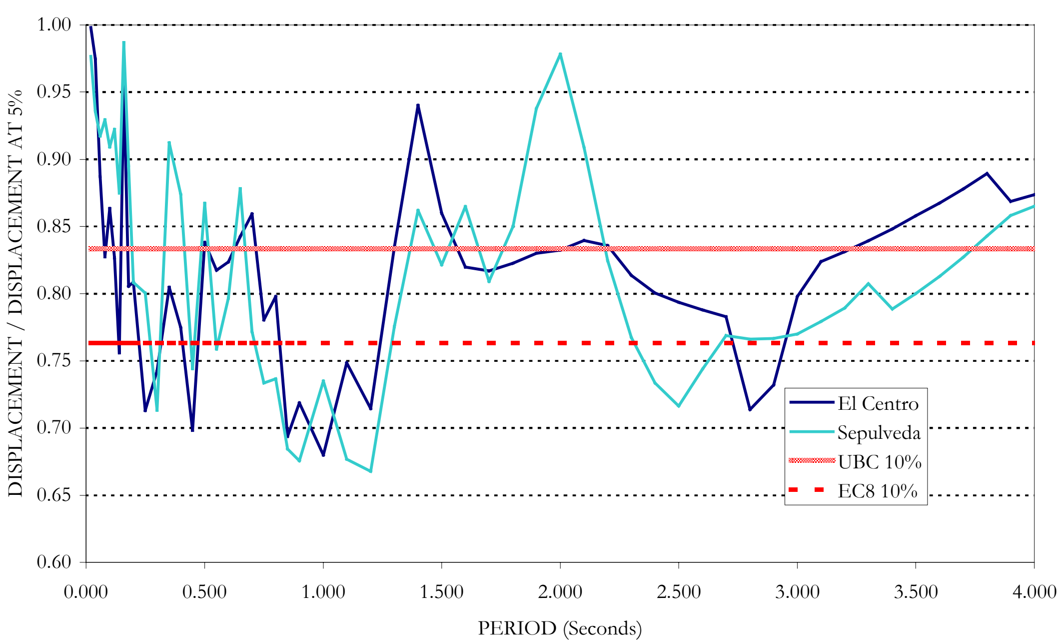


FIGURE 2-13 B FACTOR FOR 30% DAMPING: ACCELERATION

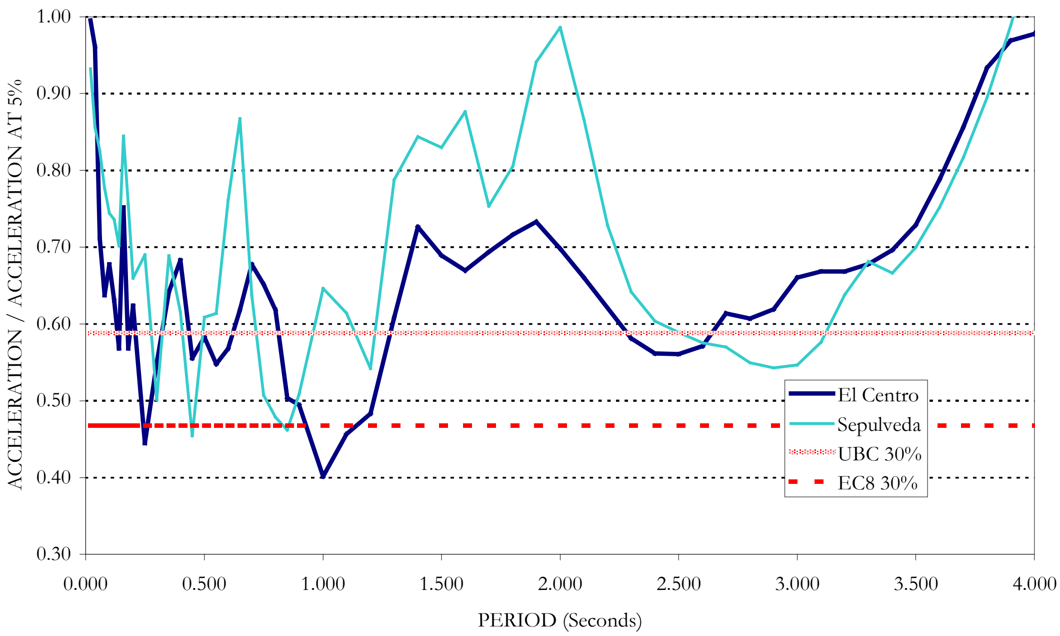
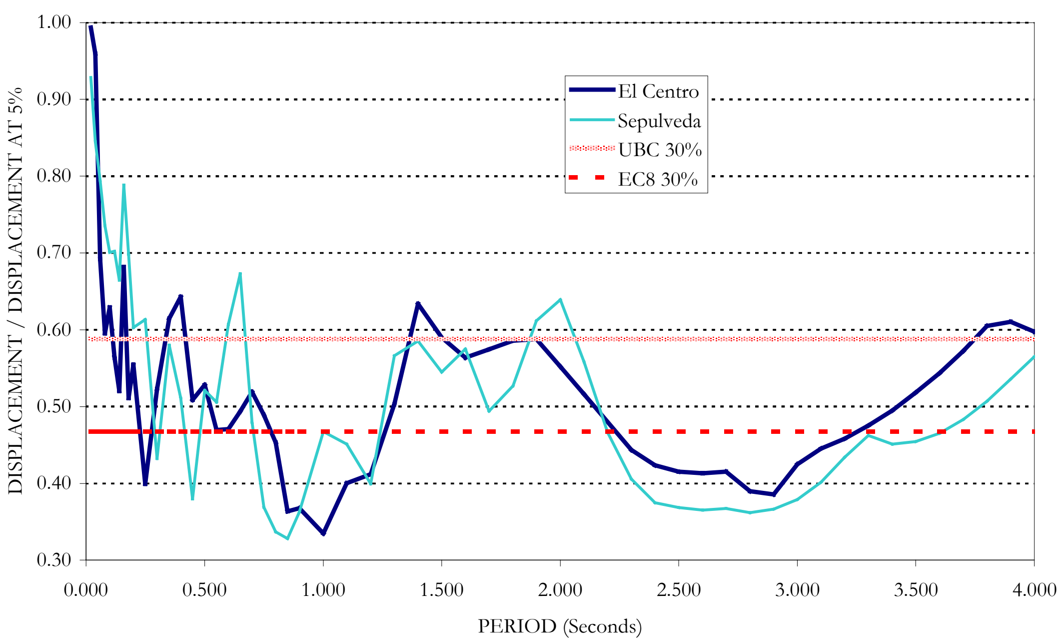


FIGURE 2-14 B FACTOR FOR 30% DAMPING: DISPLACEMENT



2.2.2 TYPES OF DAMPING

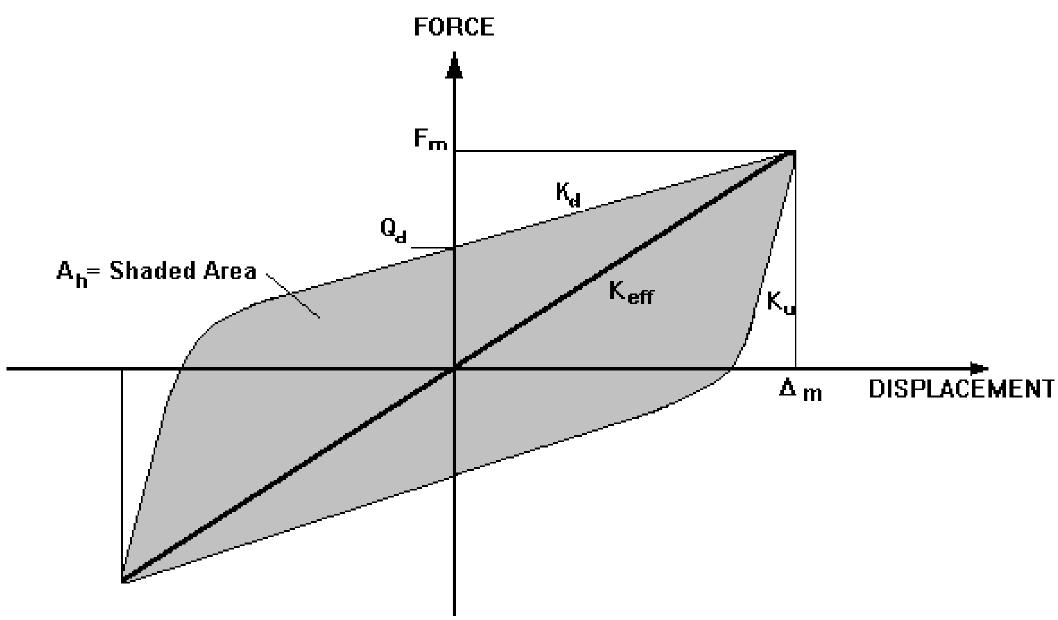
Base isolation codes represent damping arising from all sources as *equivalent viscous damping*, damping which is a function of velocity. Most types of isolators provide damping which is classified as *hysteretic,* damping which is a function of displacement.

The conversion of hysteretic damping to equivalent viscous damping, β, is discussed later but for a given displacement, Δ, is based on calculating:

$$\beta = \frac{1}{2\pi}\left(\frac{A_h}{K_{eff}\Delta^2}\right)$$

Where A_h is the area of the hysteresis loop and K_{eff} is the effective stiffness of the isolator at displacement Δ, as shown in Figure 2.15.

FIGURE 2-15 : EQUIVALENT VISCOUS DAMPING



The accuracy of using the damping factor, B, is not well documented but it appears to produce results generally consistent with a nonlinear analysis.

Figures 2-16 and 2-17 compare the results from an equivalent elastic analysis with the results from a series of nonlinear analyses using 7 earthquakes each frequency scaled to match the design spectrum. For all practical isolation periods (T > 1 second) the approximate procedure produces a result which falls between the maximum and minimum values from the nonlinear analyses. As the effective period increases beyond 2 seconds the approximate procedure tends to give results close to the mean of the nonlinear analyses.

The example given is for a single isolation system yield level and design spectral shape. However, unpublished research appears to show that the equivalent elastic approach does produce results that fall within nonlinear analysis bounds for most practical conditions. The B factor approach has the advantage that the curves shown on Figures 2-16 and 2-17 can be produced using a spreadsheet rather than a nonlinear analysis program.

FIGURE 2-16 NONLINEAR ACCELERATION SPECTRA

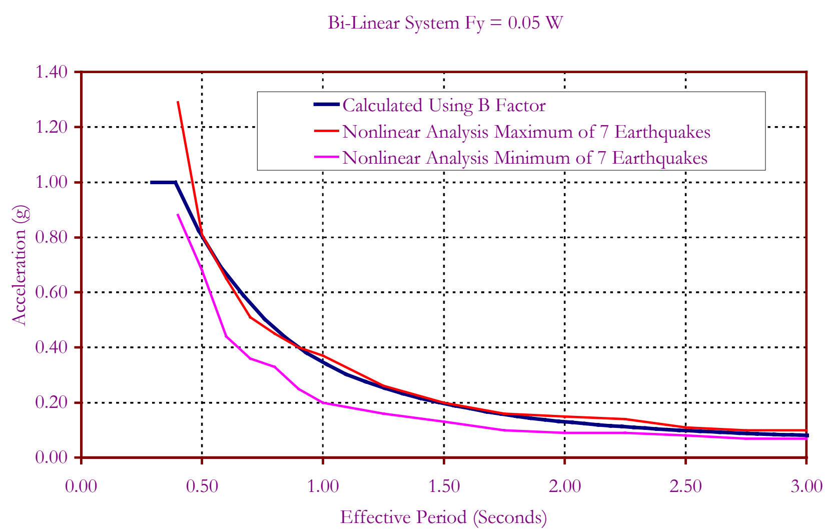
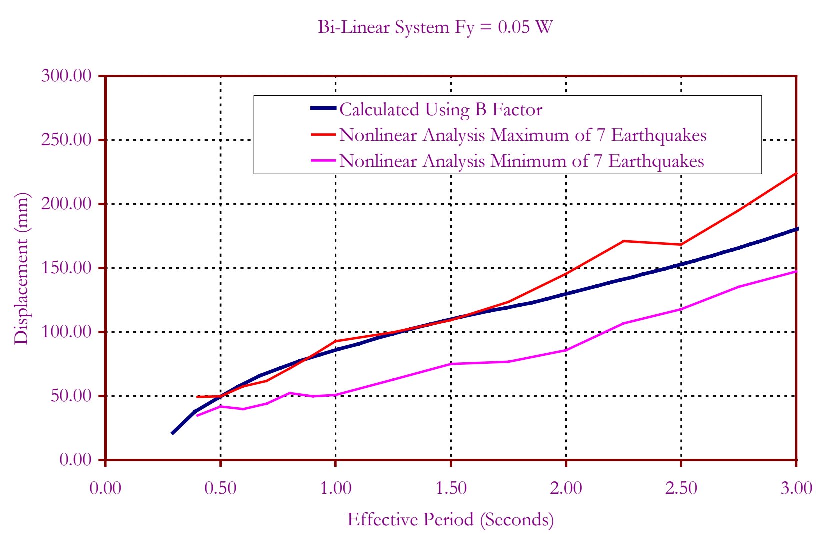


FIGURE 2-17 NONLINEAR DISPLACEMENT SPECTRA



2.3 Flexibility + Damping

Of the two main components of the isolation system, flexibility and damping, the former generally has the greatest effect on response modification, especially if the building mounted on the isolation system has a relatively short period, less than about 0.7 seconds.

Table 2-5 lists the base shear coefficients and displacements for a structure in a high seismic zone with near fault effects. In terms of displacements;

- If the structure is stiff, with a period of 0.50 seconds, a 5% damped isolation system with a 2-second period will reduce the coefficient from 1.76 to 0.57, a reduction by a factor of 3. If the damping of the isolation system is increased from 5% to 20% the coefficient reduces to 0.38, two-thirds the 5% damped value.

- If the structure is more flexible, with a period of 1.00 seconds, the 5% damped isolation system with a 2-second period will reduce the coefficient from 1.15 to 0.57, a reduction by a factor of 2. The effect of the damping increase from 5% to 20% remains the same, the coefficient reduces to 0.38, two-thirds the 5% damped value.

The reduction in acceleration response due to the flexibility of the isolation system depends on the stiffness of the building but the reduction due to damping is independent of the building stiffness.

TABLE 2-5 EFFECT OF FLEXIBILITY + DAMPING

	Base Shear Coefficient	**Total Displacement**
Fixed Base Structure		
Period 0.50 Seconds	1.76	4.30" (109 mm)
Period 1.00 Secondss	1.15	11.21" (285 mm)
Isolated to 2.00 Seconds		
5% Damping	0.57	22.43" (570 mm)
10% Damping	0.48	18.69" (475 mm)
20% Damping	0.38	14.96" (380 mm)

In terms of displacements, the flexibility effect of increasing the period to 2.0 seconds increases the displacements but the damping reduces this displacement. For the fixed base structure the displacement occurs at the centroid of the building, approximately two-thirds the building height. For the isolated building most displacement occurs across the isolation plane, with a lesser amount occurring within the structure.

2.4 Design Assuming Rigid Structure on Isolators

The acceleration and displacement spectra can be used directly to assess the performance of a rigid structure on an isolation system. If the structure is rigid then the total displacement will occur across the isolation plane. The accelerations in the structure above will all be equal to the base shear coefficient.

For a rigid structure the performance, defined by the base shear coefficient, C, and displacement, Δ, can be assessed directly if you know the design response spectrum and the stiffness and damping of the isolation system:

1. Calculate the period of the isolation system, using the total seismic weight, W, and the effective stiffness of the system, K_{eff}. For a preliminary design, you might assume an effective period, in the range of 1.5 to 2.5 seconds, and use this to define the effective stiffness required.

$$T_e = 2\pi\sqrt{\frac{W}{g\Sigma K_{eff}}}$$

2. Calculate the shear coefficient, C, from the period, T_e and the damping factor, B. The constant C_v is a function of the design spectrum – see previous discussion.

$$C = \frac{C_v}{BT_e}$$

3. Calculate the displacement from the period and shear coefficient. Use the correct units for the gravity constant, g – 386.4 in/sec² or 9810 mm/sec².

$$\Delta = \frac{gC_vT_e}{4\pi^2 B}$$

If you are doing a preliminary design, you might want to change these equations around. For example, if you want to set a maximum base shear coefficient then you can set the product BT_e and, for an assumed period, calculate the damping required.

Consider the previous UBC design in Zone 4 with a near fault factors of $N_a = 1.2$ and $N_v = 1.6$, Soil Profile S_B. The seismic coefficients are $C_a = 0.48$ and $C_v = 0.64$. The period beyond which the velocity is assumed constant is calculated as

$$T_s = \frac{C_v}{2.5C_a} = 0.533$$

and so the formulas above are applicable for periods greater than 0.533 seconds, which applies to almost all isolation systems, as the period is generally at least 1.5 seconds. For a fixed base structure of this period, or less, the base shear coefficient will be 0.64/0.533 = 1.200.

2.4.1 Design to Maximum Base Shear Coefficient

Assume we want to limit the base shear to 20% the fixed base value = 1.2 / 5 = 0.240. From the equation above

$$C = \frac{C_v}{BT_e}$$

We require the product BT to be greater than or equal to C_v/C = 0.64 / 0.24 = 2.67. If the period, T, is 2 seconds then we need B = 1.33, which is 12.6% or more damping (from Table 2-2).

This formula can be used to develop a range of designs with periods from 2.0 seconds to 3.5 seconds, as shown in Table 2-6. For a constant force coefficient, the longer the period the less damping we need. However, the longer the period the greater the displacement in the isolation system. From 2 seconds to 3 seconds the amount of damping drops from 24% to 6% but the displacement increases from 7.83" (199 mm) to more than double, 17.62" (447 mm).

Table 2-6 Design to Constant Force Coefficient

Effective Period T_e	Damping Factor B	Equivalent Viscous Damping β	Force Coefficient C	Displacement Δ (inches)	Displacement Δ (mm)
2.00	1.60	24%	0.200	7.83	199
2.10	1.52	21%	0.200	8.63	219
2.20	1.45	18%	0.200	9.47	241
2.30	1.39	16%	0.200	10.36	263
2.40	1.33	14%	0.200	11.28	286
2.50	1.28	12%	0.200	12.23	311
2.60	1.23	11%	0.200	13.23	336
2.70	1.19	9%	0.200	14.27	362
2.80	1.14	8%	0.200	15.35	390
2.90	1.10	7%	0.200	16.46	418
3.00	1.07	6%	0.200	17.62	447

2.4.2 Design to Maximum Displacement

Assume now we want to design for the same spectrum but limit the displacement to 12" (254 mm). Rearranging the equation for displacement:

$$\Delta = \frac{gC_v T_e}{4\pi^2 B}$$

We get

$$\frac{B}{T_e} = \frac{gC_v}{4\pi^2\Delta} = \frac{386.4x0.64}{4\pi^2 12} = 0.522 \quad (= 13.26 \text{ in mm units})$$

For T_e = 2 seconds, we require B = 2 x 0.522 = 1.044 which corresponds to 6.1% damping. As for the constant force coefficient, we can generate the required damping for a range of effective periods from 2 to 3 seconds, as listed in Table 2.7.

This case is the reverse of the constant force coefficient in that the longer the period, the more damping is required to keep displacements to 12". However, in this case the force coefficient reduces with increasing period. As the period increases from 2 seconds to 3 seconds the damping we need increases from 6% to 23". However, we get the benefit of the force coefficient reducing from 0.307 to 0.135, less than one-half the 2-second value.

Table 2-7 Design to Constant Displacement

Effective Period T_e	Damping Factor B	Equivalent Viscous Damping β	Force Coefficient C	Displacement Δ (inches)	Displacement Δ (mm)
2.00	1.04	6%	0.307	12.00	305
2.10	1.10	7%	0.278	12.00	305
2.20	1.15	8%	0.253	12.00	305
2.30	1.20	10%	0.232	12.00	305
2.40	1.25	11%	0.213	12.00	305
2.50	1.31	13%	0.196	12.00	305
2.60	1.36	15%	0.181	12.00	305
2.70	1.41	16%	0.168	12.00	305
2.80	1.46	18%	0.156	12.00	305
2.90	1.51	20%	0.146	12.00	305
3.00	1.57	23%	0.136	12.00	305

2.5 WHAT VALUES OF PERIOD AND DAMPING ARE REASONABLE?

Once you generate a spreadsheet using the formulas above for a particular spectrum you will find that you can solve for almost any force coefficient and/or displacement if you have complete freedom to select a period and damping. In the real world, you don't get that freedom. The detailed design procedures presented later provide a means to design devices for particular stiffness and damping. However, a few rules of thumb will help assist in using realistic values before you try to design the devices:

Effective Period

1. For most type of device, the lower the weight the lower the effective period. The period is proportional to $\sqrt{M/K}$ and so if M is small then K must also be small. Most isolator types are difficult to design for a very low stiffness. If you have a very light structure, such as a single story building or lightweight steel buildings of 2 or 3 stories then it will be difficult to isolate to periods much greater than 1.0 to 1.5 seconds.

2. Conversely, very heavy buildings are relatively simple to isolate to a long period of 2.5 to 3.0 seconds.

3. Most other buildings usually target an effective period in the range of 1.5 seconds to 2.5 seconds.

Damping

Most practical isolator types are hysteretic and produce a force-displacement curve that can be approximated as bi-linear, with an initial elastic stiffness and then a strain-hardening branch. An exception is sliding bearings, which have zero strain hardening. However, sliders are usually used in parallel with an elastic-restoring element so the overall force-displacement function has positive strain hardening.

If we assume a ratio of elastic to yielded stiffness then the function of damping versus displacement can be generated, as shown in Figure 2-18 for a ratio of K_u:K_r = 12:1, a typical value. This shows some important trends:

1. Damping reduces with displacement after reaching a peak at a relatively small displacement.

2. Damping reduces with reducing yield level (as a fraction of the building weight).

Unfortunately, these trends are the opposite of what we would want for an ideal isolation system.

1. The larger the earthquake the larger the displacement and so this is where we require maximum damping to control the displacements and forces.

2. The higher the yield level the less effective the isolation system under small to moderate earthquakes. This is because the isolation system does not start to work until the yield threshold is exceeded. If this threshold is set too high then the system will not function under more frequent earthquakes.

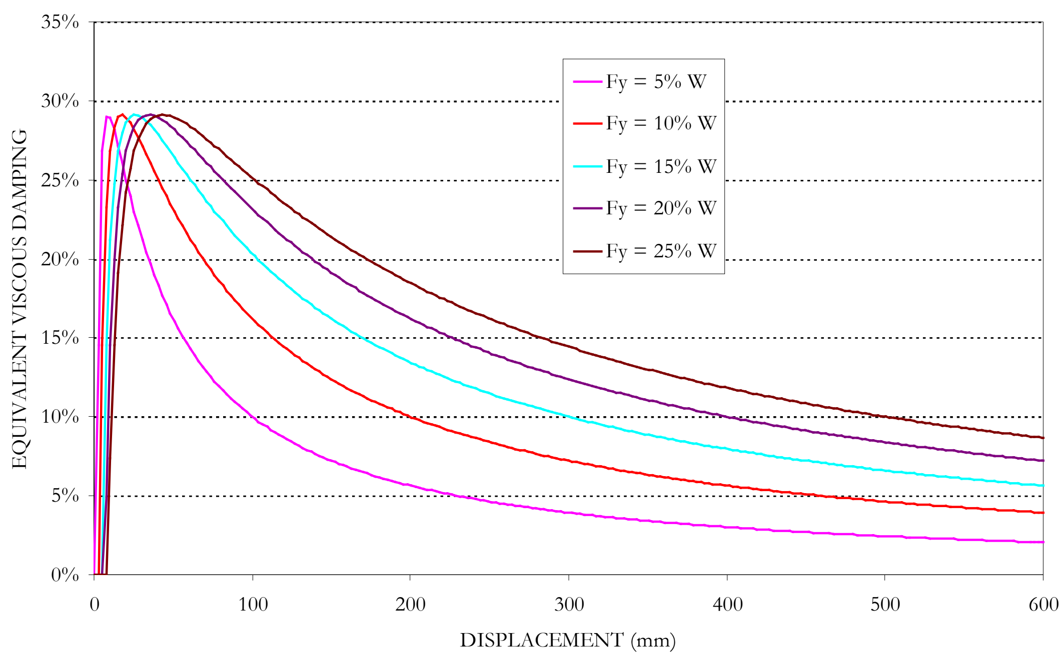
These characteristics become most problematic when design is to a two level earthquake, such as the DBE and MCE levels defined by the UBC. Rules of thumb for available damping are:

1. At DBE levels of earthquake, damping of 15% to 20% can generally be achieved.

2. In a high seismic zone the damping at MCE levels of load will often not exceed 10% to 12%.

In some projects, where MCE motions are very large, supplemental dampers may be required to boost the damping at large displacements. This is generally a very expensive option.

FIGURE 2-18 HYSTERETIC DAMPING VERSUS DISPLACEMENT



These damping values are based on the rigid structure assumption. As discussed below, for a flexible structure there are other considerations when selecting damping as highly damped systems may give rise to higher floor accelerations than lightly damped systems.

2.6 APPLICABILITY OF RIGID STRUCTURE ASSUMPTION

The situation of a rigid structure on isolators is rare. Almost all structures have a flexibility that will modify the response of the combined structure – isolation system. Some exceptions will be rigid items of equipment but most building structures and bridges will need to be evaluated beyond this simple approximation. However, the rigid structure assumption always forms the first step in developing an isolation system design.

The effects of flexibility are typically different for buildings and bridges. For buildings, the flexibility is in the structural system above the isolation plane and the isolators modify the motions input into the superstructure. For buildings, the flexibility is most commonly below the isolation plane and so the substructure modifies the motion input into the isolators. The effect of these on isolator response is discussed in more detail later in these guidelines.

2.7 Non-Seismic Loads

By definition, the base isolation system separates the structure from the ground and so must transmit all loads from the structure into the ground. Although the isolators are designed for earthquake loads, they must also be able to resist loads arising from a number of other sources:

- Gravity. The isolators must be able to support permanent (dead) and transient (live) vertical loads.

- Wind. All isolator systems except those under internal equipment must resist lateral wind loads. Almost all systems are designed to remain stationary under wind loads and so they do not damp loads from this source.

- Thermal movements are most commonly a design condition for bridges but may also effect some large building structures. Temperature variations will cause movements in the isolators. As thermal movements are a relatively frequently occurring load, the isolators must able to resist a large number of cycles of positive and negative displacements. If the isolators are installed at temperatures above or below average temperatures the cycling may be about a non-zero displacement.

- Creep and Shrinkage. As for temperature, these load conditions most commonly affect bridge isolation systems. For buildings, the flexibility of the isolators may allow large concrete floors to be constructed without joints. (For example, the Te Papa Tongarewa corner isolators have a permanent offset of 25 mm (1") due to creep of the base slab). Creep and shrinkage is uni-directional and non-reversible. For long span bridges it may form the dominant load condition. Special measures may be required, such as pre-deformed bearings or the facility to relieve creep deformations some time after construction.

- Shock and operating loads. Some equipment will have other load cases resulting from operating or extreme conditions that may apply loads to the isolators.

2.8 Requirements for a Practical Isolation System

In summary, the requirements for practical isolation system are defined by the performance objectives discussed above:

1. Flexibility.

2. Damping.

3. Resistance to service loads.

Additional requirements such as durability, cost, ease of installation and specific project requirements will influence device selection but all practical systems must contain these three essential elements.

2.9 Types of Isolators

Many types of isolation system have been proposed and have been developed to varying stages, with some remaining no more than concepts and others having a long list of installed projects. The following sections provide a discussion of generic types of system. Later chapters discuss devices that are commercially available.

2.9.1 Sliding Systems

Sliding systems are simple in concept and have a theoretical appeal. A layer with a defined coefficient of friction will limit the accelerations to this value and the forces which can be transmitted will also be limited to the coefficient of friction times the weight.

Sliders provide the three requirements of a practical system if the coefficient of friction is high enough to resist movement under service loads. Sliding movement provides the flexibility and the force-displacement trace provides a rectangular shape that is the optimum for equivalent viscous damping.

A pure sliding system will have unbounded displacements, with an upper limit equal to the maximum ground displacement for a coefficient of friction close to zero. The system provides no restoring force and so the isolated structure will likely end up in a displaced position after an earthquake and may continue to displace with aftershocks.

The lack of a restoring force may be remedied by using sliding bearings in parallel with other types which do have a restoring force or by using a shaped rather than flat sliding surface, for example, a spherical sliding surface.

2.9.2 Elastomeric (Rubber) Bearings

Elastomeric bearings are formed of horizontal layers of natural or synthetic rubber in thin layers bonded between steel plates. The steel plates prevent the rubber layers from bulging and so the bearing is able to support higher vertical loads with only small deformations. Under a lateral load the bearing is flexible.

Plain elastomeric bearings provide flexibility but no significant damping and will move under service loads. Methods used to overcome these deficits include lead cores in the bearing, specially formulated elastomers with high damping and stiffness for small strains or other devices in parallel.

2.9.3 Springs

There are some proprietary devices based on steel springs but they are not widely used and their most likely application is for machinery isolation. The main drawback with springs is that most are flexible in both the vertical and the lateral directions. The vertical flexibility will allow a pitching mode of response to occur. Springs alone have little damping and will move excessively under service loads.

2.9.4 Rollers and Ball Bearings

Rolling devices include cylindrical rollers and ball races. As for springs, they are most commonly used for machinery applications. Depending on the material of the roller or ball bearing the resistance to movement may be sufficient to resist services loads and may generate damping.

2.9.5 Soft Story, Including Sleeved Piles

The flexibility may be provided by pin ended structural members such as piles inside a sleeve that allows movement or a soft first story in a building. These elements provide flexibility but no damping or service load resistance and so are used in parallel with other devices to provide these functions.

2.9.6 Rocking Isolation Systems

Rocking isolation systems are a special case of energy dissipation that does not fit the classic definition of isolation by permitting lateral translation. The rocking system is used for slender structures and is based on the principle that for a rocking body the period of response increases with increasing amplitude of rocking. This provides a period shift effect. Resistance to service loads is provided by the weight of the structure. Damping can be added by using devices such as yielding bolts or steel cantilevers.

2.10 Supplementary Damping

Some of the isolation types listed above provide flexibility but not significant damping or resistance to service loads. Supplementary devices that may be used include:

- Viscous dampers. These devices provide damping but not service load resistance. They have no elastic stiffness and so add less force to the system than other devices.

- Yielding steel devices, configured as either cantilevers yielding acting in flexure or beams yielding in torsion. These provide stiffness and damping.

- Lead yielding devices, acting in shear, provide stiffness and damping.

- Lead extrusion devices where lead is forced through an orifice. Added stiffness and damping.

All devices apart from the viscous dampers are displacement dependent and so provide a maximum force at maximum displacement, which is additive to the force in the isolation device. Viscous dampers are velocity dependent and provide a maximum force at zero displacement. This out-of-phase response adds less total force to the system.

3 IMPLEMENTATION IN BUILDINGS

3.1 When to Use Isolation

The simple answer is when it provides a more effective and economical alternative than other methods of providing for earthquake safety. The first criterion to consider obviously relates to the level of earthquake risk – if the design for earthquake loads requires strength or detailing that would not otherwise be required for other load conditions then base isolation may be viable.

When we evaluate structures which meet this basic criterion, then the best way to assess whether your structure is suitable for isolation is to step through a check list of items which make isolation either more or less effective:

The Weight of the Structure

Most practical isolation systems work best with heavy masses. As we will see, to obtain effective isolation we need to achieve a long period of response. The period is proportional to the square root of the mass, M, and inversely proportional to the square root of the stiffness, K:

$$T = 2\pi\sqrt{\frac{M}{K}}$$

To achieve a given isolated period, a low mass must be associated with a low stiffness. Devices that are used for isolation do not have an infinite range of stiffness. For example, elastomeric bearings need to have a minimum diameter to ensure that they remain stable under seismic displacements. This minimum plan size sets a minimum practical stiffness.

Sliding systems do not have this constraint and so low weight buildings may be able to be isolated with sliding systems. However, even these tend not to be cost effective for light buildings for different reasons. Regardless of the weight of the building, the displacement is the same for a given effective period and so the size of the slide plates, the most expensive part of sliding bearings, is the same for a heavy or a light structure. In real terms, this usually makes the isolators more expensive as a proportion of first cost for light buildings.

There have been systems proposed to isolate light buildings. However, the fact remains that there are few instances of successful isolation of light structures such as detached residential dwellings.

The Period of the Structure

The most suitable structures are those with a short natural period, less than about 1 second. For buildings, that is usually less than 10 stories and for flexible types of structure, such as steel moment frames, probably less than 5 stories.

As you'll see later, practical isolation systems don't provide an infinite period, rather they shift the period to the 1.5 to 3.5 second range. If your structure is already in this period range then you won't get much benefit from isolation, although is some cases energy dissipation at the base may help. This is used quite often in bridges with a long period, less so for buildings.

Seismic Conditions Causing Long Period Waves

Some sites have a travel path from the epicenter to the site such that the earthquake motion at the site has a long period motion. This situation most often occurs in alluvial basins and can cause resonance in the isolated period range. Isolation may make the response worse instead of better in these situations. Examples of this type of motion have been recorded at Mexico City and Budapest. This is discussed later.

Subsoil Condition

Isolation works best on rock and stiff soil sites. Soft soil has a similar effect to the basin type conditions mentioned above, it will modify the earthquake waves so that there is an increase in long period motion compared to stiff sites. Soft soil does not rule out isolation in itself but the efficiency and effectiveness will be reduced.

Near Fault Effects

One of the most controversial aspects of isolation is now well the system will operate if the earthquake occurs close to the structure (within about 5 km). Close to the fault, a phenomenon termed "fling" can occur. This is characterized by a long period, high velocity pulse in the ground acceleration record. Isolation is being used in near fault locations, but the cost is usually higher and the evaluation more complex. In reality, any structure near to a fault should be evaluated for the "fling" effect and so this is not peculiar to isolation.

The Configuration

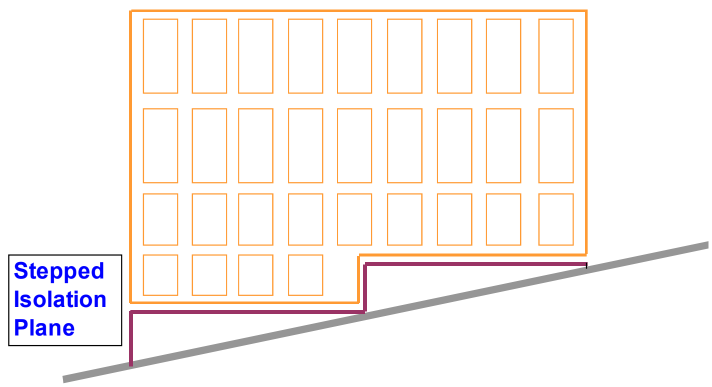
If the dynamic characteristics and site conditions are suitable for isolation, the most important item to consider is the configuration of the structure. Base isolation requires a plane of separation. Large horizontal offsets will occur across this plane during an earthquake. The space needed to allow for these displacements (often termed the "rattle" space) may range from less than 100 mm (4 inches) in low and moderate seismic zones up to 1 meter or more (40 inches) in high seismic zones close to a fault.

If there is an obstruction within this distance then isolation will not work. Impact with other structures, or retaining walls, will cause large impact accelerations that negate the use of isolation in the first place. For new buildings this is not usually a problem although the maximum clearance available may impose a restraint on the design of the isolation system. It will rule out the retrofit of buildings that closely abut other buildings.

Detailing of the isolation system is simplest if the plane of isolation is horizontal so sloping sites may cause problems. In theory, there is no reason not to step the isolation plane. In practice, it may cause a lot of trouble. As you see in Figure 3-1, a stepped isolation plane will require a vertical separation plane as well as the horizontal plane.

For buildings, the most efficient building configuration to isolate is one that requires a crawl space or basement anyway. The isolation system requires a diaphragm immediately above to distribute loads and this means the ground floor must be a suspended floor. If the ground floor would otherwise be a slab on grade then isolation will add a significant first cost. This cost penalty is accentuated if there are only a few floors in the building. For example, adding an extra suspended floor to a two story building will add a high percentage cost.

FIGURE 3-1 ISOLATING ON A SLOPE



For retrofit of structures, cost effectiveness is usually determined by how difficult it is to separate the structure and support it while the isolators are installed. Providing the separation space is often more difficult for existing structures than for new ones.

Aspect Ratio of Structural System

Most practical isolation devices have been developed to operate under compression loads. Sliding systems will separate if vertical loads are tensile. Elastomeric based systems must resist tension loads by tension in the elastomer. In tension, cavitation occurs at relatively low stresses (compared to allowable compressive stresses) which reduces the stiffness of the isolator. For these reasons, isolation systems are generally not practical for structural systems that rely on tension elements to resist lateral loads, for example, tall cantilever shear walls or narrow braced or moment frames.

A general rule of thumb is that the system should be suitable for isolation provided significant tension does not occur at any isolator location for the Design Level Earthquake. Tension is accepted for the Maximum Considered Earthquake but may complicate the analysis. If tensile stresses in elastomeric bearings exceed the cavitation limit then the effect of the reduced axial stiffness may need to be assessed; for sliding systems, uplift will occur at these locations and again, the effect of this may need to be assessed.

TABLE 3-1 A SUITABILITY CHECK LIST

Item	Checks	Score
Need for Isolation Level of Earthquake Risk Seismic Design Requirements	 Earthquake Design Required? Does seismic design add to cost?	
Site Suitability Geologic Conditions Site Subsoil Conditions Distance to Fault	 Potential for resonance effects? Stiff Soil? > 5 km from nearest active fault?	
Structure Suitability Weight of the Structure Period of the Structure Structural Configuration	 Heavy < 2 seconds Basement No tall piers Retrofit, can it be separated from the ground?	

3.2 BUILDING CODES

The most detailed code is probably the United States Uniform Building Code (UBC), eventually to be replaced with the International Building Code (IBC). Base Isolation provisions first appeared in the UBC in 1991, as an Appendix to the seismic design requirements in Chapter 23. Since then, the requirements have been extensively revised with each successive edition.

Many practitioners consider than the current UBC requirements act as an impediment to the widespread adoption of base isolation as the provisions introduce clauses on near fault effects, analysis requirements and detailed system requirements which are far more comprehensive than those required for other forms of construction. These effect both the level of engineering required to design the product and also the finished cost of the product.

Designs to the 1991 UBC typically produced isolation displacements of from 6" to 8" (150 mm to 200 mm). Designs to the latest revision, 1999, require isolator displacements typically of at least 12" (300 mm) and in some cases twice that much, 24" (600 mm). The larger displacements require bearings with larger plan sizes and this in turns leads to higher bearings to retain the flexibility. Doubling the design displacement will usually at least double the cost of the isolators and also increase the cost of architectural finishes, services etc.

The only New Zealand code that specifically addresses seismic isolation and energy dissipation is the 1995 concrete design code, NZS3101. The requirements of this code are performance based and non-prescriptive:

4.4.12 Structures incorporating mechanical energy dissipating devices

The design of structures incorporating flexible mountings and mechanical energy dissipating devices is acceptable provided that the following criteria are satisfied at ultimate limit state:

a) The performance of the devices used is substantiated by tests.

b) Proper studies are made towards the selection of suitable design earthquakes for the structure.

c) The degree of protection against yielding of the structural members is at least as great as that implied in this Standard relating to the conventional seismic design approach without energy dissipating devices.

d) The structure is detailed to deform in a controlled manner in the event of an earthquake greater than the design earthquake.

These requirements permit structural design to take advantage of the lower earthquake forces although the requirement for site specific earthquake assessment may result in a larger level of input than would be used for non-isolated structures, especially in the highest seismic regions of New Zealand. This is because site specific seismic studies include factors such as near fault effects that are not included in the code seismic loading. Conversely, the site specific seismic studies often produce seismic loads less than code levels in low seismic regions.

The commentary to the concrete code refers to design procedures, although this material is somewhat dated given developments in other parts of the world in the last 15 years.

3.3 Implementation of Base Isolation

3.3.1 Conceptual / Preliminary Design

If your project passes the check list for the need for isolation, site suitability and structure suitability then the isolation system is implemented by stepping through a conceptual/preliminary design process:

- Define the seismic isolation objectives. Does the project suit full isolation or is it more suited to an energy dissipation solution?

- Decide on the seismic isolation plane and the location of isolation devices. This is often obvious, for example, a frame building will be isolated below the ground floor with isolators supporting each column. Other structures may not be so apparent and more than one option may be carried forward to the next stages.

- Select appropriate devices. This will depend on seismicity (some devices are better suited to low displacements, some have high damping etc.) and any restrictions on size – some devices require less vertical headroom. Generally, at this stage select several potential types.

- Assess the isolation system performance for each device type. At concept stage, a single mass approximation will provide displacements and base shear coefficients. If floor accelerations are critical you may do some analyses at this stage.

- Select one or more preferred devices.

This is usually the best point to evaluate the costs and benefits of seismic isolation for the project. You should have sufficient information to approach manufacturers of the preferred devices for hardware costs. You can estimate the cost of structural changes required to install the system. The maximum displacements can be used to cost the provision of clearances and specification of flexible utility connections.

The benefits will arise from the reduction in the base shear coefficient and the floor accelerations as they affect non-structural fixings. There may be direct, first cost savings associated with these. More likely, there will be indirect savings from increased seismic safety and reduced earthquake damage. Whether these can be included into the accounting depends very much on the building owner.

3.3.2 Procurement Strategies

If seismic isolation proves effective and economical then the design process continues to the detailed design phase. The process at this stage depends on the approach taken to seismic isolation system procurement, whether prescriptive or performance based or a mix of the two.

The prescriptive approach is where you provide detailed device requirements, which usually include the maximum vertical loads for each combination, the design level and the maximum earthquake displacements and the effective stiffness and damping at each of these displacement levels. To use this method of procurement, you need a good knowledge of device characteristics. You do not want to specify requirements which are very difficult, or even impossible, to achieve.

The performance based approach is where you specify the performance you want to achieve and leave the detailed device properties up to the vendor. For example, you might specify an effective period, maximum displacements and maximum base shear coefficients at the design level and maximum earthquake. In this variation, you would usually require the vendor to submit analysis results to demonstrate compliance and you should also perform some analyses yourself to verify this.

Sometimes these two approaches are mixed – the specification details the device characteristics that will achieve the required system performance. Vendors are then permitted to submit alternative devices that will at least match this performance.

Which approach you take depends on your confidence in designing isolation systems, how much control you want to retain over the process and your capability of evaluating a range of isolation systems.

The two approaches represent a current ambivalence in seismic isolation system supply, as to whether the product is a commodity (where the prescriptive approach is typical) or an engineered system (where the performance approach is typical). In the early 1980's isolation was clearly an engineered system and design almost always involved vendors at the early stages of a project. As patents expire, more manufacturers enter the market and the product is moving more to a commodity. However, this process is not proving simple as we do not have the equivalent of steel beam safe load tables.

Eventually there will no doubt be tables of standard isolation devices and progress in this has been made in Japan. In the U.S. the wide variations in seismic requirements, particularly the near fault effects, preclude this approach.

TABLE 3-2 PROCUREMENT STRATEGIES

Approach	Description	Advantages	Disadvantages
Prescriptive	Specify detailed device characteristics, including stiffness and damping. May specify sizes.	Structural engineer retains control. Simple to evaluate bids.	Requires a structural engineer expert in isolation design. Limits potential bidders. May not be optimal system.
Performance	Specify performance requirements of the isolation system (period, displacement, damping). Vendors design devices.	Does not require expertise in device design. Wider range of bidders. Less engineering effort at design stage.	Difficult to evaluate bids. May need to check analysis of a large number of systems.
Combined	Specify a complying system as for prescriptive approach. List performance of this system and allow other devices that can match this.	Widest range of bidders. Most likely to attract optimal design.	Requires design expertise. Difficult to evaluate bids. May need to check analysis of a large number of systems.

3.3.3 DETAILED DESIGN

Once the concept is accepted and a procurement strategy established, the detailed design follows much the same process as for any other structural design:

- Analyze the structure and assess the detailed structural performance for the selected system.
- Develop either device characteristics or performance criteria for the project specifications.

- Design and detail the connections of the isolators to the structure above and below.

- Document as for any other design – contract drawings and specifications.

3.3.4 Construction

As for detailed design, the construction phase proceeds as for any other structure although there are some additional requirements for a seismic isolation system:

- Codes typically require prototype and quality control tests. You will need to supervise these tests and evaluate the results for compliance with the specifications. These may also affect the construction schedule as typically 2 to 3 months are required to manufacture, test and evaluate prototype bearings.

- The installation may have special requirements, particularly for seismic retrofit.

- A program is required for maintenance, servicing and post-earthquake inspection.

3.4 Costs of Base Isolation

Regardless of the answers to the why and how questions, the cost of isolation will always be an important consideration and this is one of the first questions asked by most engineers considering isolation. There are both direct and indirect costs and cost savings to consider.

In most cases, a new isolated building will cost more than a non-isolated building, usually in the range of 0% to 5% of total cost more. The installation of the isolation system will always add to first cost as a non-isolated building would not have bearings. The structure is designed for a higher level of performance than non-isolated buildings and full advantage is not taken of the reduction in forces to reduce costs in the structure above the isolators (the ductility is generally less than one-half that for a non-isolated building – see Chapter 12). This restricts savings in the structural system that might otherwise offset the isolation system costs.

For the retrofit of buildings, a solution using isolation will often cost less than other non-isolation strengthening schemes. This is because ductile design is less common in retrofit and so the isolated and non-isolated designs are more comparable.

3.4.1 Engineering, Design and Documentation Costs

An isolated structure requires lots of extra engineering effort to analyze, design, detail and document – the scope can be appreciated from the tasks in the design process described above. The extra costs associated with this very much depend on the project. A few things to consider:

- Analysis effort is usually the largest added engineering cost. The analysis type, and cost, depends on the building and location. Few isolated structures can be analyzed using the equivalent static load method so at least a response spectrum analysis is required. Some structures require a time history analysis. Even if a non-isolated building would be analyzed the same way, an isolated structure analysis requires more effort. For example, a response spectrum analysis of an isolated building is usually iterative as the stiffness properties and damping are a function of the displacement, which is itself a function of stiffness and damping.

- The vendor will often perform design of the isolation system and so this may not add a lot to the engineering costs. However, there will still be time involved in evaluating designs.

- Detailing of the isolator connections is an added cost. The large displacements cause secondary moments (P-Δ effects) which involve significant design effort.

- At the tender stage, you will generally have to evaluate a number of proposals, often complex and difficult to verify.

- You will need to allow for supervision and evaluation of prototype and production tests.

- Extra site supervision may be needed for installation.

No cost savings in design and documentation from using isolation come to mind. There may be simplifications from using elastic design versus ductile design but that is unusual.

3.4.2 Costs of the Isolators

There is a wide range of cost of the isolators. For most types, the cost is influenced most by the maximum displacement and to a lesser extent by the loads that they support. For a given level of seismic load, displacement is proportional to the isolated period and so the greater the extent of isolation, the greater the cost.

The cost per device can range from $500 to $10,000 or more (US dollars, year 2001).

The total cost for the isolation system depends on the efficiency of the isolator layout. Generally, the higher the load supported per isolator the higher the efficiency. For example, the total system cost for a structure supported on 50 isolators in a high seismic zone will be probably about 20% to 40% less than if a structure of the same weight were supported on 100 isolators. This is because isolator sizes, and so cost, will be determined primarily by the displacement and is only a weak function of axial load for most device types.

3.4.3 Costs of Structural Changes

The cost of changes to the structural configuration is potentially the largest component of the first cost and is very much a function of the building layout. A building with a basement can often be isolated below the ground floor level with little added cost. A building that would have a slab on grade will require a suspended floor. The difference in cost between a suspended floor and a slab on grade will add significantly to the construction cost.

Other costs may arise for the portion of the structure below the isolation plane. For example, if the isolators are on top of basement walls, below ground floor, they will apply out-of-plane loads to the basement walls. Pilasters or buttresses may be needed to resist these loads.

Obviously, the costs of structural changes to accommodate isolation are very project specific. They generally range from 0% to a high of perhaps 20% of structural costs, although the extremes are unlikely. The most common added cost will be in the range of 1% to 3%. In some cases, other savings above the isolators will offset these – see below.

3.4.4 Architectural Changes, Services and Non-Structural Items

Most added architectural costs arise from the detailing of the separation around the building. There can be no obstructions within a distance equal to at least the maximum displacement of the isolation system. This will require special detailing, especially as regards entrances to the building. Stairs will need to cantilever down from the isolated superstructure or be supported on sliding bearings.

All services entering the building will cross the isolation plane and so will have imposed displacements during earthquakes. The provision of flexible joints will have a cost. Elevator shafts will cross the isolation plane and will require special detailing, often cantilevering down from the isolated superstructure.

As for structural changes, the cost of these items varies widely and the range of costs is usually similar to the structural changes, about 1% to 3% of structural cost.

3.4.5 Savings in Structural System Costs

The philosophy of seismic isolation is to reduce earthquake forces on the structural system and so it follows that a system designed for lower forces will cost less. The extent of force reduction depends on the structure, the level of seismicity and the extent of isolation. Generally the earthquake forces will be reduced by a factor of at least 3 and may be reduced by a factor of 8 or more for ideal situations.

Unfortunately, a reduction in forces by a factor of say 5 does not reduce costs by the same amount. The structural system must still resist other loads such as gravity and wind and these may set minimum sizes and strengths of structural elements.

More importantly, the force reductions provided by isolation are generally of the same order as the force reductions used to account for structural ductility in a non-isolated structure. For example, in the 1997 UBC the maximum earthquake force in a non-isolated building is reduced by a response modification factor, R, which ranges from a minimum of 2.2 for cantilevered column buildings to a maximum of 8.5 for special moment-resisting frames.

An isolated building absorbs energy through the isolators rather than through ductile response of the structural system. If the structure above the isolator were designed for the same levels of ductility as for a non-isolated structure then it is likely that the structural yielding would reduce the efficiency of the isolation system. Further, a ductile system softens and extensive ductility could lead to the period of the structure degrading to a value similar to that of the isolation system, leading to the possibility of coupling between the two systems and undesirable resonance. For these reasons, the structure above the isolation system is designed for very low levels of ductility, if any. Again using the UBC as an example, the response modification factor for isolated structure, R_I, ranges from a minimum of 1.4 for cantilevered column buildings to a maximum of 2.0 for special moment-resisting frames.

A consequence of this restriction on the extent of ductile response in isolated structures is that the potential for cost savings in the structural system is highest for structural types with low inherent ductility. For very ductile systems, such as special moment resisting frames, there are unlikely to be any savings in the structural system cost. The non-isolated R = 8.5, the isolated R_I =2.0, so the isolation system must reduce maximum forces by a factor of 4.25 just to match the design forces in the non-isolated frame.

Although the potential for cost savings is low if the same structural system is used, isolation may permit a less ductile system to be used. For example, the value of R_I is the same for all isolated moment-resisting frame types (special, intermediate or ordinary) whereas for a non-isolated frame the value of R ranges from 3.5 to 8.5. Depending on seismicity, it may be possible to design an isolated intermediate frame instead of a special moment frame. This will result in some savings.

For retrofit, the savings in the structural system are usually far greater than for a new building. This is because most existing structures that pose an earthquake risk have this classification because of a lack of ductility. New structural systems have to be designed for near elastic levels of load (low R factors) else the inelastic displacements will cause failure of the existing elements. In this case, the force reductions achieved by isolation can be used directly in the structural design.

3.4.6 Reduced Damage Costs

The reality is, no matter how much first cost saving is targeted, the isolated building will be less damaged than a non-isolated building. This is because of the lower levels of ductility designed into the isolated building. The reduced costs may be even more dramatic in the non-structural items and contents of the structure than it is in the structural system. This arises from the reductions in floor accelerations and in structural drifts.

It is difficult to quantify reduced damage costs because life cycle analysis is not usually performed for most structures. As Performance Based Design becomes more widespread it is possible that this may occur. In the meantime there are some tools available to assess the reduced costs of damage (e.g. Ferritto, from which Tables 3-3 and 3-4 have been extracted).

With life cycle cost analysis the costs of earthquake damage are estimated from data such as that in Tables 3-3 and 3-4. There are two components of damage in earthquakes:

1. Drift related damage. Imposed deformations from drift will damage the primary structure and also non-structural components such as cladding, windows, partitions etc.

2. Accelerations. Inertia forces from floor accelerations will damage components such as ceilings and contents.

For non-isolated buildings, it is difficult to control both of these causes of damage. A building can be designed stiffer to reduce drifts and reduce damage costs from this cause but the floor accelerations tend to be higher in stiffer buildings and so acceleration-related damage will increase.

This can lead to some counter intuitive situations; the design studies on the Museum of New Zealand (Te Papa Tongarewa) showed that, without isolation, damage costs tended to increase as the building was made stronger. This is because the increased acceleration related damage costs more than outweighed the reductions in drift related damage costs.

Unless the building is of special importance, it is rare for life cycle costs to be calculated and so earthquake damage cost reduction can only be accounted for in a qualitative way. However, Tables 3-3 and 3-4 indicate the extent of cost reductions. For example, assume a base isolation system reduces drifts from 2% to 0.5% and accelerations from 0.5g to 0.18g, reductions which are usually easily achieved with isolation. The average drift related damage cost ratios will reduce from 0.29 to 0.06 and the acceleration costs from 0.39 to 0.09. On average, damage costs will reduce from about 35% of the total building cost to about 8% of the building cost. On this basis, a first cost increase of less than 5% is well justified.

In some earthquake prone regions, such as California, building purchasers and financiers take into account the Probable Maximum Loss (PML) for a structure in determining its value. The reduction in PML will generally show a positive net return from the use of isolation.

TABLE 3-3 DAMAGE RATIOS DUE TO DRIFT

	Story Drift (%)									
	Repair Multiplier	**0.1**	**0.5**	**1.0**	**2.0**	**3.0**	**4.0**	**7.0**	**10**	**14**
Rigid Frame	2.0	0	0.01	0.02	0.05	0.10	0.20	0.35	0.50	1.0
Braced Frame	2.0	0	0.03	0.14	0.22	0.40	0.85	1.0	1.0	1.0
Shear Wall	2.0	0	0.05	0.30	0.30	0.60	0.85	1.0	1.0	1.0
Non-seismic Frame	1.5	0	0.005	0.01	0.02	0.10	0.30	1.0	1.0	1.0
Masonry	2.0	0	0.10	0.20	0.50	1.00	1.00			
Windows and Frames	1.5	0	0.30	0.80	1.00					
Partitions, architectural elements	1.25	0	0.10	0.30	1.00					
Floor	1.5	0	0.01	0.04	0.12	0.20	0.35	0.80	1.00	1.00
Foundation	1.5	0	0.01	0.04	0.10	0.25	0.30	0.50	1.00	1.00
Equipment and plumbing	1.25	0	0.02	0.07	0.15	0.35	0.45	0.80	1.00	1.00
Contents	1.0	0	0.02	0.07	0.15	0.35	0.45	0.80	1.00	1.00

TABLE 3-4 DAMAGE RATIOS DUE TO FLOOR ACCELERATION

		Floor Acceleration (g)				
	Repair Multiplier	**0.08**	**0.18**	**0.50**	**1.2**	**1.4**
Floor and Roof System	1.5	0.01	0.02	0.10	0.50	1.0
Ceilings and Lights	1.25	0.01	0.10	0.60	0.95	1.0
Building Equipment & Plumbing	1.25	0.01	0.10	0.45	0.60	1.0
Elevators	1.5	0.01	0.10	0.50	0.70	1.0
Foundations (slab on grade, sitework)	1.5	0.01	0.02	0.10	0.50	1.0
Contents	1.05	0.05	0.20	0.60	0.90	1.0

3.4.7 DAMAGE PROBABILITY

Major earthquakes have a low probability but high consequences. For benefit cost studies, they have a low annual probability and so the earthquake damage cost may be very low on a net present value calculation. However, the low annual probability may not be reassuring to an owner who wants to know "What happens if the earthquake occurs next year?" For evaluating this, cost benefit analysis can be based on *conditional probability*, assuming that the event occurs within the design life of the building. This approach tends to markedly increase the B/C ratio.

3.4.8 Some Rules of Thumb on Cost

The additional engineering and documentation costs compared to a non-isolated design will probably be at least 20% and may be much more for your first project. The total range of costs will be about that shown in Table 3-5. Excluding reduced damage costs, the added costs may range from a minimum of –3.5% to +12% of the total building cost.

Table 3-5 Isolation Costs as Ratio to Total Building Cost

Item	Lower Bound	Upper Bound
Engineering and Documentation	0.1%	0.5%
Isolators	0.5%	5%
Structural Changes	0%	5%
Architectural & Services Changes	1%	5%
Savings in Structural System	-5%	0%
Reduced Damage Costs	-25%	-50%

3.5 Structural Design Tools

3.5.1 Preliminary Design

Isolator design is based on material and section properties as for any other type of structural section. Similar tools are used as for example for reinforced concrete sections, such as spreadsheets. The solution for the response of an isolated system based on a single mass is a straightforward procedure although for a non-linear system the solution will be iterative. Again, spreadsheets can be set up to solve this type of problem. In my experience, the complete isolator design and performance evaluation can be performed using a single spreadsheet.

3.5.2 Structural Analysis

The analysis of an isolated building uses the same procedures as for a non-isolated building, that is, in increasing order of complexity, equivalent static analysis, response spectrum analysis or time history analysis. The criteria for an isolated structure to be to be designed using the equivalent static load method are so restrictive that this method is almost never used. The most common methods are dynamic, as would be expected given the characteristics of an isolated building.

Most isolated structures completed to date have used a time history analysis as part of the design verifications. Codes often permit a response spectrum analysis, which requires a lot less analytical effort than time history. Recent codes (e.g. UBC) do not require time history analysis unless the site is especially soft or the isolation system selected has special characteristics (lack of restoring force or dependence on such factors as rate of loading, vertical load or bilateral load).

Although a response spectrum analysis may be used for most structures, the procedure is usually more complex than for non-isolated structures as a linear analysis procedure is used to represent a non-linear system. For most isolation systems, both the stiffness and the damping are displacement dependent. However, for a given earthquake the displacement is itself a factor of both stiffness and damping. This leads to an iterative analysis procedure – a displacement is assumed, stiffness and damping calculated and the model analyzed. The properties are then adjusted based on the displacement from the analysis.

Because the response of the isolated structure is dominated by the first mode the performance evaluation based on a single mass approximation will generally give a good estimate of the maximum displacement and so the number of iterations is usually not more than one or two.

The response spectrum analysis can be performed using any computer program with these capabilities (e.g. ETABS, SAP2000, and LUSAS). Most of these programs can also be used for a time history analysis if required.

As discussed later, the studies we have performed suggest that the response spectrum analysis seriously under-estimates overturning moments and floor accelerations for most isolation systems. Until this issue is resolved, we should not use this method for final design. Note that our most common linear elastic analysis tools, ETABS and SAP2000, can be used to perform the time history analysis and so this is not an undue impediment to use.

3.6 So, Is It all Too Hard?

To most engineers, seismic isolation is a new technology and the sheer scopes of things to consider may make it just seem too hard. Current codes do not help as, for example, the UBC has a complete section on seismic isolation which will be entirely new territory to an engineer starting out in isolation design.

The key is to realistically evaluate your structure, and not to have too high an expectation of cost savings from the outset. If the project that you select is a good candidate for isolation then the procedure will follow in a straightforward manner. If it has characteristics which make it a marginal or bad candidate than eventually problems will arise with the isolation system design and evaluation. These may be such that you will abandon the concept and never want to try it again. So, pick your target carefully!

As discussed earlier, the reduction in earthquake forces achieved with isolation does not translate into a similar reduction in design forces. The reason for this is ductility, as the force reductions permitted for ductility in non-isolated buildings are similar to those achieved by the isolation system. However, the isolated building will have a higher degree of protection against earthquake damage for the same, or lower, level of design force.

These features of isolation lead to building types which are more suitable for isolation for other because of particular characteristics. Table 3-6 list categories which are most suited. If you examine the HCG project list at the start of these guidelines, you will see that most completed projects fall into one of these categories.

Items which are positive indications of suitability are:

- Buildings for which continued functionality during and after the earthquake are essential. These buildings generally have a high importance factor, I.

- Buildings which have low inherent ductility, such as historic buildings of unreinforced masonry. These buildings will have a low ductility factor, R.

- Buildings which have valuable contents.

If any of these conditions apply to your project then it will generally by easier to justify the decision to isolate than is none apply.

TABLE 3-6 SUITABLE BUILDINGS FOR ISOLATION

Type of Building	Reasons for Isolating
Essential Facilities	Functionality High Importance Factor, I
Health Care Facilities	Functionality High Importance Factor, I
Old Buildings	Preservation Low R
Museums	Valuable Contents
Manufacturing Facilities	Continued Function High Value Contents

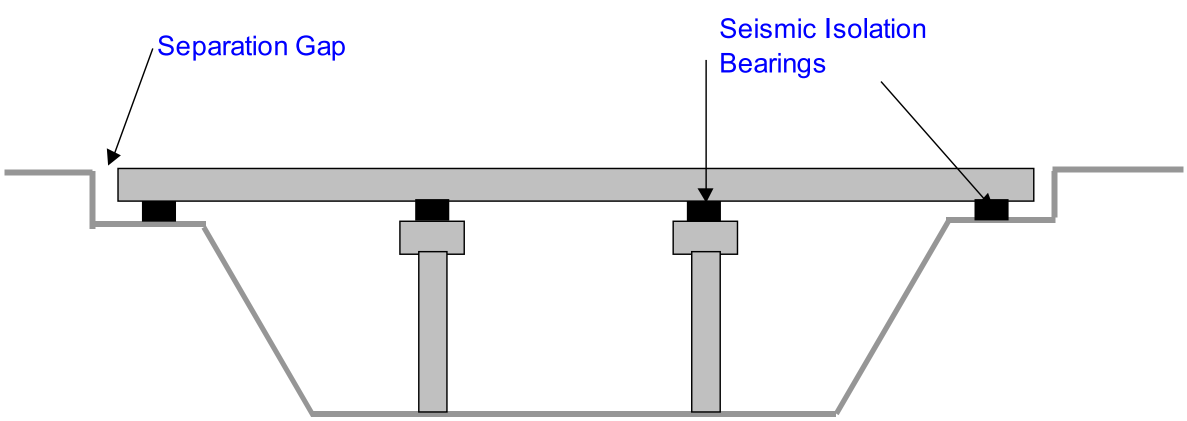
4 IMPLEMENTATION IN BRIDGES

The concept of isolation for bridges is fundamentally different than for building structures. There are a number of features of bridges which differ from buildings and which influence the isolation concept:

- Most of the weight is concentrated in the superstructure, in a single horizontal plane.
- The superstructure is robust in terms of resistance to seismic loads but the substructures (piers and abutments) are vulnerable.
- The seismic resistance is often different in the two orthogonal horizontal directions, longitudinal and transverse.
- The bridge must resist significant service lateral loads and displacements from wind and traffic loads and from creep, shrinkage and thermal movements

The objective of isolation a bridge structure also differs. In a building, isolation is installed to reduce the inertia forces transmitted into the structure above in order to reduce the demand on the structural elements. A bridge is typically isolated immediately below the superstructure and the purpose of the isolation is to protect the elements below the isolators by reducing the inertia loads transmitted from the superstructure.

FIGURE 4-1 TYPICAL ISOLATION CONCEPT FOR BRIDGES



Although the type of installation shown in Figure 4-1 is typical of most isolated bridges, there are a number of variations. For example, the isolators may be placed at the bottom of bents; partial isolation may be used if piers are flexible (bearings at abutments only); a rocking mechanism for isolation may be used.

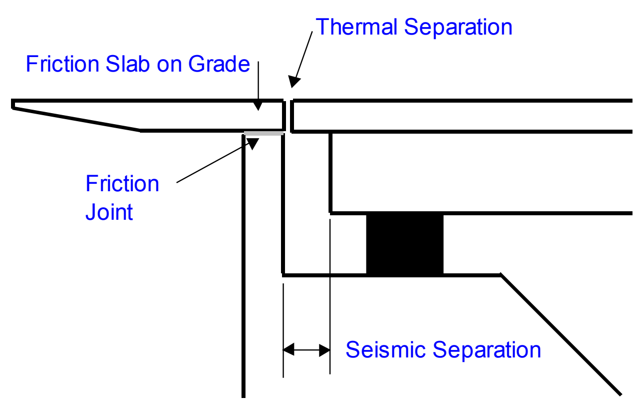
Bridge isolation does not have the objective of reducing floor accelerations which is common for most building structures. For this reason, there is no imposed upper limit on damping provided by the isolation system. Many isolation systems for bridge are designed to maximize energy dissipation rather than providing a significant period shift.

4.1 Seismic Separation of Bridges

It is often difficult to provide separation for bridges, especially in the longitudinal direction. However, there is always the question, does it matter? For any isolated structure, if there is insufficient clearance for the displacement to occur then impact will occur.

For buildings, impact almost always has very undesirable consequences. The impact will send a high frequency shock wave up the building, damaging the contents that the isolation system is intended to protect.

Figure 4-2 Example "Knock-Off" Detail



For bridges, the most common impact will be the superstructure hitting the abutment back wall. Generally, the high accelerations will not in themselves be damaging and so the consequences of impact may not be high. The consequences may be minimized by building in a failure sequence at the location of impact. For example, a slab and "knock off" detail as shown in Figure 4-2.

An example of the seismic separation reality is the Sierra Point Bridge, on US Highway 101 between San Francisco and the airport. This bridge was retrofitted with lead rubber bearings on top of existing columns that had insufficient strength and ductility. The bearings were sized such that the force transmitted into the columns at maximum displacement would not exceed the moment capacity of the columns. The existing superstructure is on a skew and has a separation of only about 50 mm (2") at the abutments. In an earthquake, it is likely that the deck will impact the abutment. However, regardless of whether this occurs, or the superstructure moves transversely along the direction of skew, the columns will be protected as the bearings cannot transmit a level of shear sufficient to damage them. There may well be local damage at the abutments but the functionality of the bridge is unlikely to be impaired. This type of solution may not achieve "pure" isolation, and may be incomplete from a structural engineer's perspective, but nevertheless it achieves the project objectives.

4.2 Design Specifications for Bridges

Design of seismic isolations systems for bridges often follow the AASHTO Guide Specifications, published by the American Association of State Highway and Transportation Officials. The initial specifications were published in 1991, with a major revision in 1999.

These bridge design specifications have in some ways followed the evolution of the UBC code revisions. The original 1991 edition was relatively straightforward and simple to apply but the 1999 revision added layers of complexity. Additionally, the 1999 revision changed the calculations of the seismic limit state to severely restrict the use of elastomeric type isolators under high seismic demands.

4.2.1 The 1991 AASHTO Guide Specifications

The 1991 AASHTO seismic isolation provisions permitted isolated structures to be designed for the same ductility factors (as implemented through the R factor) as for non-isolated bridges. This differed from buildings where the UBC at this time recommended an R value for isolated structures of one-half the value for non-isolated structures. However, AASHTO recommended a value of R = 1.5 for essentially elastic response as a damage avoidance design strategy.

AASHTO defined two response spectrum analysis procedures, the single-mode and multi-mode methods. The former was similar to a static procedure and the latter to a conventional response spectrum analysis. Time history analysis was permitted for all isolated bridges and required for systems without a self-centering capability (sliding systems).

Prototype tests were required for all isolation systems, following generally similar requirements to the UBC both for test procedures and system adequacy criteria.

In addition to the seismic design provisions, the 1991 AASHTO specifications provided additions to the existing AASHTO design provisions for Elastomeric Bearings when these types of bearing were used in implementing seismic isolation design. This section provided procedures for designing elastomeric bearings using a limiting strain criterion. As this code was the only source providing elastomeric design conditions for seismic isolation the formulations provided here were also used in design of this type of isolator for buildings (see Chapter 9 of these Guidelines).

4.2.2 The 1999 AASHTO Guide Specifications

The 1999 revision to the AASHTO Guide Specifications implemented major changes. The main differences between the 1991 and 1999 Guide Specifications were:

- Limitations on R factors. The R factor was limited to one-half the value specified for non-isolated bridges but not less than 1.5. For bridges, this provided a narrow range of R from 1.5 to 2.5, implying very limited ductility.

- An additional analysis procedure, the Uniform Load Method. This is essentially a static load procedure that takes account of sub-structure flexibility.

- Guidelines are provided for analyzing bridges with added viscous damping devices.

- Design must account for lower and upper bounds on displacements, using multipliers to account for temperature, aging, wear contamination and scragging. These factors are device-specific and values are provided for sliding systems, low-damping rubber systems and high-damping rubber systems. In general the multipliers tend to have the greatest effect in increasing displacements in sliding systems. This is similar to UBC that requires a displacement multiplier of 3.0 for sliders.

- More extensive testing requirements, including system characterization tests. There are requirements for vertical load stability design and testing using multipliers that are a function of seismic zone.

- Additional design requirements for specific types of device such as elastomeric bearings and sliding bearings.

The 1999 AASHTO specifications introduce a number of new factors and equations but a commentary is provided and the procedures are straight-forward to apply. The HCG spreadsheet Bridge.xls incorporates the 1999 AASHTO provisions and performs analysis based on (1) the uniform load method and (2) the time history analysis method.

Figure 4-3 is an example of the *Control* sheet of the Bridge workbook. The procedure for a bridge isolation system design is as follows:

- Enter design information on the *Design* worksheet. Data includes bent and superstructure weights, bent types and dimensions and span lengths.

- Enter isolator data on the *Control* worksheet. This includes number and type of bearings per bent, plan size, layers etc. Use the detailed isolator assessment on the *Isolators* sheet to select plan size. The layers and lead core sizes are selected by trial and error.

- Activate the *Solve Displacement* macro from the button on the *Control* sheet. This solves for the isolation performance using the uniform load method.

- Activate the *Nonlinear Analysis* macro from the button on the *Control* sheet. This solves for the isolation performance using the time history method. This macro assembles longitudinal and transverse models and analyzes them for seven spectrum compatible acceleration records using a version of the DRAIN-2D program. This will run in a window. You need to wait until this is complete (20 to 30 seconds usually) and then activate the *Import Results* macro.

The results of each step are summarized on the *Control* sheet. The comparison between the analysis and design results should be checked. Usually the longitudinal analysis will produce results between 10% to 20% lower than the design procedure, which is a function of the more accurate damping model. The transverse analysis will often provide a different load distribution from the design procedure, especially if the deck if flexible. This is because the effects of torsion and deck flexibility are more accurately modeled in the time history type of analysis.

The *Control* sheet lists the status of each isolator is terms of the 1999 AASHTO equations at either OK or NG. The *Isolators* sheet provides detailed calculations for the seismic and non-seismic load combinations.

FIGURE 4-3 BRIDGE BEARING DESIGN PROCESS

ISOLATION BEARING DESIGN

Solve Displacement

Nonlinear Analysis

Import Results

Job title	EAST MISSOULA - BONNER MP 109.409		
Bridge Number:	1		
Directory	C:\JOBS\SKELLERU\montana		
Units	US	(US (kip,ft) or Metric (KN,m)	
Gravity	32.2	386.4	12

AASHTO	
G	0.10
S	1.50

ISOLATORS (Units inches)	BENT 2	PIER 3	BENT 4
Number of Bearings	4	4	4
Type (LR, HDR, TFE, FIX, NONE)	LR	LR	LR
Isolator Plan Dimension	12	14	12
Number of Layers	15	15	15
Isolator Rubber Thickness	3.00	3.00	3.00
Isolator Lead Core Size	4.0	3.0	4.0
Kr	11.0	14.3	11.0
Ku	171.0	141.4	171.0
Qd	60.3	33.9	60.3
Dy	0.38	0.27	0.38

	BENT 2	PIER 3	BENT 4	SUM	MAXIMUM		
PERFORMANCE							
NO ISOLATORS							
Longitudinal Displacement	1.0	1.0	1.0		1.0	T	0.68
Longitudinal Force	31	202	47	280	202		
Transverse Displacement	0.4	0.3	0.4		0.4	T	0.36
Transverse Force	54	180	73	308	180		
WITH ISOLATORS							
Longitudinal Displacement	1.4	1.4	1.4		1.41	CONVERGED	
Longitudinal Force	37	50	52	139	52	T	1.15
Transverse Displacement	1.0	0.9	0.9		0.98	CONVERGED	
Transverse Force	64	45	64	173	64	T	0.83
LONGITUDINAL ANALYSIS							
Displacement	1.3	1.3	1.3		1.3		
Isolator Force	33	49	47	129	49		
TRANSVERSE ANALYSIS							
Displacement	0.5	1.2	0.6		1.2		
Isolator Force	37	50	49	136	50		
RATIO ANALYSIS/DESIGN							
Longitudinal Displacement	90%	90%	90%				
Longitudinal Force	90%	97%	90%				
Transverse Displacement	52%	128%	66%				
Transverse Force	58%	110%	76%				

ISOLATOR STATUS	BENT 2	PIER 3	BENT 4
Maximum Displacement	0.2	1.1	0.3
AASHTO Condition 1	OK	OK	OK
AASHTO Condition 2	OK	OK	OK
AASHTO Condition 3	OK	OK	OK
Buckling	OK	OK	OK
Reduced Area			

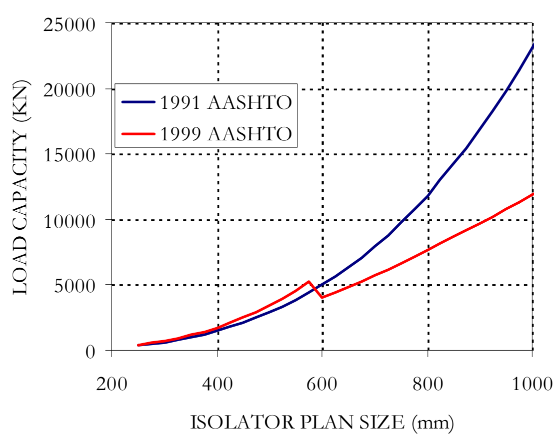
4.3 USE OF BRIDGE SPECIFICATIONS FOR BUILDING ISOLATOR DESIGN

Codes for building isolation system design, such as UBC and FEMA-273, provide detailed requirements for isolation system design, analysis and testing but do not provide detailed design requirements for the design of the devices themselves. Most projects have adopted provisions of bridge codes for the device design as bridges have always been supported on bearings and so contain specific requirements for sliding bearings and elastomeric bearings. As AASHTO incorporates design requirements for using these bearings as seismic isolators this has been the code of choice for this aspect on most projects.

In the 1999 revision, the formulas for elastomeric bearing design are based on a total shear strain formulation, as in the 1991 edition, but with modifications. One of the major changes is the inclusion of bulk modulus effects on load capacity. We have always used the bulk modulus to calculate vertical stiffness but have not used it to calculate the shear strain due to compression. Its inclusion in AASHTO for calculating vertical load capacity is controversial as other codes (for example, AustRoads) explicitly state that the bulk modulus effect does not reduce the load capacity.

Figure 4-4 shows the difference in load capacity at earthquake displacements for elastomeric bearings designed using the 1991 and 1999 AASHTO load specifications. The plot is for typical isolators (10 mm layer thickness, area reduction factor of 0.5 and seismic shear strain of 150%). The load capacity is similar for smaller isolators (600 mm plan size or less) but the 1999 requirements reduce the load capacity for larger isolators such that for 1000 mm isolators the load capacity is only one-half that permitted by the earlier revision. Isolators of 1 m diameter or more are quite common for high seismic zones.

FIGURE 4-4 ELASTOMERIC BEARING LOAD CAPACITY



This change in load capacities has little effect on most projects but has a major impact on design for conditions of high vertical loads and high seismic displacements.

For example, the Berkeley Civic Center bearings were 970 mm diameter, designed to the 1991 AASHTO requirements. If we had used the 1999 provisions the diameter would need to be increased to 1175 mm. This 48% increase in plan area would require a corresponding increase in height to achieve the same flexibility. This would have made base isolation using LRBs impossible for this retrofit project as there were space restrictions.

These Berkeley bearings were successfully tested beyond the design displacement to a point close to the design limit of the 1991 code. This implies a factor of safety of at least 2 for vertical loads relative to the 1999 AASHTO. This factor of safety is beyond what would normally be required for displacements based on an extreme MCE event.

If project specifications require compliance with 1999 AASHTO then we need to use the formulation for total shear strain that includes the bulk modulus. However, there does not seem enough evidence that designs excluding the bulk modulus are non-conservative for us to change our procedures for other projects for which compliance with this code is not mandatory.

5 SEISMIC INPUT

5.1 FORM OF SEISMIC INPUT

Earthquake loads are a dynamic phenomenon in that the ground movements that give rise to loads change with time. They are indeterminate in that every earthquake event will generate different ground motions and these motions will then be modified by the properties of the ground through which they travel. Structural engineers prefer a small number of defined loads so codes try to represent earthquake loads in a format more suited to design conditions. The codes generally specify seismic loads in three forms, in increasing order of complexity:

Equivalent static loads.

These are intended to represent an envelope of the story shears that will be generated by an earthquake with a given probability of occurrence. Most codes now derive these loads as a function of the structure (defined by period), the soil type on which it is founded and the seismic risk (defined by a zone factor). The static load is applied in a specified distribution, usually based on an assumption of inertia loads increasing linearly with height. This distribution is based on first mode response and may be modified to account for structural characteristics (for example, an additional load at the top level or use of a power function with height).

Base isolation modifies the dynamic characteristics of the structure and usually also adds damping. These effects are difficult to accommodate within the limitations of the static load procedure and so most codes impose severe limitations on the structures for which this procedure is permitted for isolated structures.

Response Spectrum

A response spectrum is a curve that plots the response of a single degree of freedom oscillator of varying period to a specific earthquake motion. Response spectra may plot the acceleration, velocity or displacement response. Spectra may be generated assuming various levels of viscous damping in the oscillator.

Codes specify response spectra which are a composite, or envelope, spectrum of all earthquakes that may contribute to the response at a specific site, where the site is defined by soil type, and zone factor. The code spectra are smooth and do not represent any single event.

A response spectrum analysis assumes that the response of the structure may be uncoupled into the individual modes. The response of each mode can be calculated by using the spectral acceleration at the period of the mode times a participation factor that defines the extent to which a particular mode contributes to the total response. The maximum response of all modes does not occur at the same time instant and so probabilistic methods are used to combine them, usually the Square Root of the Sum of the Square (SRSS) or, more recently, the Complete Quadratic Combination (CQC). The latter procedure takes account of the manner in which the response of closely spaced modes may be partially coupled and is considered more accurate than the SRSS method.

The uncoupling of modes is applicable only for linear elastic structures and so the response spectrum method of analysis cannot be used directly for most base isolated structures, although this restriction also applies in theory for yielding non-isolated structures. Most codes permit response spectrum analysis for a much wider range of isolated structures than the static load procedure. In practice, the isolation system is modeled as an equivalent elastic system and the damping is implemented by using the appropriate damped spectrum for the isolated modes.

The analyses described in Chapter 6 of these Guidelines suggest that this procedure may underestimate floor acceleration and overturning effects for non-linear systems by a large margin. It is recommended that this procedure not be used for design pending resolution of this issue.

Time History

Earthquake loads are generated in a building by the accelerations in the ground and so in theory a load specified as a time history of ground accelerations is the most accurate means of representing earthquake actions. Analysis procedures are available to compute the response of a structure to this type of load. The difficulty with implementing this procedure is that the form of the acceleration time history is unknown.

Recorded motions from past earthquakes provide information on the possible form of the ground acceleration records but every record is unique and so does not provide knowledge of the motion which may occur at the site from future earthquakes.

The time history analysis procedure cannot be applied by using composite, envelope motions, as can be done for the response spectrum procedure. Rather, multiple time histories that together provide a response that envelops the expected motion must be used. Seismology is unlikely ever to be able to predict with precision what motions will occur at a particular site and so multiple time histories are likely to be a feature of this procedure in the foreseeable future.

Codes provide some guidance in selecting and scaling earthquake motions but none as yet provide specific lists of earthquakes with scaling factors for a particular soil condition and seismic zone. The following sections discuss aspects of earthquake motions but each project will require individual selection of appropriate records.

5.2 RECORDED EARTHQUAKE MOTIONS

5.2.1 PRE-1971 MOTIONS

The major developments in practical base isolation systems occurred in the late 1960's and early 1970's and used the ground motions that had been recorded up to that date. An example of the data set available to those researchers is the Caltech SMARTS suite of motions (Strong Motion Accelerogram Record Transfer System) which contained 39 sets of three recorded components (two horizontal plus vertical) from earthquakes between the 1933 Long Beach event and the 1971 San Fernando earthquake.

A set of these records was selected for processing, excluding records from upper floors of buildings and the Pacoima Dam record from San Fernando, which included specific site effects. Response spectra were generated from the remaining 27 records, using each of the two horizontal components, and the average values over all 54 components calculated. The envelope and mean 5% damped acceleration spectra are shown in Figure 5-1 and the equivalent 5% damped displacement spectra in Figure 5-2.

A curve proportional to 1/T fits both the acceleration and the displacement spectra for periods of 0.5 seconds and longer quite well, as listed in Table 5-1:

- If it is assumed that the acceleration is inversely proportional to T for periods of 0.5 seconds and longer, the equation for the acceleration coefficient is $S_a = C_0/T$. The coefficient C_0 can be calculated from the acceleration at 0.5 seconds as $C_0 = 0.5 \times 0.278 = 0.139$. The accelerations at periods of 2.0, 2.5 and 3.0 seconds calculated as $S_a = 0.139/T$ match the actual average spectrum accelerations very well.

- The spectral displacements is related to the spectral acceleration as $S_d = S_a g T^2/4\pi^2$. For mm units, g = 9810 mm/sec^2 and so $S_d = 248.5 S_a T^2$. Substituting $S_a = 0.139/T$ provides for an equation for the spectral displacement $S_d = 34.5$ T, in mm units. The values are listed in Table 5-1 and again provide a very close match to the calculated average displacements.

These results show that the code seismic load coefficients, defined as inversely proportional to the period, had a sound basis in terms of reflecting the characteristics of actual recorded earthquakes. Figures 5-3 and 5-4, from the 1940 El Centro and 1952 Taft earthquake respectively, are typical of the form of the spectra of the earlier earthquakes. For medium to long periods (1 second to 4 seconds) the accelerations reduced with increased period and the displacement increased with increasing period. However, as discussed in the following sections later earthquake records have not shown this same trend.

TABLE 5-1 AVERAGE 5% DAMPED SPECTRUM VALUES

	Period 0.5 Seconds	**Period 2.0 Seconds**	**Period 2.5 Seconds**	**Period 3.0 Seconds**
Acceleration (g)				
Average Values	0.278	0.074	0.057	0.048
Calculated as 0.139/T	0.278	0.070	0.056	0.046
Displacement (mm)				
Average Values	17	73	89	106
Calculated as 34.5T	17	69	86	104

Figure 5-1 SMARTS 5% damped accleration spectra

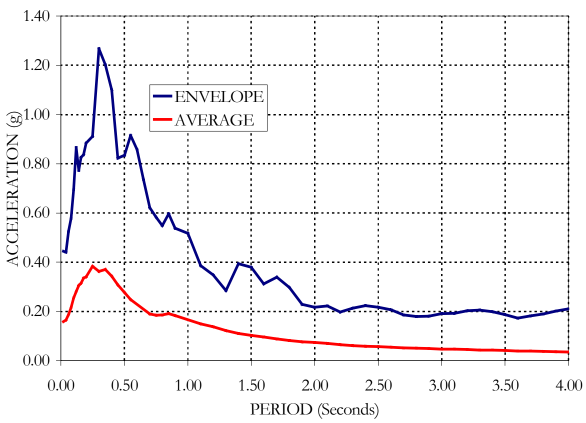


Figure 5-2 SMARTS 5% Damped Displacement Spectra

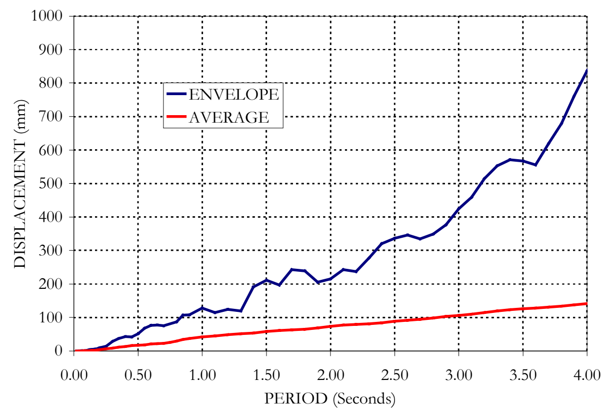
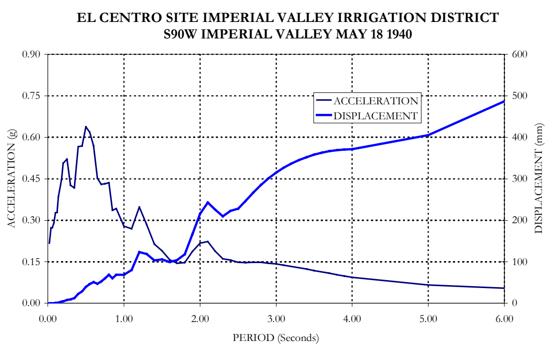


Figure 5-3 1940 El Centro Earthquake



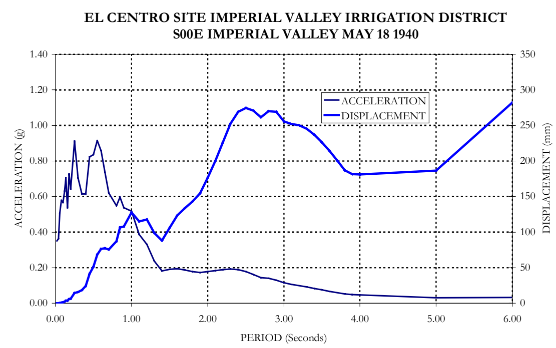
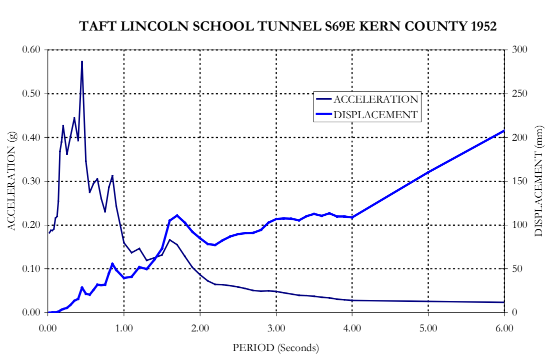
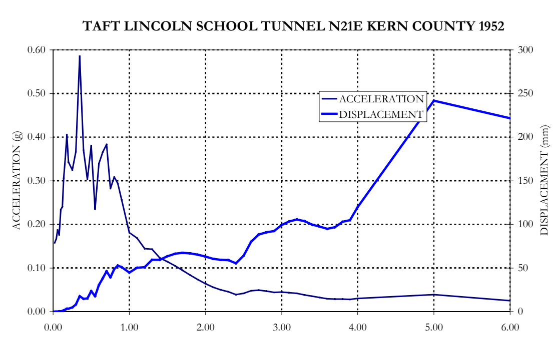


Figure 5-4 1952 Kern County Earthquake





5.2.2 Post-1971 Motions

Since 1971 the number of seismic arrays for recording ground motions has greatly increased and so there is an ever increasing database of earthquake records. As more records are obtained it has become apparent that there are far more variations in earthquake records than previously assumed. In particular, ground accelerations are much higher and near fault effects have modified the form of the spectra for long period motions.

Figures 5-5 to 5-10, each of which are 5% damped spectra of the two horizontal components for a particular earthquake, illustrate some of these effects:

- The 1979 El Centro event was recorded by a series of accelerographs that straddled the fault. Figure 5-5 shows the spectra of Array 6, less than 2 km from the fault. This shows near fault effects in the form of a spectral peak between 2 seconds and 3 seconds and a spectral displacement that exceeded 1 m for a period of 3.5 seconds. For this record, an isolation system would perform best with a period of 2 seconds or less. If the period increased beyond two seconds, both the acceleration and the displacement would increase.

- The 1985 Mexico City earthquake caused resonance at the characteristic site period of 2 seconds, as shown clearly in the spectra in Figure 5.6. An isolated structure on this type of site would be counter-effective and cause damaging motions in the structure.

- The 1989 Loma Prieta earthquake produced a number of records on both stiff and soft sites. Figure 5-7 shows a stiff site record. This record shows the characteristics of decreasing acceleration with period but the stronger component has a constant displacement for periods between 2 seconds and 4 seconds. Within this range, isolation system flexibility could be increased to reduce accelerations with no penalty of increased displacements.

- The 1992 Landers earthquake produced records with extreme short period spectral accelerations (Figure 5-8), exceeding 3g for the 5% damped spectra, and constant acceleration in the 2 second to 4 second range for the 270° component. For this type of record isolation would be very effective for short period buildings but the optimum isolation period would not exceed 2 seconds. For longer periods the displacement would increase for no benefit of reduced accelerations.

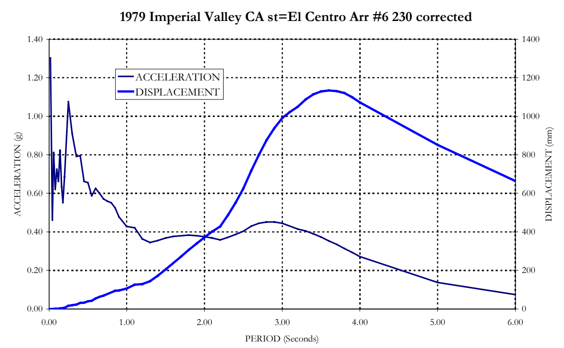
- The Sepulveda VA record of the 1994 Northridge earthquake, Figure 5-9, produced very high short period spectral accelerations, exceeding 2.5g, but the 360° component also had a secondary peak at about 2 seconds. For this component, the displacement would increase extremely rapidly for an isolated period exceeding 2 seconds.

- The Sylmar County Hospital record, also from the 1994 Northridge earthquake (Figure 5-10) also produced short period spectral accelerations exceeding 2.5g for one component. This record was unusual in that both components produced very high spectral accelerations at longer periods, exceeding 0.5g for 2 second periods. An isolation system tailored for this earthquake would use an isolated period exceeding 3 seconds as beyond this point both displacements and accelerations decrease with increasing period.

One common factor to all these earthquakes is that the particular characteristics of each earthquake suggest an optimum isolation system for that earthquake. However, an optimum system selected on the basis of one earthquake would almost certainly not be optimal for all, or any, of the other earthquakes.

Code requirements for time history selection require use of records appropriate to fault proximity and so often one or more records similar to those shown in Figure 5-5 to 5-10 will be used for a project. The manner of scaling specified by codes such as UBC and FEMA-273 also result in relatively large scaling factors. Naeim and Kelly [1999] discuss this in some detail.

FIGURE 5-5 1979 EL CENTRO EARTHQUAKE : BONDS CORNER RECORD



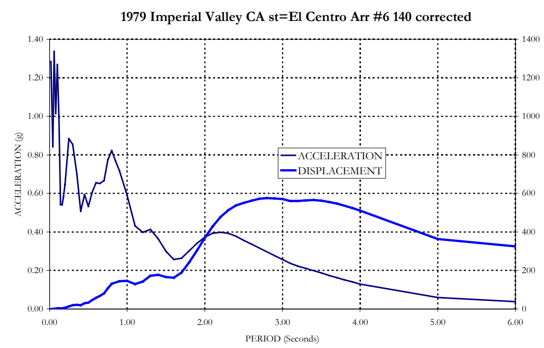
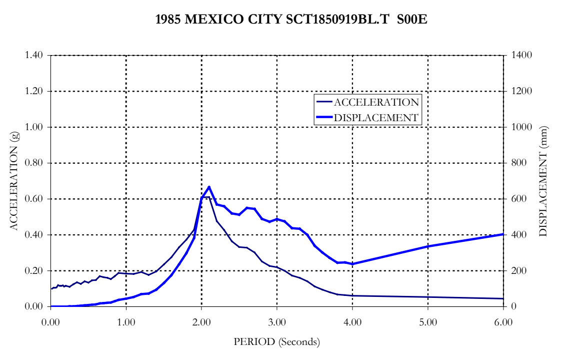


FIGURE 5-6 1985 MEXICO CITY EARTHQUAKE



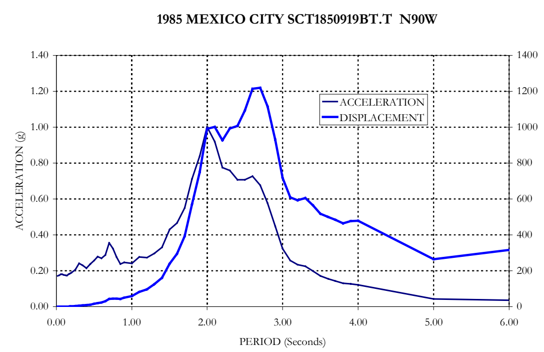
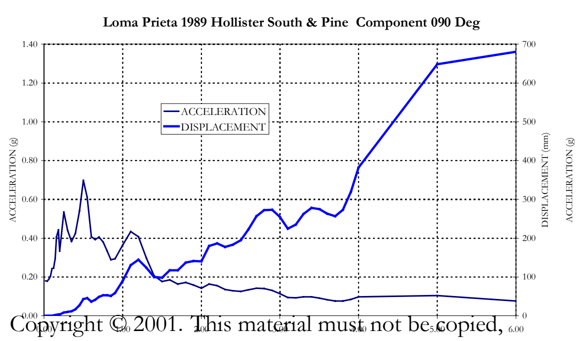


FIGURE 5-7 1989 LOMA PRIETA EARTHQUAKE



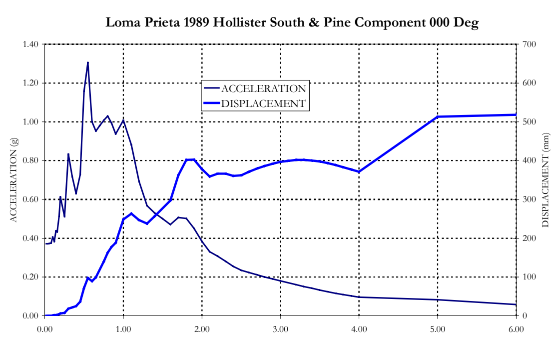


FIGURE 5-8 1992 LANDERS EARTHQUAKE

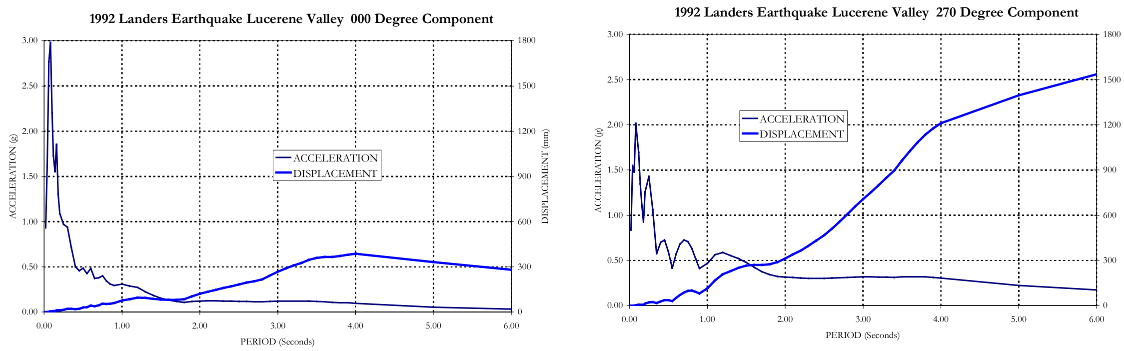


FIGURE 5-9 1994 NORTHRIDGE EARTHQUAKE

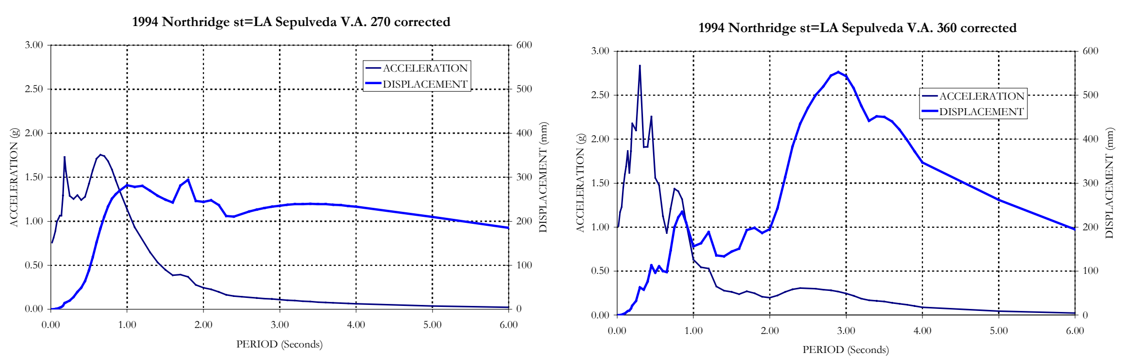
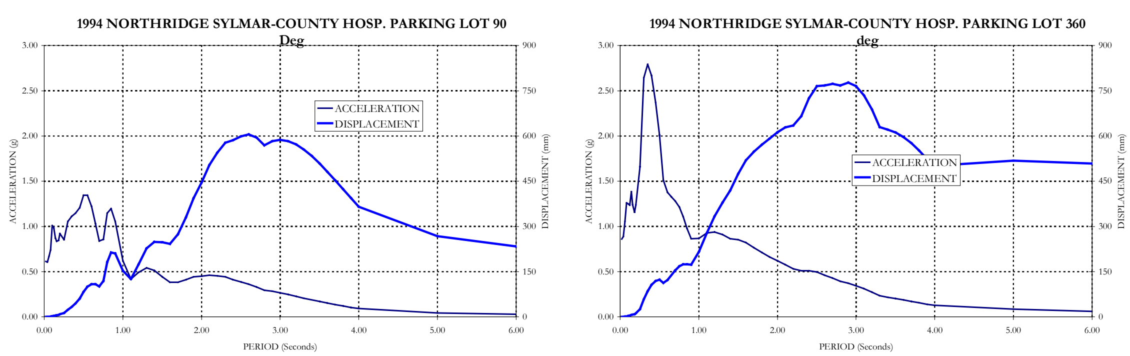


FIGURE 5-10 1994 NORTHRIDGE EARTHQUAKE

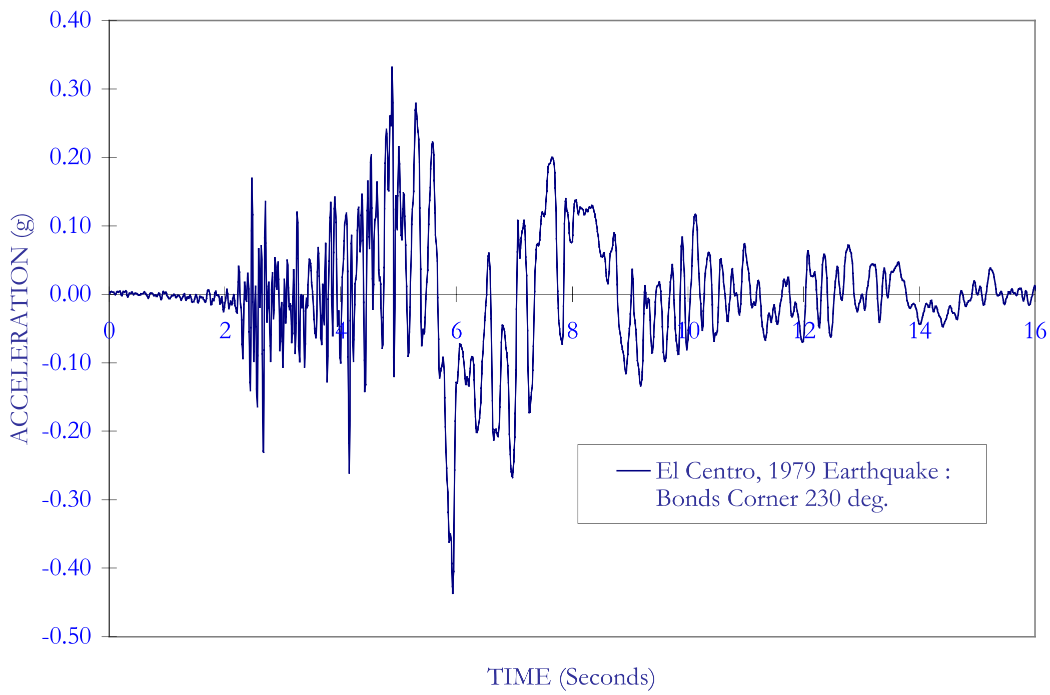


5.3 Near Fault Effects

Near fault effects cause large velocity pulses close to the fault rupture. Effects are greatest within 1 km of the rupture but extend out to 10 km. The UBC requires that near fault effects be included by increasing the seismic loads by factors of up to 1.5, depending on the distance to the nearest active fault and the magnitude of earthquake the fault is capable of producing. The current edition of the UBC does not require that this effect be included in the design of non-isolated buildings.

There has been some research in New Zealand on this effect and recent projects for essential buildings have included time histories reflecting near fault effects. Figure 5-11 shows one such record used for the Parliament project. Between 6 and 9 seconds relatively large accelerations are sustained for long periods of time, causing high velocities and displacements in structures in the medium period range of 1.5 to 3.0 seconds. This type of accelerogram will affect a wide range of structures, not just isolated buildings.

Figure 5-11 Acceleration Record With Near Fault Characteristics



There is a need for data on how this effect should be included in seismic design for New Zealand locations.

5.4 VARIATIONS IN DISPLACEMENTS

Figure 5-12 shows the variation in maximum displacements from 7 earthquakes each scaled according to UBC requirements for a site in California. Displacements range from 392 mm to 968 mm, with a mean of 692 mm. If at least 7 records are used, the UBC permits the mean value to be used to define the design quantities. The mean design displacement, 692 mm, is exceeded by 4 of the 7 earthquake records. These records, from Southern California, were selected because each contained near fault effects. Each has been scaled to the same amplitude at the isolated period. The scatter from these earthquakes is probably greater than would be obtained from similarly scaled records that do not include near fault effects.

Available options to the designer, all of which are acceptable in terms of UBC, are:

1. Use the mean of 7 records, a displacement of 692 mm.

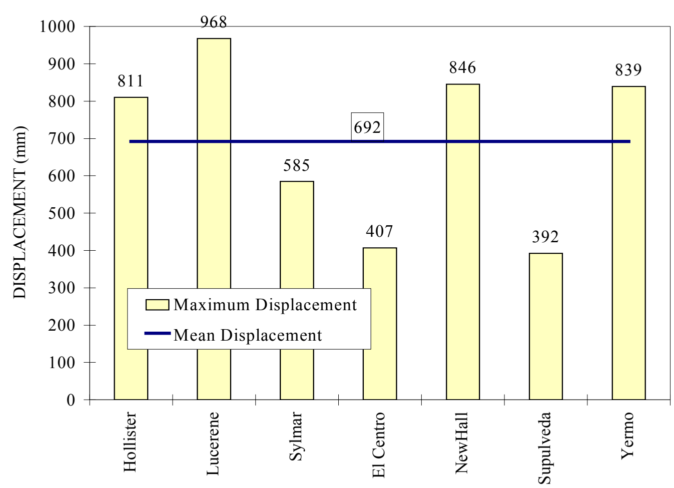
2. Select the three highest records and use the maximum response of these, 968 mm.

3. Select the three lowest records and use the maximum response of these, 585 mm.

It is difficult to rationalize a design decision where the majority of earthquakes will produce displacements greater than the design values. However, the NZS4203 requirement of a minimum of 3 time histories could also be satisfied using the 3rd, 4th and 6th records from Figure 5-12, resulting in a design displacement of 585 mm as in option 3. above.

There is clearly a need to develop specific requirements for time histories to ensure that anomalies do not occur and that the probability of maximum displacements being exceeded is not too high. Neither NZS4203 nor UBC procedures currently ensure this.

FIGURE 5-12 : VARIATION BETWEEN EARTHQUAKES



One procedure, which has been used on several projects, is to use at least one frequency scaled earthquake in addition to the scaled, actual earthquakes. This ensures that the full frequency range of response is included in the analysis.

5.5 Time History Seismic Input

A major impediment to the implementation of seismic isolation is that the time history method is the only reliable method of accurately assessing performance but code requirements for selecting time histories result in much higher levels of input than alternative methods such as the response spectrum procedure.

Overly conservative seismic design input for base isolation not only results in added costs but also degrades the performance at the more likely, lower levels of earthquake. All practical isolation systems must be targeted for optimum performance at a specified level of earthquake. This is almost always for the maximum considered earthquake as the displacements at this level must be controlled. This results in a non-optimum system for all lower levels of earthquake.

It appears that we may be required to consider too low a probability of occurrence in the earthquake records codes require for isolation. We are assuming not only that the MCE magnitude of earthquake will occur but also that it will occur at a distance so as to produce near fault effects. If the probability of both these occurrences were calculated they may be lower than is customarily used to develop earthquake loads.

In the interim, we need to use records in accordance with the applicable code requirements. Wherever possible, we should get advice from the seismological consultant as to near fault effects and both the return period for earthquake magnitude and the probability of the site being subjected to near fault effects. We should also request that the seismologist provide appropriate time histories, with scaling factors, to use to represent both the DBE and MCE events.

5.6 Recommended Records for Time History Analysis

The best method of selecting time histories is to have the seismologist supply them. However, this option is not always available and, if not, some guidance can be obtained from codes as to means of selecting and scaling records.

The New Zealand code NZS4203 requires a minimum of 3 records but is non-explicit as to scaling:

> *Scaling shall be by a recognized method. Scaling shall be such that over the period range of interest for the structure being analyzed, the 5% damped spectrum of the earthquake records does not differ significantly from the design spectrum.*
>
> *The record shall contain at least 15 seconds of strong shaking or have a strong shaking duration of at least 5 times the fundamental period of the structure, which ever is greater.*

The UBC and FEMA-273 Guidelines are more explicit and generally follow the same requirements. These sources require a minimum of three pairs of time history components. If seven of more pairs are used then the average results can be used for design else maximum values must be used. The records are required to have appropriate magnitudes, fault distances and source mechanisms for the site. Simulated time histories are permitted.

The UBC provides an explicit method of scaling records:

*For each pair of horizontal components, the square root of the sum of the squares (SRSS) of the 5% damped spectrum shall be constructed. The motions shall be scaled such that the average value of the SRSS spectra does not fall below 1.3 times the 5% damped spectrum of the design basis earthquake by more than 10% for periods from 0.5*T_D *to 1.25*T_M.

In this definition, T_D is the period at the design displacement (DBE) and T_M the period at the maximum displacement (MCE).

To me, this requirement is unclear as to whether the *average value of the SRSS* is the average over all periods for each record or the average at each period over all records. Consensus seems to be for the latter (based on a BRANZ study group). If so, then the scaling factor for any particular record would depend on the other records selected for the data set.

The ATC-40 document provides 10 records identified as suitable candidates for sites distant from faults (Table 5-2) and 10 records for sites near to the fault (Table 5-3). These records are available on the C:\QUAKES\FARFAULT and C:\QUAKES\NEARFA directories respectively. Each directory contains a spreadsheet, ACCEL.XLS, which contains the 5% damped spectrum for each component of each record and has functions to compute scaling factors.

Each record has been formatted for use as input to ANSR-L. To use them, use the options for *User-Selected* earthquakes in ModelA. The file names for each record, in ANSR-L format, are given in the final columns of Tables 5-1 and 5-2. See the *Performance Based Evaluation* guidelines for further information on time histories and scaling.

Table 5-2 Records at Soil Sites > 10 km From Sources

No.	M	Year	Earthquake	Station	File
1	7.1	1949	Western Washington	Station 325	wwash.1
2	6.5	1954	Eureka, CA	Station 022	eureka.9
3	6.6	1971	San Fernando, CA	Station 241	sf241.2
4	6.6	1971	San Fernando, CA	Station 241	sf458.10
5	7.1	1989	Loma Prieta, CA	Hollister, Sth & Pine	holliste.5
6	7.1	1989	Loma Prieta, CA	Gilroy #2	gilroy#2.3
7	7.5	1992	Landers, CA	Yermo	yermo.4
8	7.5	1992	Landers, CA	Joshua Tree	joshua.6
9	6.7	1994	Northridge, CA	Moorpark	moorpark.8
10	6.7	1994	Northridge, CA	Century City LACC N	lacc_nor.7

Table 5-3 Records at Soil Sites Near Sources

No.	M	Year	Earthquake	Station	File
1	6.5	1949	Imperial Valley, CA	El Centro Array 6	ecarr6.8
2	6.5	1954	Imperial Valley, CA	El Centro Array 7	ecarr7.9
3	7.1	1971	Loma Prieta, CA	Corralitos	corralit.5
4	7.1	1971	Loma Prieta, CA	Capitola	capitola.6
5	6.9	1989	Cape Mendocino, CA	Petrolia	petrolia.10
6	6.7	1989	Northridge, CA	Newhall Fire Station	newhall.7
7	6.7	1992	Northridge, CA	Sylmar Hospital	sylmarh.4
8	6.7	1992	Northridge, CA	Sylmar Converter Stat.	sylmarc.2
9	6.7	1994	Northridge, CA	Sylmar Converter St E	sylmare.1
10	6.7	1994	Northridge, CA	Rinaldi Treatment Plant	rinaldi.3

6 EFFECT OF ISOLATION ON BUILDINGS

As discussed earlier, there a number of types of isolation system which provide the essential elements of (1) flexibility (2) damping and (3) rigidity under service loads. Other systems provide some of these characteristics and can be used in parallel with other components to provide a complete system. To provide some guidance in selecting systems for a particular project, three prototype buildings have been used to examine the response under seismic loads of five types of system, each with variations in characteristics.

An example is then provided of parametric studies that are performed to refine the system properties for a particular building.

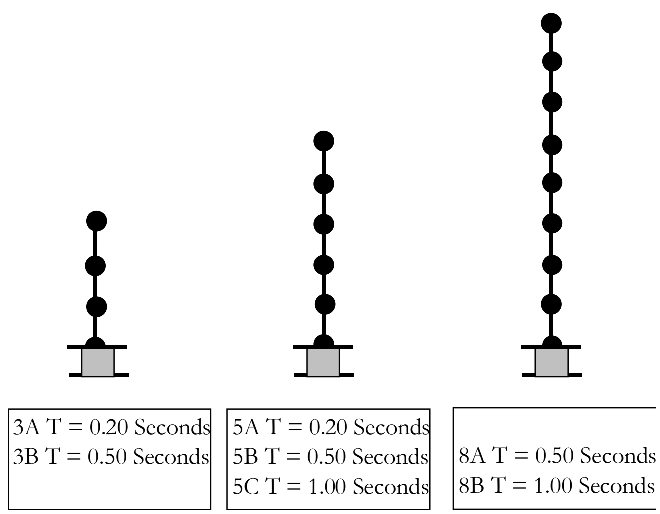
6.1 Prototype Buildings

The evaluations of prototype buildings in this section are intended to provide overall response characteristics of each system type. The buildings used were assumed linear elastic and the evaluation was not fully code compliant. The evaluation procedure used was consistent for all buildings and isolation systems and so provides a reasonable comparison between systems. However, it is not intended to provide final design displacements and forces for this particular seismic zone. Factors such as three-dimensional analysis, eccentricity and MCE factors would need to be included in a final design.

6.1.1 Building Configuration

Three simple shear buildings as shown in Figure 6-1 were used for the evaluation. Each building was assumed to have a total seismic weight of 5000 KN, distributed equally over all floors including the base floor. The assumption of equal total seismic weight allowed the same isolation systems to be used for all buildings. The buildings were also assumed to have equal story stiffness at all levels. For each building, the story stiffness was adjusted to provide a target fixed base period:

FIGURE 6-1 PROTOTYPE BUILDINGS



- Two variations of the three story building were used, with periods of 0.20 and 0.50 seconds respectively. The shorter period corresponds approximately to historic unreinforced masonry (URM) types buildings that tend to have large wall area and story stiffness. A three story building with a 0.50 second period would correspond to a stiff frame or perhaps a wall structural system.

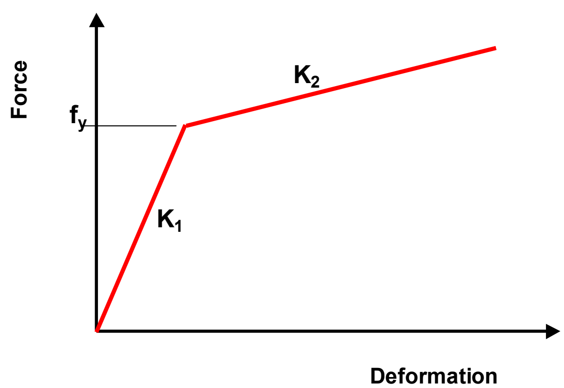
- The periods for the five story building were defined as 0.20, 0.50 and 1.00 seconds. This is the range of periods which would be encountered for this height of building for construction ranging from URM (0.2 seconds) to a moment frame (1.0 seconds).
- The eight story building was modeled with periods of 0.50 and 1.00 seconds, corresponding respectively to a stiff URM type building and a stiff moment frame, braced frame or structural wall building.

The height and period range of the prototypes have been restricted to low to mid-rise buildings with relatively short periods for their height. This is the type of building that is most likely to be a candidate for base isolation.

6.1.2 Design of Isolators

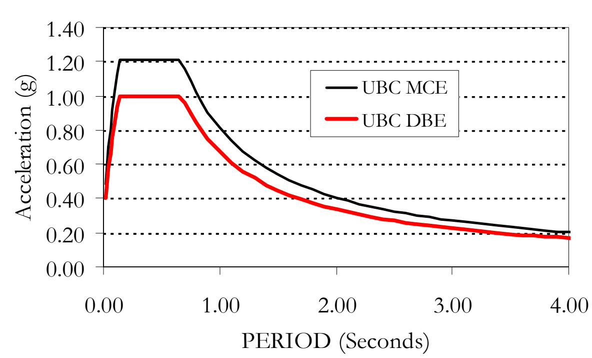
A total of 32 variations of five types of isolation system were used for the evaluation. The designs were completed using the Holmes UBC Template.xls spreadsheet which implements the design procedures described later in these guidelines. For most systems the solution procedure is iterative; a displacement is assumed, the effective period and damping is calculated at this displacement and the spectral displacement at this period and damping extracted. The displacement is then adjusted until the spectral displacement equals the trial displacement.

FIGURE 6-2 SYSTEM DEFINITION



Each system was designed to the point of defining the required stiffness and strength properties required for evaluation, as shown in Figure 6-2.

FIGURE 6-3 UBC DESIGN SPECTRUM



The design basis for the isolation system design was a UBC seismic load using the factors listed in Table 6-1. The site was assumed to be in the highest seismic zone, Z = 0.4, within 10 kms of a Type A fault. This produced the design spectrum shown in Figure 6-3. The UBC requires two levels of load, the Design Basis Earthquake (DBE) which is used to evaluate the structure and the Maximum Capable Earthquake (MCE, formerly the Maximum Credible Earthquake) which is used to obtain maximum isolator displacements.

Each system, other than the sliding bearings, was defined with effective periods of 1.5, 2.0, 2.5 and 3.0 seconds, which covers the usual range of isolation system period. Generally, the longer period isolation systems will be used with flexible structures. The sliding system was designed for a range of coefficients of friction.

TABLE 6-1 UBC DESIGN FACTORS

Seismic Zone Factor, Z	0.4	Table 16-I
Soil Profile Type	S_C	Table 16-J
Seismic Coefficient, C_A	0.400	Table 16-Q
Seismic Coefficient, C_V	0.672	Table 16-R
Near-Source Factor N_a	1.000	Table 16-S
Near-Source Factor N_v	1.200	Table 16-T
MCE Shaking Intensity M_MZN_a	0.484	
MCE Shaking Intensity M_MZN_v	0.581	
Seismic Source Type	A	Table 16-U
Distance to Known Source (km)	10.0	
MCE Response Coefficient, M_M	1.21	Table A-16-D
Lateral Force Coefficient, R_I	2.0	Table A-16-E
Fixed Base Lateral Force Coefficient, R	3.0	Table 16-N
Importance Factor, I	1.0	Table 16-K
Seismic Coefficient, C_{AM}	0.484	Table A-16-F
Seismic Coefficient, C_{VM}	0.813	Table A-16-G

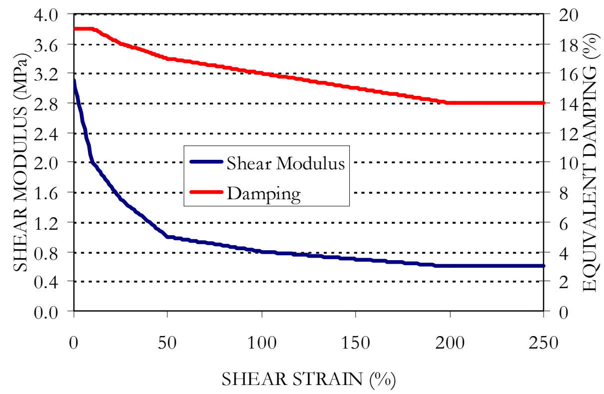
1. The ELASTIC system is an elastic spring with no damping. This type of system is not practical unless used in parallel with supplemental dampers as displacements will be large and the structure will move under service loads. However, it serves as a benchmark analysis to evaluate the effect of the damping in the other systems. This is modeled as a linear elastic spring with the yield level set very high.

2. The LRB is a lead rubber bearing. Variations were designed with three values of Q_d, corresponding to 0.05W, 0.075W and 0.010W. Q_d is the force intercept at zero displacement and defines the yield level of the isolator. For this type of bearing the effective damping is a function of period and Q_d and ranges from 8% to 37% for the devices considered here.

3. HDR is a high damping rubber system. There are a large number of high damping formulations available and each manufacturer typically provides a range of elastomers with varying hardness and damping values. The properties are a function of the applied shear strain. The properties used for this design were as plotted in Figure 6-4.

These properties represent a mid-range elastomer with a shear modulus of approximately 3 MPa at very low strains reducing to 0.75 MPa for a strain of 250%. The damping has a maximum value of 19% at low strains, reducing to 14% at 250% strain. Most elastomeric materials have strain-stiffening characteristics with the shear modulus increasing for strains exceeding about 250%. If the bearings are to work within this range then this stiffening has to be included in the design and evaluation.

FIGURE 6-4 HDR ELASTOMER PROPERTIES



The strain-dependent damping as plotted in Figure 6-4 is used to design the bearing. For analysis this is converted to an equivalent hysteresis shape. Although complex shapes may be required for final design, the analyses here used a simple bi-linear representation based on the approximations from FEMA-273. A yield displacement, Δ_y, is assumed at 0.05 to 0.10 times the rubber thickness and the intercept, Q, calculated from the maximum displacement and effective stiffness as:

$$Q = \frac{\pi \beta_{eff} \Delta^2}{2(\Delta - \Delta_y)}$$

The damping for these bearings varies over a narrow range of 15% to 19% for the isolator periods included here.

4. PTFE is a sliding bearing system. Sliding bearings generally comprise a sliding surface of a self-lubricating polytetrafluoroethylene (PTFE) surface sliding across a smooth, hard, noncorrosive mating surface such as stainless steel. (Teflon © is a trade name for a brand of PTFE). These bearings are modeled as rigid-perfectly plastic elements ($k_1 = \infty$, $K_2 = 0$). A range of coefficients of friction, m, was evaluated. The values of μ = 0.06, 0.09, 0.12 and 0.15 encompass the normal range of sliding coefficients. Actual sliding bearing coefficients of friction are a function of normal pressure and the velocity of sliding. For final analysis, use the special purpose ANSR-L element that includes this variability.

   For this type of isolator the coefficient of friction is the only variable and so design cannot target a specific period. The periods as designed are calculated based on the secant stiffness at the calculated seismic displacement. The hysteresis is a rectangle that provides optimum equivalent damping of $2/\pi$ = 63.7%.

5. FPS is a patented friction pendulum system, which is similar to the PTFE bearing but which has a spherical rather than flat sliding surface. The properties of this type of isolator are defined by the radius of curvature of the bowl, which defines the period, and the coefficient of friction. Two configurations were evaluated, using respectively coefficients of friction of 0.06 and 0.12. Bowl radii were set to provide the same range of periods as for the other isolator types. Equivalent viscous damping ranged from 9% to 40%, a similar range to the LRBs considered.

   These bearings are modeled as rigid-strain hardening elements ($k_1 = \infty$, $K_2 > 0$). As for the PTFE bearings, the evaluation procedure was approximate and did not consider variations in the coefficient of friction with pressure and velocity. A final design and evaluation would need to account for this.

Table 6-2 lists the variations considered in the evaluation and the hysteresis shape parameters used for modeling.

Figure 6-5 plots the hysteresis curves for all isolator types and variations included in this evaluation. The elastic isolators are the only type which have zero area under the hysteresis curve, and so zero equivalent viscous damping. The LRB and HDR isolators produce a bi-linear force displacement function with an elastic stiffness and a yielded stiffness. The PTFE and FPS bearings are rigid until the slip force is reached and the stiffness then reduces to zero (PTFE) or a positive value (FPS).

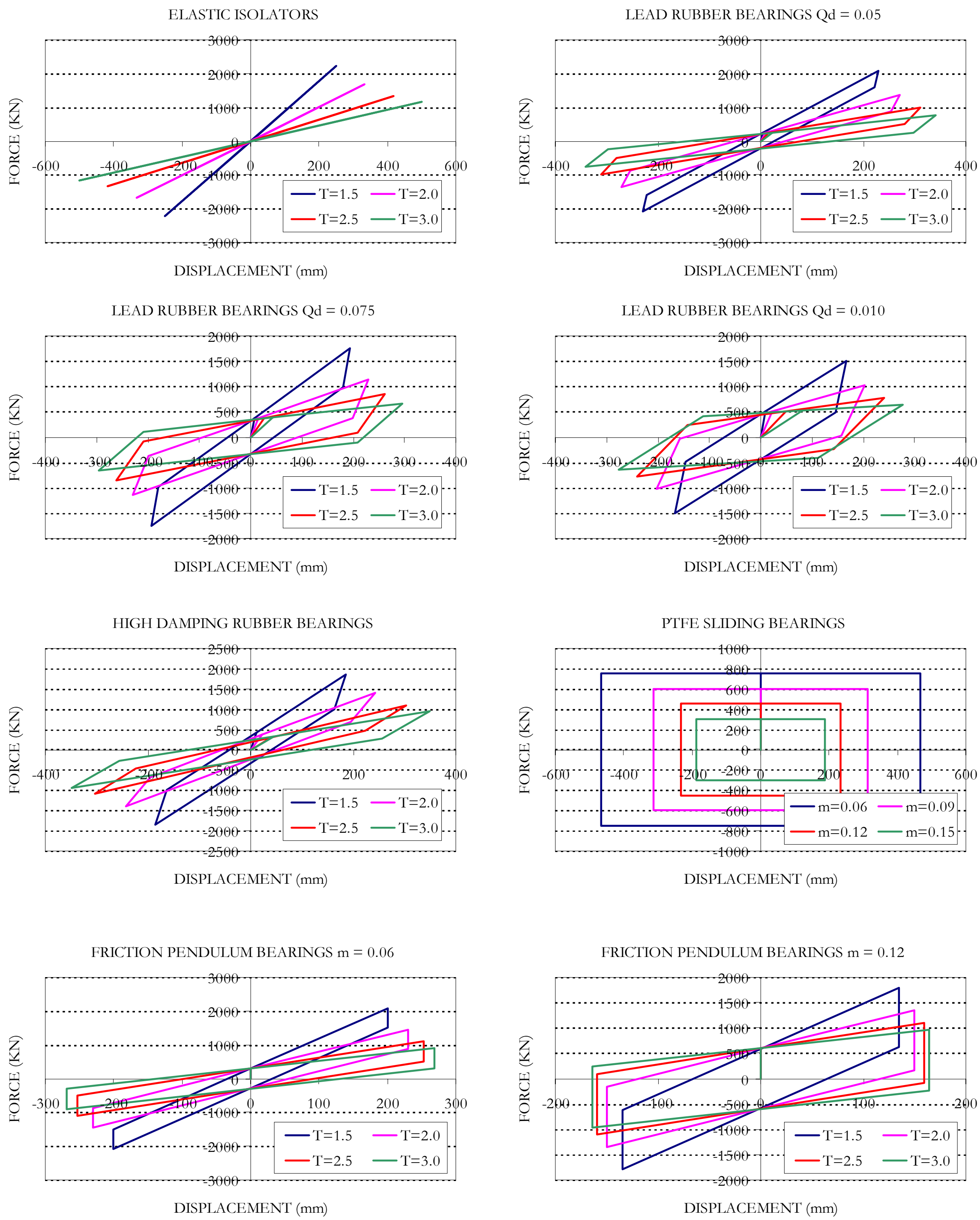
It is important to note that these designs are not necessarily optimum designs for a particular isolation system type and in fact almost surely are not optimal. In particular, the HDR and FPS bearings have proprietary and/or patented features that need to be taken into account in final design. You should get technical advice from the manufacturer for these types of bearing.

The UBC requires that isolators without a restoring force be designed for a displacement three times the calculated displacement. A system with a restoring force is defined as one in which the force at the design displacement is at least 0.025W greater than the force at 0.5 times the design displacement. This can be checked from the values in Table 6-2 as $R = (k_2 \times 0.5\Delta)/W$. The only isolators which do not have a restoring force are the LRB with $Q_d = 0.100$ and an isolated period of 3 seconds and all the sliding (PTFE) isolation systems.

Table 6-2 Isolation System Variations

System	Variation	Isolated Period (Seconds)	β (%)	Δ (mm)	C	k_1 (KN/mm)	k_2 (KN/mm)	f_y
NONE		0.0	0%	0		100000	0	100000
ELASTIC		1.5	5%	250	0.447	8.94	8.94	100000
ELASTIC		2.0	5%	334	0.336	5.03	5.03	100000
ELASTIC		2.5	5%	417	0.269	3.22	3.22	100000
ELASTIC		3.0	5%	501	0.234	2.34	2.34	100000
LRB	Q_d=0.050	1.5	8%	230	0.417	62.83	7.98	287
LRB		2.0	11%	272	0.273	32.82	4.10	287
LRB		2.5	15%	310	0.199	19.87	2.40	287
LRB		3.0	20%	342	0.153	12.82	1.49	287
LRB	Q_d=0.075	1.5	13%	194	0.349	60.52	7.05	426
LRB		2.0	20%	229	0.227	28.94	3.30	426
LRB		2.5	26%	262	0.168	15.96	1.77	426
LRB		3.0	31%	295	0.134	9.81	0.96	426
LRB	Q_d=0.100	1.5	20%	167	0.299	55.10	5.96	562
LRB		2.0	28%	203	0.206	24.72	2.60	562
LRB		2.5	33%	240	0.156	11.56	1.14	562
LRB		3.0	37%	276	0.128	6.83	0.41	562
HDR		1.5	15%	186	0.184	45.28	7.62	514
HDR		2.0	16%	242	0.140	20.06	4.28	462
HDR		2.5	17%	303	0.110	10.34	2.60	414
HDR		3.0	19%	348	0.094	9.02	1.74	414
PTFE	μ=0.06	5.6	64%	467	0.060	500	0	300
PTFE	μ=0.09	3.7	64%	312	0.090	500	0	450
PTFE	μ=0.12	2.8	64%	234	0.120	500	0	600
PTFE	μ=0.15	2.2	64%	187	0.150	500	0	750
FPS	μ=0.06	1.5	9%	200	0.417	500	8.94	300
FPS		2.0	13%	231	0.292	500	5.03	300
FPS		2.5	17%	253	0.223	500	3.22	300
FPS		3.0	21%	269	0.180	500	2.24	300
FPS	μ=0.12	1.5	21%	135	0.359	500	8.94	600
FPS		2.0	28%	150	0.270	500	5.03	600
FPS		2.5	34%	159	0.222	500	3.22	600
FPS		3.0	40%	164	0.193	500	2.24	600

Figure 6-5 Isolation System Hysteresis



6.1.3 EVALUATION PROCEDURE

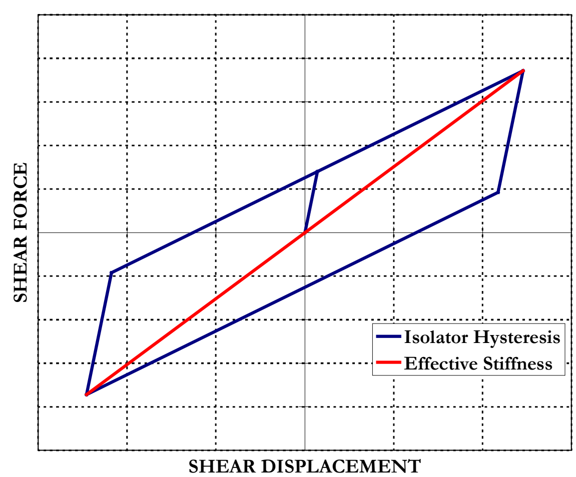
As discussed later, the procedures for evaluating isolated structures are, in increasing order of complexity, (1) static analysis, (2) response spectrum analysis and (3) time-history analysis. The static procedure is permitted for only a very limited range of buildings and isolation systems and so the response spectrum and time-history analyses are the most commonly used methods. There are some restrictions on the response spectrum method of analysis that may preclude some buildings and/or systems although this is unusual. The time-history method can be used without restriction. As the same model can be used for both types of analysis it is often preferable to do both so as to provide a check on response predictions.

In theory the response spectrum analysis is simpler to evaluate as it provides a single set of results for a single spectrum for each earthquake level and eccentricity. The time-history method produces a set of results at every time step for at least three earthquake records, and often for seven earthquake records. In practice, our output processing spreadsheets produce results in the same format for the two procedures and so this is not as issue. Also, the response spectrum procedure is based on an effective stiffness formulation and so is usually an iterative process. The effective stiffness must be estimated, based on estimated displacements, and then adjusted depending on the results of the analysis.

The evaluations here are based on both the response spectrum and the time-history method of analysis, respectively termed the Linear Dynamic Procedure (LDP) and the Non-Linear Dynamic Procedure (NDP) in FEMA-273.

6.1.3.1 Response Spectrum Analysis

FIGURE 6-6 EFFECTIVE STIFFNESS



The response spectrum analysis follows the usual procedure for this method of analysis with two modifications to account for the isolation system:

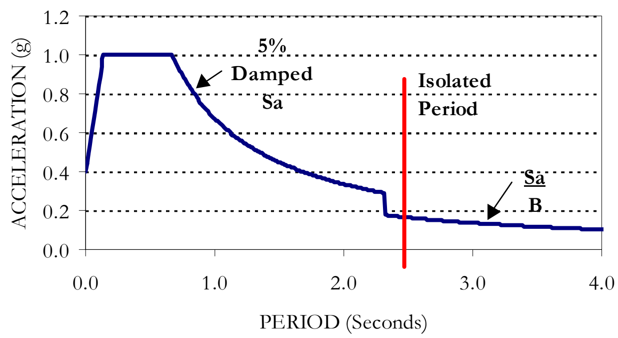
1. Springs are modeled to connect the base level of the structure to the ground. These springs have the effective stiffness of the isolators. For most isolator systems, this stiffness is a function of displacement – see Figure 6-6.

2. The response spectrum is modified to account for the damping provided in the isolated modes. Some analysis

programs (for example, ETABS) allow spectra for varying damping to be provided, otherwise the 5% damped spectrum can be modified to use a composite spectra which is reduced by the B factor in the isolated modes – see Figure 6-7.

More detail for the response spectrum analysis based on effective stiffness and equivalent viscous damping is provided in Chapter 10 of these guidelines.

FIGURE 6-7 COMPOSITE RESPONSE SPECTRUM

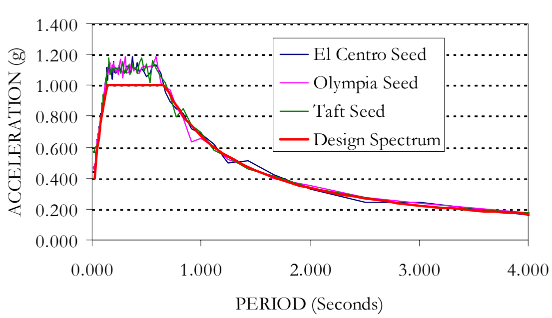


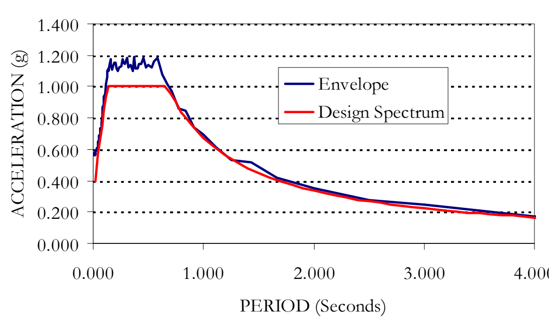
6.1.3.2 Time History Analysis

Each building and isolation system combination was evaluated for three earthquake records, the minimum number required by most codes. The record selection was not fully code compliant in that only a single component was applied to a two-dimensional model and the records selected were frequency scaled to match the design spectrum, as shown in Figure 6-8.

The frequency scaled records were chosen as these analyses are intended to compare isolation systems and analysis methods rather than obtain design values. The time history selection procedure specified by most codes result in seismic input which exceeds the response spectrum values and so would produce higher results than those reported here.

FIGURE 6-8 5% DAMPED SPECTRA OF 3 EARTHQUAKE RECORDS





Each building model and damping system configuration was analyzed for the 20 second duration of each record at a time step of 0.01 seconds. At each time step the accelerations and displacements at each level were saved as were the shear forces in each story. These values were then processed to provide isolator displacements and shear forces, structural drifts and total overturning moments.

6.1.4 Comparison with Design Procedure

The isolator performance parameters are the shear force coefficient, C, (the maximum isolator force normalized by the weight of the structure) and the isolator displacement, Δ. The design procedure estimates these quantities based on a single mass assumption – see Table 6-2.

6.1.4.1 Response Spectrum Analysis

The response spectrum results divided by the design estimates are plotted in Figure 6-9. These values are the average over all buildings. Numerical results are listed in Table 6-3. A value of 100% in Figure 6-9 indicates that the analysis matched the design procedure, a value higher than 100% indicates that the time history provided a higher value than the design procedure.

The response spectrum analysis displacements and shear coefficients were consistently lower than the design procedure results with one exception. The results were lower by a relatively small amount. Both the shear coefficients and the displacements were generally from 0% to 10% lower than the predicted values. An exception was type H (High Damping Rubber) where the differences ranged from +10% to –20%. This is because the design for these types was based on tabulated viscous damping whereas the analysis was based on an equivalent hysteresis shape.

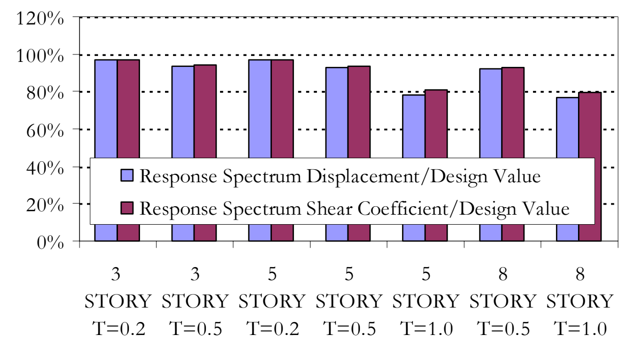
FIGURE 6-9 ISOLATOR RESULTS FROM RESPONSE SPECTRUM ANALYSIS COMPARED TO DESIGN

TABLE 6-3 ISOLATION SYSTEM PERFORMANCE (MAXIMUM OF ALL BUILDINGS, ALL EARTHQUAKES)

			Design Procedure		Response Spectrum Analysis		Time History Maximum of 3 Earthquakes	
System	Variation	Period (Seconds)	Δ (mm)	C	Δ (mm)	C	Δ (mm)	C
NONE					0	0.678		1.551
ELAST		1.5	250	0.447	236	0.423	309	0.552
ELAST		2.0	334	0.336	323	0.325	369	0.371
ELAST		2.5	417	0.269	409	0.263	434	0.279
ELAST		3.0	501	0.234	483	0.225	528	0.247
LRB	Q_d=0.050	1.5	230	0.417	206	0.379	144	0.280
LRB		2.0	272	0.273	256	0.260	213	0.225
LRB		2.5	310	0.199	295	0.192	269	0.180
LRB		3.0	342	0.153	325	0.148	344	0.153
LRB	Q_d=0.075	1.5	194	0.349	175	0.322	140	0.272
LRB		2.0	229	0.227	212	0.216	195	0.204
LRB		2.5	262	0.168	248	0.164	258	0.167
LRB		3.0	295	0.134	278	0.130	332	0.141
LRB	Q_d=0.100	1.5	167	0.299	152	0.282	140	0.267
LRB		2.0	203	0.206	190	0.199	197	0.203
LRB		2.5	240	0.156	226	0.153	269	0.163
LRB		3.0	276	0.128	258	0.127	384	0.137
HDR		1.5	186	0.184	206	0.366	148	0.277
HDR		2.0	242	0.140	212	0.254	177	0.225
HDR		2.5	303	0.110	270	0.202	269	0.202
HDR		3.0	348	0.094	279	0.165	320	0.179
PTFE	μ=0.15	2.2	187	0.150	177	0.149	204	0.150
PTFE	μ=0.12	2.8	234	0.120	225	0.120	223	0.120
PTFE	μ=0.09	3.7	312	0.090	305	0.090	309	0.090
PTFE	μ=0.06	5.6	467	0.060			430	0.060
FPS	μ=0.06	1.5	200	0.359	179	0.328	124	0.280
FPS		2.0	231	0.270	216	0.255	160	0.221
FPS		2.5	253	0.222	239	0.213	199	0.188
FPS		3.0	269	0.193	259	0.188	228	0.162
FPS	μ=0.12	1.5	135	0.328	117	0.381	103	0.301
FPS		2.0	150	0.255	135	0.277	111	0.231
FPS		2.5	159	0.213	145	0.214	122	0.198
FPS		3.0	164	0.188	152	0.176	130	0.178

The close correlation between the two methods is not surprising as they are both based on the same concepts of effective stiffness and equivalent viscous damping. The main difference is that the design procedure assumes a rigid structure above the isolators whereas the response spectrum analysis includes the effect of building flexibility.

FIGURE 6-10 SPECTRUM RESULTS FOR LRB1 T=1.5 SECONDS



The effect of building flexibility is illustrated by Figure 6-10, which plots the ratio of response spectrum results to design procedure values for the lead rubber bearing (LRB 1) with a period of 1.5 seconds. Figure 6-9 shows that the average ratio for this system is 90% of the design values. However, Figure 6-10 shows that the ratio actually ranges from 97% for buildings with a period of 0.2 seconds to 77% for the building with a 1.0 second period.

As the building period increases the effects of building flexibility become more important and so the response spectrum values diverge from the design procedure results. The effects shown in Figure 6-10 tend to be consistent in that for all systems the base displacement and base shear coefficient was lower for the buildings with longer periods. The only exception was for the sliding systems (PTFE) where the shear coefficient remained constant, at a value equal to the coefficient of friction of the isolators.

6.1.4.2 Time History Analysis

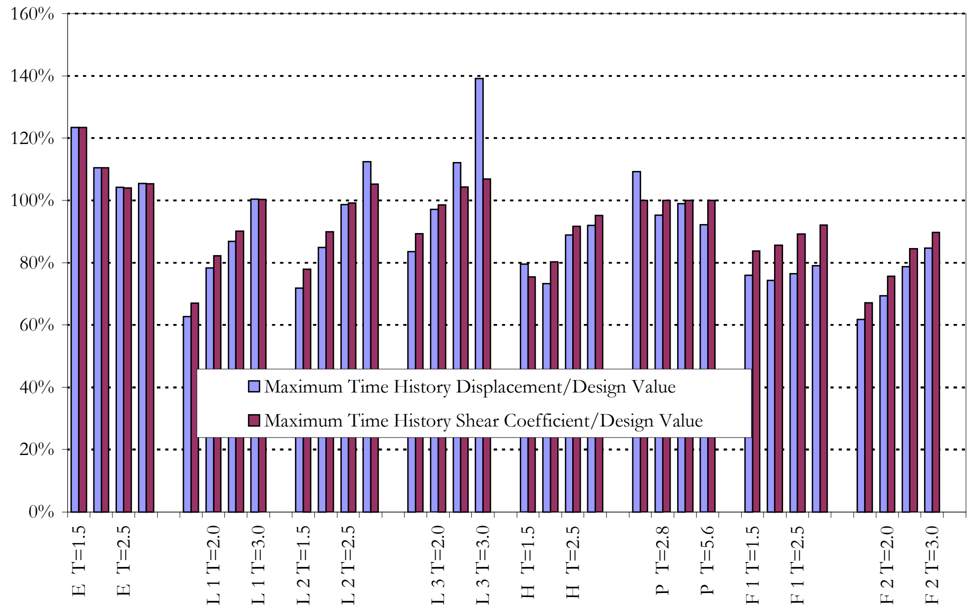
The ratios of the displacements and shear coefficients from the time history analysis to the values predicted by the design procedure are plotted in Figure 6-11. Two cases are plotted (a) the maximum values from the three time histories and (b) the average values from the three time histories. In each case, the values are averaged over the 7 building configurations.

The time history results varied from the design procedure predictions by a much greater amount than the response spectrum results, with discrepancies ranging from +40% to -40% for the maximum results and from +20% to –42% for the mean results. For the elastic systems the time history analysis results tended to be closer to the design procedure results as the period increased but this trend was reversed for all the other isolation system types. As the elastic system is the only one which does not use equivalent viscous damping this suggests that there are differences in response between hysteretic damping and a model using the viscous equivalent.

As the period increases for the hysteretic systems, the displacement also increases and the equivalent viscous damping decreases. The results in Figure 6-11 suggest that the viscous damping formulation is more accurate for large displacements than for small displacements.

Figure 6-11 Isolator Results from Time History Analysis Compared to Design

(A) Maximum From Time History Analysis



(B) Mean From Time History Analysis

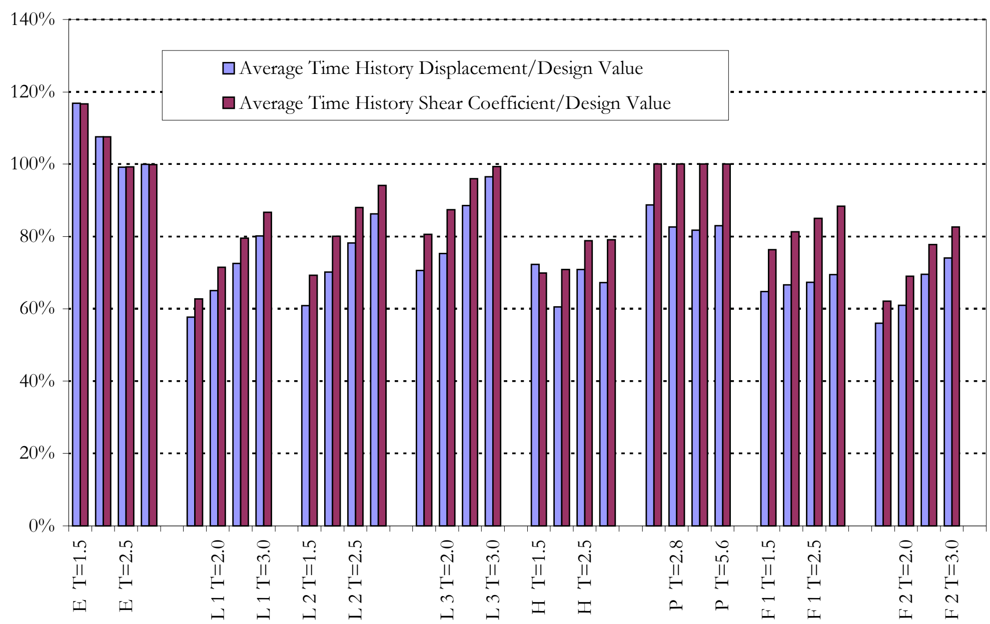


Figure 6-12 plots the ratios based on the maximum values from the three earthquakes compared to the design procedure values for the lead rubber bearing (LRB 1) with a period of 1.5 seconds (compare this figure with Figure 6-10 which provides the similar results from the response spectrum analysis). Figure 6-12 suggests that results are relatively insensitive to the period of the structure above the isolators. However, Figure 6-12, which plots the results for the individual earthquakes, shows that for EQ 1 and EQ 3 the results for the 1.0 second period structures are less than for the stiffer buildings, as occurred for the response spectrum analysis. However, this effect is masked by EQ 2 which produces a response for the 1.0 second period structures which is much higher than for the other buildings. This illustrates that time history response can vary considerably even for earthquake records which apparently provide very similar response spectra.

The mean time history results show that the design procedure generally provided a conservative estimate of isolation system performance except for the elastic isolation system, where the design procedure under-estimated displacements and shear forces, especially for short period isolation systems.

FIGURE 6-12 TIME HISTORY RESULTS FOR LRB1 T=1.5 SEC

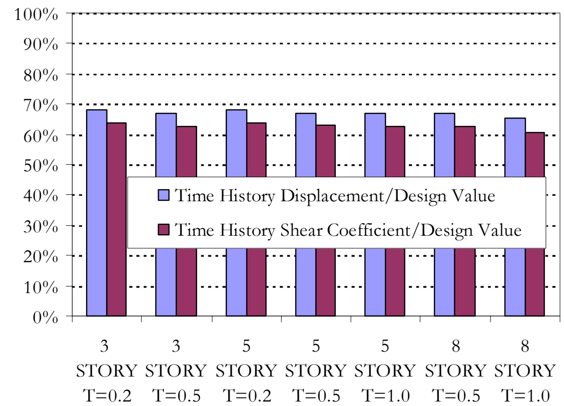
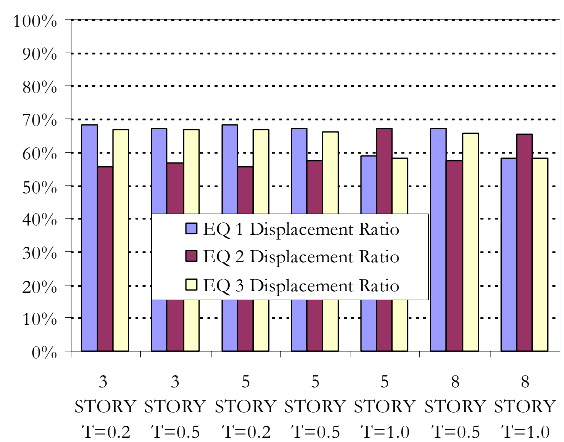


FIGURE 6-13 VARIATION BETWEEN EARTHQUAKES



6.1.5 ISOLATION SYSTEM PERFORMANCE

The mean and maximum results from the three time histories were used above to compare displacements and base shear coefficients with the design procedure and the response spectrum procedure. For design, if three time histories are used then the maximum rather than the mean values are used. (Some codes permit mean values to be used for design if at least 7 earthquakes are used).

Table 6-3 listed the average isolation response over the 7 building configurations for each system. These results are plotted in Figures 6-14 and 6-15, which compare respectively the shear coefficients and displacements for each isolation system for both the response spectrum method and the time history method.

- The plots show that although both methods of analysis follow similar trends for most isolation systems, the response spectrum results are higher in many cases. This is consistent with the comparisons with the design procedure discussed earlier, where the time history tended to produce ratios that were lower than the response spectrum.

- For all isolation systems, the base shear coefficient decreases with increasing period and the displacement increases. This is the basic tradeoff for all isolation system design.

- The PTFE (sliding) bearings produce the smallest shear coefficients and the smallest displacements of all systems except the FPS. However, as these bearings do not have a restoring force the design displacements are required to be increased by a factor of 3. With this multiplier the PTFE displacements are higher than for all other isolator types.

- There are relatively small variations between the three types of Lead Rubber Bearings (LRB). For these systems the yield force is increased from 5% W to 7.5% W to 10% W for systems 1, 2 and 3 respectively. The LRB systems produce the smallest shear coefficients after the PTFE sliders.

- The two Friction Pendulum Systems (FPS) variations are the values of the coefficient of friction, 0.06 for Type 1 and 0.12 for Type 2. The increased coefficient of friction has little effect on the base shear coefficients but reduces displacements. The FPS with $\mu = 0.12$ produces the smallest displacements of any system.

There is no one optimum system, or isolated period, in terms of minimizing both base shear coefficient and displacement. This isolator performance in only one aspect in selecting a system, the performance of the structure above is usually of at least equal performance. This is examined in the following sections and then well-performing systems are identified in terms of parameters that may be important depending on project objectives.

FIGURE 6-14 ISOLATOR PERFORMANCE : BASE SHEAR COEFFICIENTS

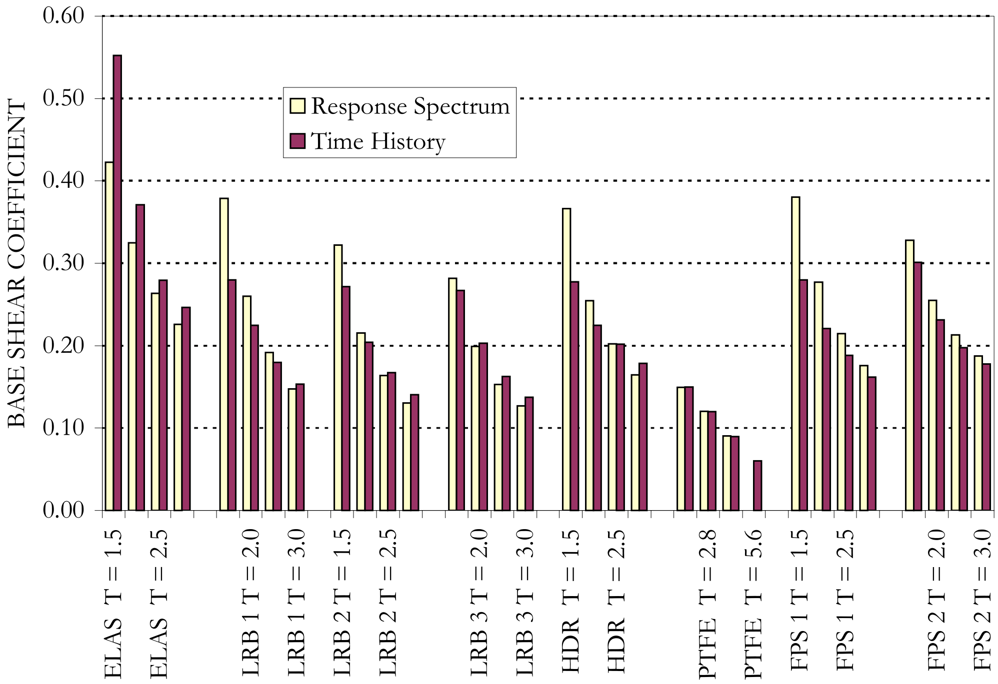
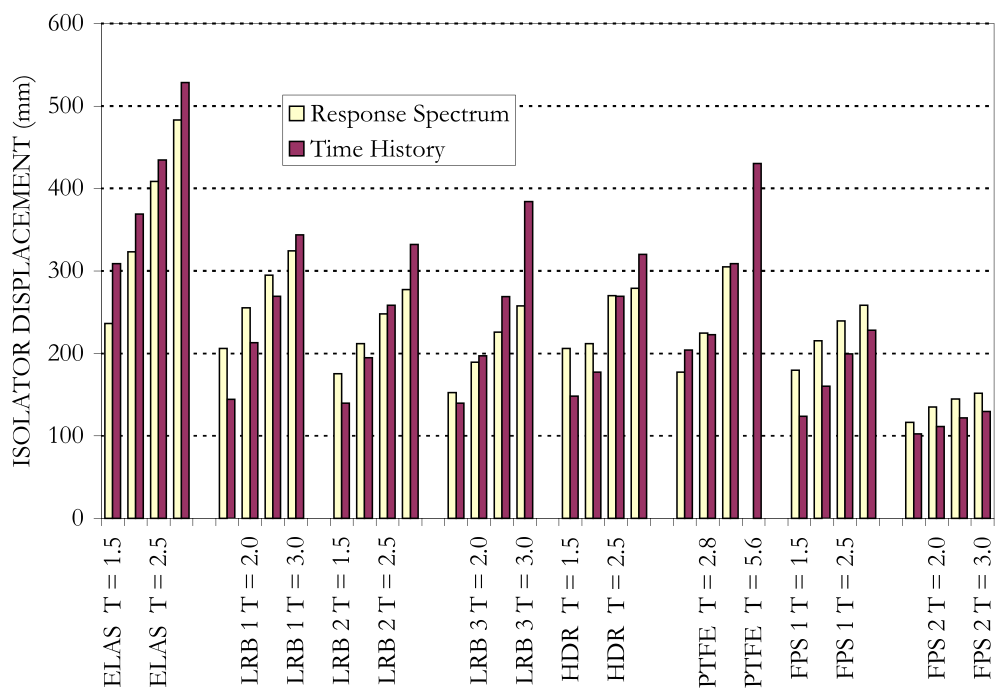


FIGURE 6-15 ISOLATOR PERFORMANCE : ISOLATOR DISPLACEMENTS



6.1.6 BUILDING INERTIA LOADS

The isolation system response provides the maximum base shear coefficient, that is the maximum simultaneous summation of the inertia forces from all levels above the isolator plane. The distribution of these inertia forces within the height of the structure defines the design shears at each level and the total overturning moments on the structure.

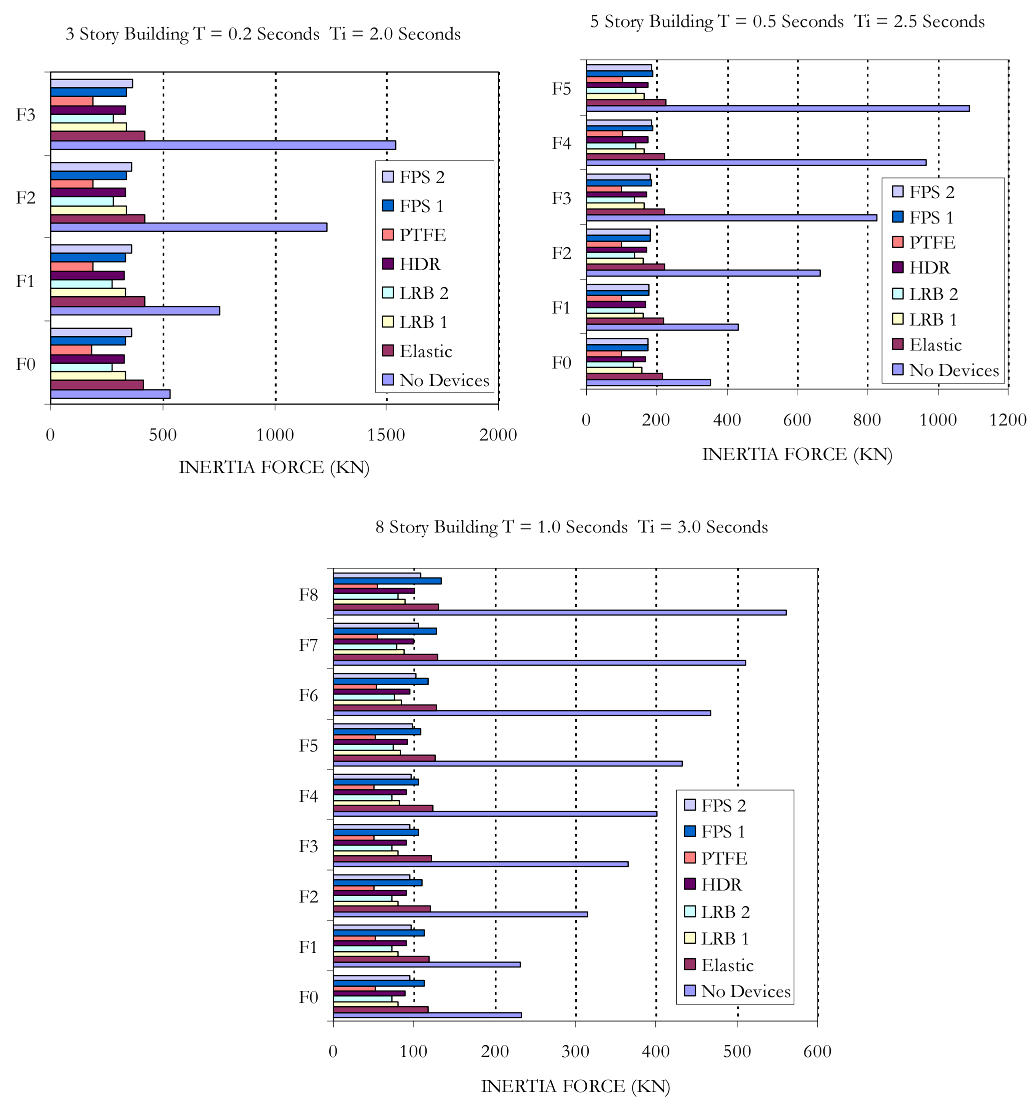
6.1.6.1 Response Spectrum Analysis

The inertia forces are obtained from the response spectrum analyses as the CQC of the individual modal responses, where modal inertia forces are the product of the spectral acceleration in that mode times the participation factor times the mass. Figure 6-16 plots these distributions for three building configurations, each for one isolator effective period. The combinations of building period and isolator period have been selected as typical values that would be used in practice.

Figure 6-16 shows that the inertia force distributions for the buildings without isolation demonstrate an approximately linear increase with height, compared to the triangular distribution assumed by most codes for a uniform building with no devices. Note that the fixed base buildings have an inertia force at the base level. This is because a rigid spring was used in place of the isolation system for these models and the base mass was included. As all modes were extracted this spring mode has acceleration equal to the ground acceleration and so generates an inertia force.

All isolation systems exhibit different distributions from the non-isolated building in that the inertia forces are almost constant with the height of the building for all buildings. Some systems show a slight increase in inertia force with height but this effect is small and so for all systems the response spectrum results suggest that a uniform distribution would best represent the inertia forces. As the following section describes, the results from the time history analysis were at variance with this assumption.

FIGURE 6-16 RESPONSE SPECTRUM INERTIA FORCES

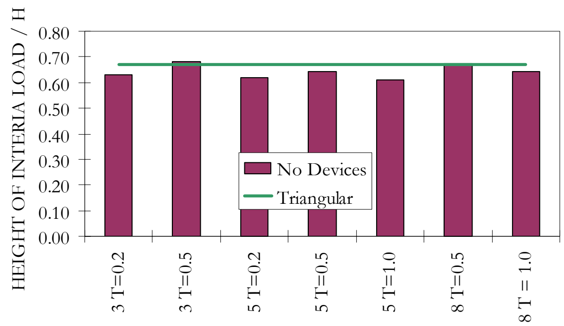


6.1.6.2 Time History Analysis

As discussed above, for a fixed base regular building most codes assumed that the distribution of inertia load is linear with height, a triangular distribution based on the assumption that first mode effects will dominate response. This distribution has an effective height at the centroid of the triangle, that is, two-thirds the building height above the base for structures with constant floor weights. A uniform distribution of inertia loads would have a centroid at one-half the height.

The effective height was calculated for each configuration analyzed by selecting the earthquake which produced the highest overturning moment about the base and calculating the effective height of application of inertia loads as $H_c = M/VH$, where M is the moment, V the base shear and H the height of the building. Figure 6-17 plots H_c for the fixed base configuration of each of the building models. Although there were some variations between buildings, these results show that the assumption of a triangular distribution is a reasonable approximation and produces a conservative overturning moment for most of the structures considered in this study.

FIGURE 6-17 HEIGHT OF INERTIA LOADS



An isolation system produces fundamental modes comprising almost entirely of deformations in the isolators with the structure above moving effectively as a rigid body with small deformations. With this type of mode shape it would be expected that the distribution of inertia load with height would be essentially linear with an effective height of application of one-half the total height, as was shown above for the response spectrum analysis results.

Figure 6-18 plots the effective heights of inertia loads, H_c, for the 8 isolation system variations considered in this study. Each plot contains the effective period variations for a particular device. Each plot has three horizontal lines

1. $H_c = 0.50$, a uniform distribution

2. $H_c = 0.67$, a triangular distribution

3. $H_c = 1.00$, a distribution with all inertia load concentrated at roof level.

Unexpectedly, few of the isolation systems provided a uniform distribution and in some, particularly the sliding (PTFE) systems, the effective height of application of the inertia forces exceeded the height of the structure by a large margin. Trends from these plots are:

- The elastic isolation systems provide inertia loads close to a uniform distribution except for the 1 second period buildings.

- The LRB systems provide a uniform distribution for the short period (0.2 seconds) buildings but a triangular distribution for the longer period buildings. As the isolation system yield level increases (going from LRB 1 to LRB 2 to LRB 3) the height of the centroid tends toward the top of the building.

- The HDR isolators exhibit similar characteristics to LRB 1, the lowest yield level.

- The PTFE (sliding) systems provide an effective height much higher than the building height for all variations and provide the most consistent results for all buildings. As the coefficient of friction decreases (increased T) the effective height increases.

- The FPS system with the lower coefficient of friction (FPS 1) provides a similar pattern to the PTFE systems but less extreme. The FPS system with the higher coefficient of friction (FPS 2) produces results closer to the LRB and HDR systems although the trends between buildings are different.

To investigate these results, the force distributions in Figures 6-19 to 6-21 have been generated. These are for the 3 story 0.2 second building with 2 second period isolators, the 5 story 0.5 second building with 2.5 second period isolators and the 8 story 1 second building with 3 second period isolators. These have been selected as typical configurations for the three building heights. For the fixed base case and each isolator case for these buildings two force distributions are plotted:

1. The force at each level when the maximum base shear force is recorded.

2. The force at each level when the maximum base overturning moment is recorded.

The distributions producing these two maximums are almost invariably at different times and in many cases are vastly different:

<u>3 Story Building (Figure 6-19)</u>

For the stiff building without isolators the distributions for both maximum shear and maximum moment are a similar shape with forces increasing approximately uniformly with height. The elastic isolators produce a very uniform distribution for both shear and moment as does the LRB with a low yield level (LRB 1). The LRB with the higher yield level (LRB 2) and the HDR isolators produce a uniform distribution for shear but the moment distribution shows a slight increase with height.

The PTFE (sliding) isolator distribution for shear is approximately linear with height, forming a triangular distribution. However, the distribution for maximum moment has very high shears at the top level with a sign change for forces at lower levels. This distribution provides a high moment relative to the base shear. This indicates that the building is "kicking back" at the base.

The FPS 2 isolators (higher coefficient of friction) produce a shear distribution that has the shape of an inverted triangle, with maximum inertia forces at the base and then reducing with height. The distribution producing the maximum moment has a similar form to the PTFE plots, exhibiting reversed signs on the inertia loads near the base. The FSP 1 isolators (lower coefficient of friction) also show this reversed sign for the moment distribution.

<u>5 Story Building (Figure 6-20)</u>

The distributions for the 5 story building follow the trends in the 3 story building but tend to be more exaggerated. The elastic isolators still produce uniform distributions but all others have distributions for moment which are weighted toward the top of the building, extremely so for the sliding bearings.

8 Story Building (Figure 6-21)

The 8 story buildings also follow the same trends but in this case even the elastic isolator moment distribution is tending toward a triangular distribution.

These results emphasize the limited application of a static force procedure for the analysis and design of base isolated buildings as the distributions vary widely from the assumed distributions. A static procedure based on a triangular distribution of inertia loads would be non-conservative for all systems in Figure 6-18 in which the height ratio exceeded 0.67. This applies to about 25% of the systems considered, including all the flat sliding systems (PTFE).

FIGURE 6-18 EFFECTIVE HEIGHT OF INERTIA LOADS FOR ISOLATION SYSTEMS

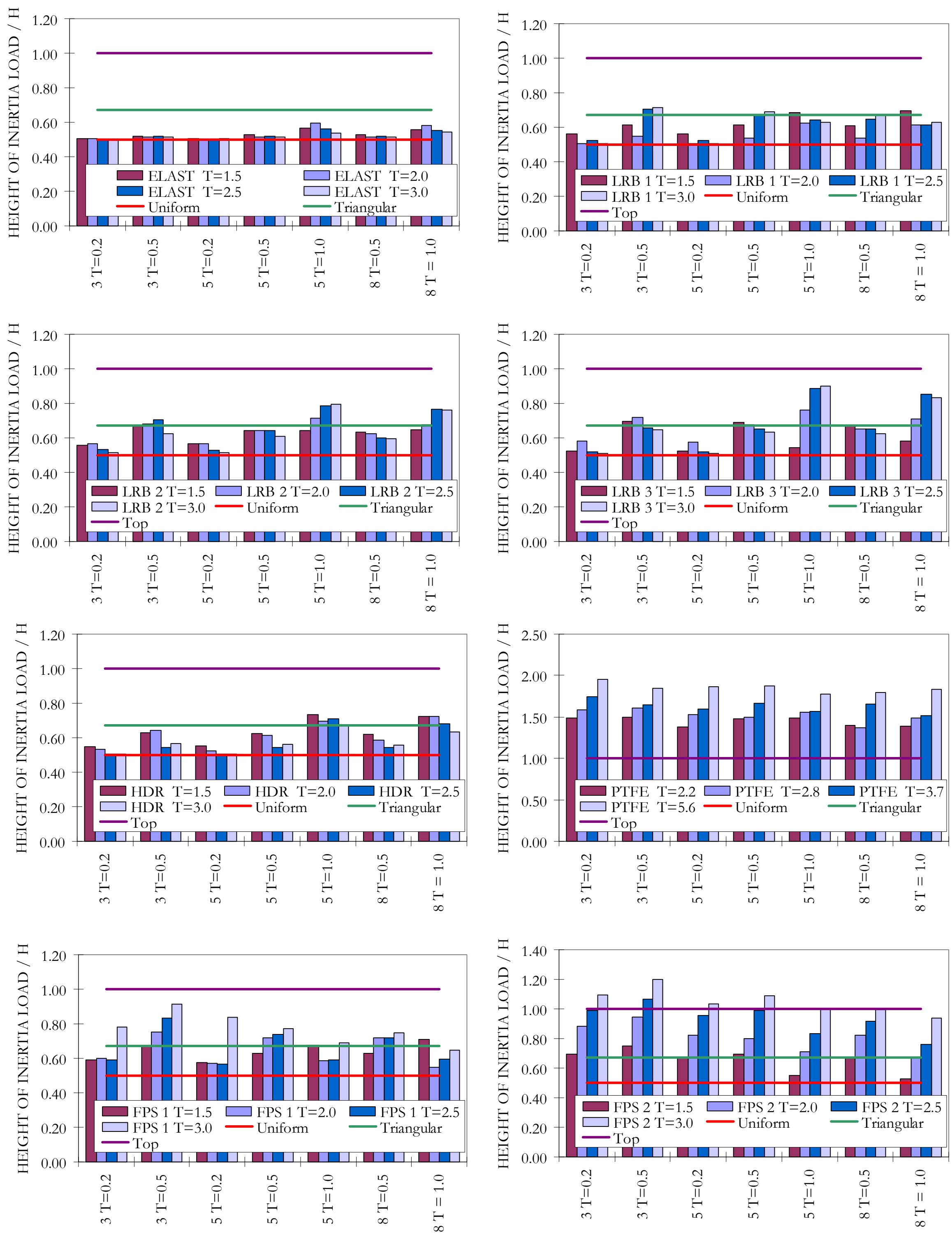


FIGURE 6-19 TIME HISTORY INERTIA FORCES : 3 STORY BUILDING T = 0.2 SECONDS

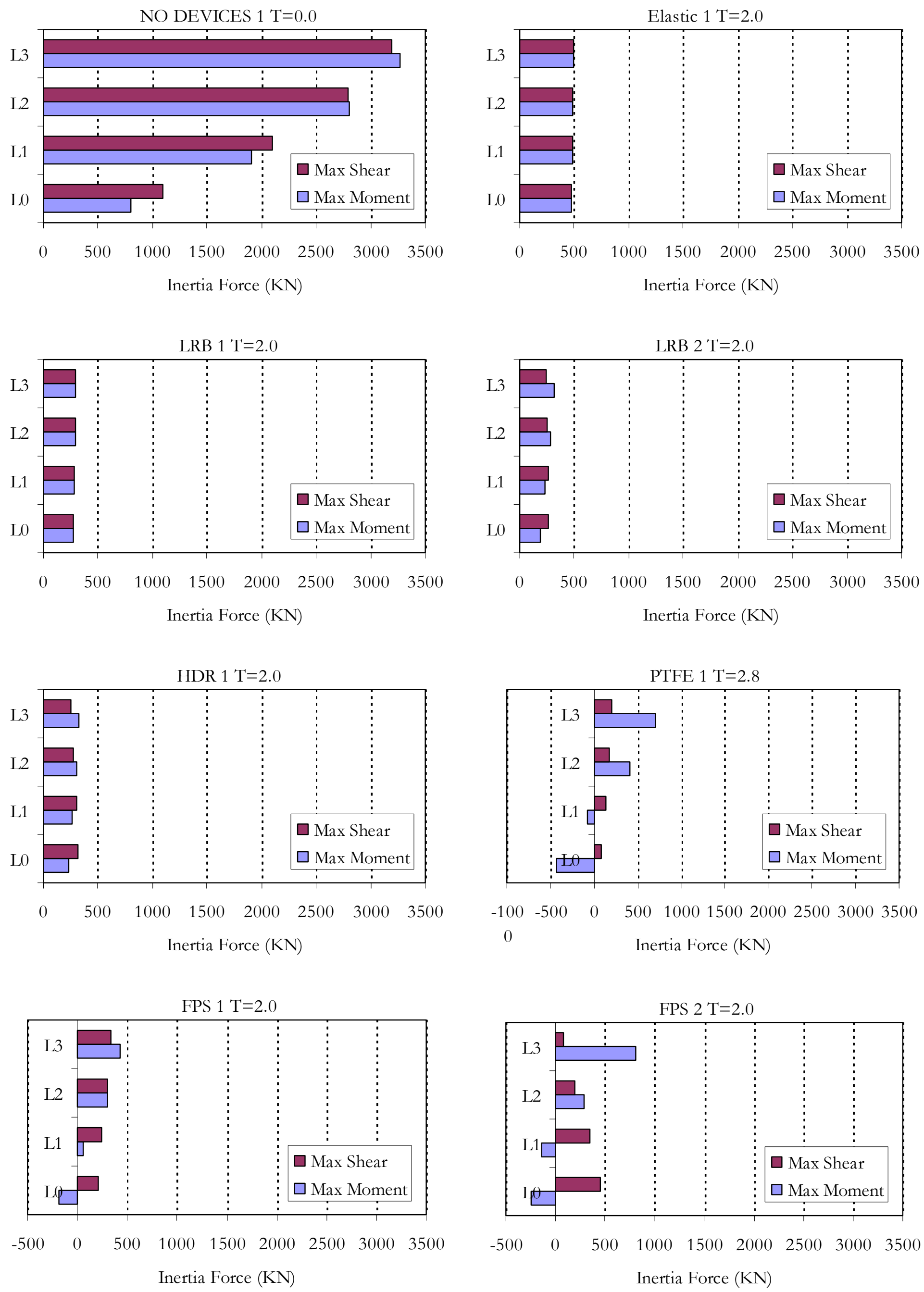


FIGURE 6-20 TIME HISTORY INERTIA FORCES 5 STORY BUILDING T = 0.5 SECONDS

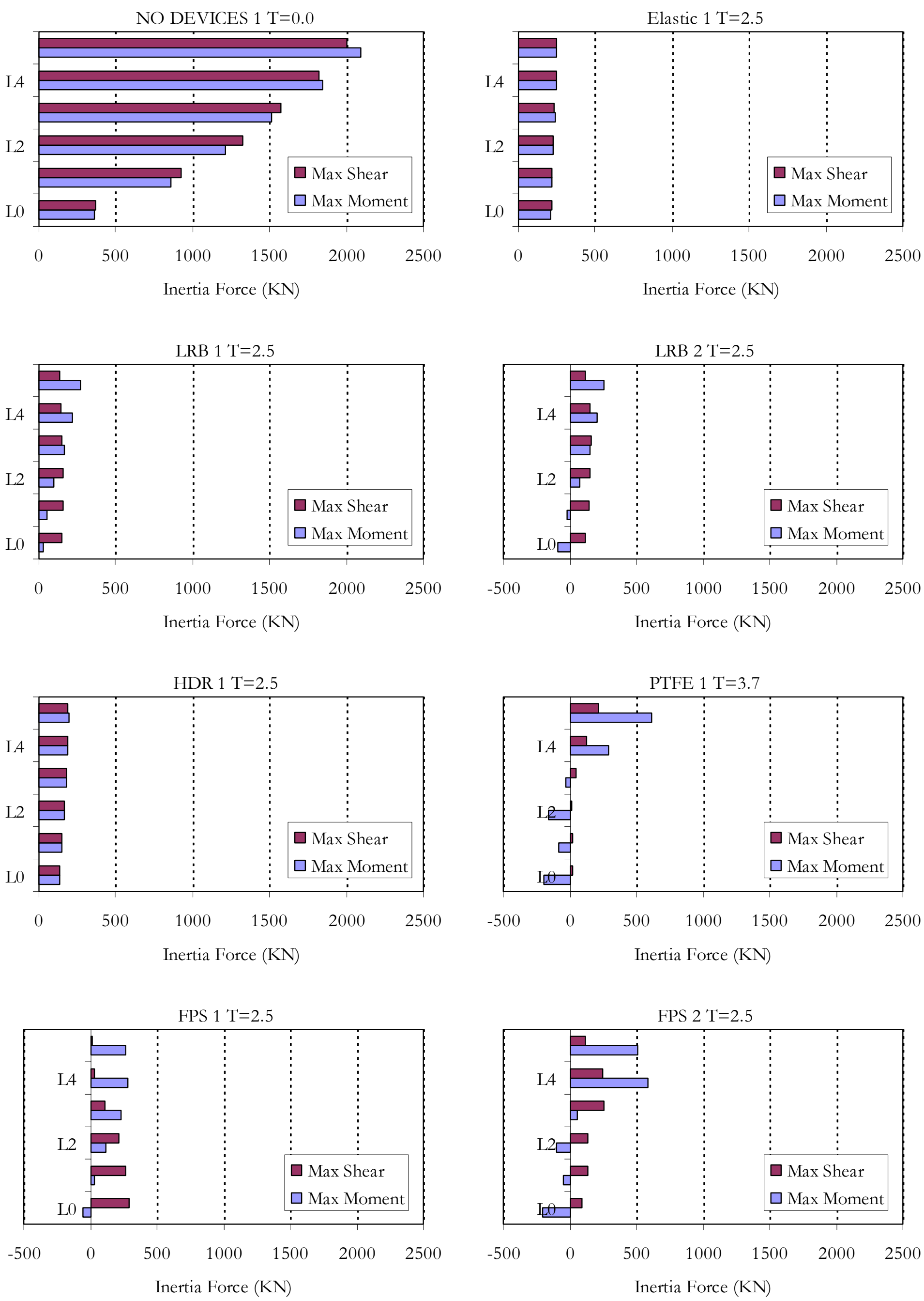
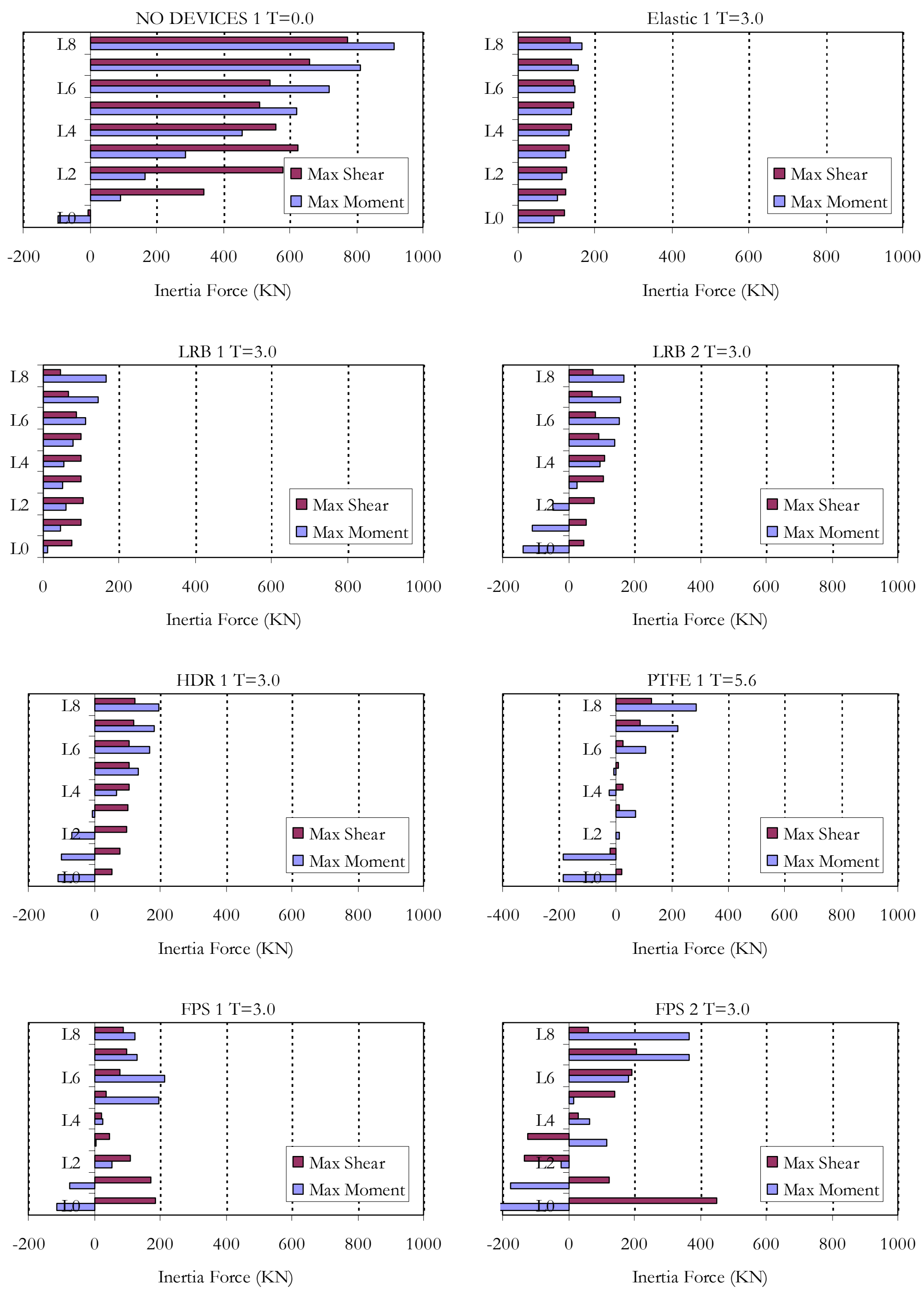


FIGURE 6-21 TIME HISTORY INERTIA FORCES 8 STORY BUILDING T = 1.0 SECONDS



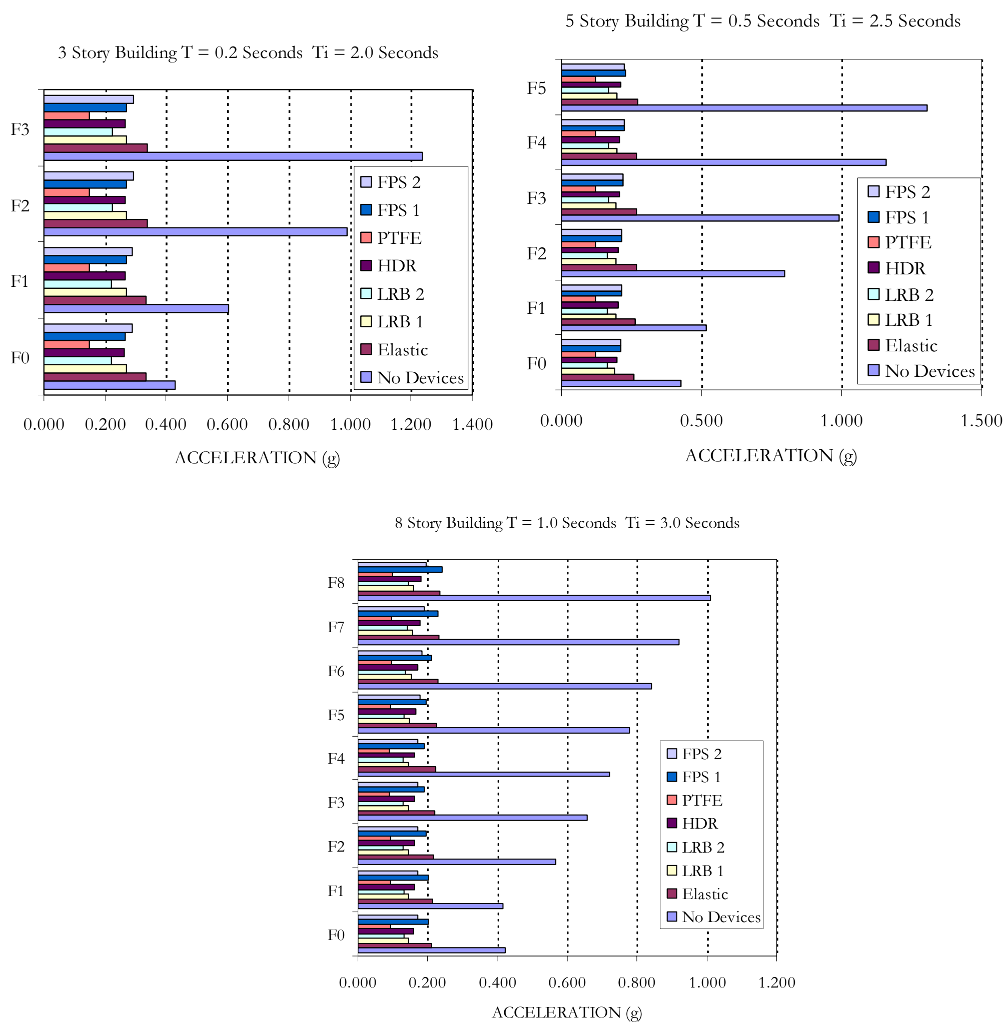
6.1.7 Floor Accelerations

The objective of seismic isolation is to reduce earthquake damage, which includes not only the structural system but also non-structural items such as building parts, components and contents. Of prime importance in attenuating non-structural damage is the reduction of floor accelerations.

6.1.7.1 Response Spectrum Analysis

The floor accelerations from the response spectrum analysis are proportional to the floor inertia forces, as shown in Figure 6-22. The accelerations for the building without devices increase approximately linear with height, from a base level equal to the maximum ground acceleration (0.4g) to values from 2.5 to 3 times this value at the roof (1.0g to 1.2g). The isolated displacements in all cases are lower than the 0.4g ground acceleration and exhibit almost no increase with height.

Figure 6-22 Response Spectrum Floor Accelerations



6.1.7.2 Time History Analysis

Plots of maximum floor accelerations for three building configurations, one of each height, are provided in Figures 6-23, 6-24 and 6-25. These are the same building and isolation system configurations for which the inertia forces are plotted in Figures 6-19 to 6-21. All plots are the maximum values from any of the three earthquakes. They include the accelerations in the building with no isolation as a benchmark. The acceleration at Elevation 0.0, ground level, is the peak ground acceleration from the three earthquakes, which is constant at 0.56g.

The most obvious feature of the plots is that most isolation systems do not provide the essentially constant floor accelerations developed from the response spectrum analysis in Figure 6-22. There are differences between isolation systems but the trends for each system tend to be similar for each building.

- The elastic (E) isolation bearings provide the most uniform distribution of acceleration. As the period of the isolators increases, the accelerations decrease. The longest period, 3.0 seconds, produces accelerations in the structure equal to about one-half the ground acceleration and as the period reduces to 1.5 seconds the accelerations in the structure are about equal to the ground acceleration. As the building period increases the short period isolators show some amplification with height but this is slight.

- The lead rubber bearings (L) produce distributions which are generally similar to those for the elastic bearings but tend to produce higher amplifications at upper levels. The amplification increases as the yield level of the isolation system increases (L1 to L2 to L3 have yield levels increasing from 5% to 7.5% to 10% of W). Again as for the elastic bearings, the accelerations are highest for the shortest isolated periods.

- The PTFE sliding bearings (T) tend to increase the ground accelerations from base level with some amplification with height. Accelerations increase as the coefficient of friction increases, that is, as the effective isolated period reduces.

- The friction pendulum bearings (F) produce an acceleration profile which, unlike the other types, is relatively independent of the isolated period. This type of isolator is more effective in reducing accelerations for the coefficient of friction of 0.06 (F 1) compared to the 0.12 coefficient (F 2). The accelerations are generally higher than for the elastic or lead rubber systems.

Although some systems produce amplification with height and may increase acceleration over the ground value, all isolation systems drastically reduce accelerations compared to the building without isolators by a large margin although, as the plots show, the system type and parameters must be selected to be appropriate for the building type.

FIGURE 6-23 FLOOR ACCELERATIONS 3 STORY BUILDING T = 0.2 SECONDS

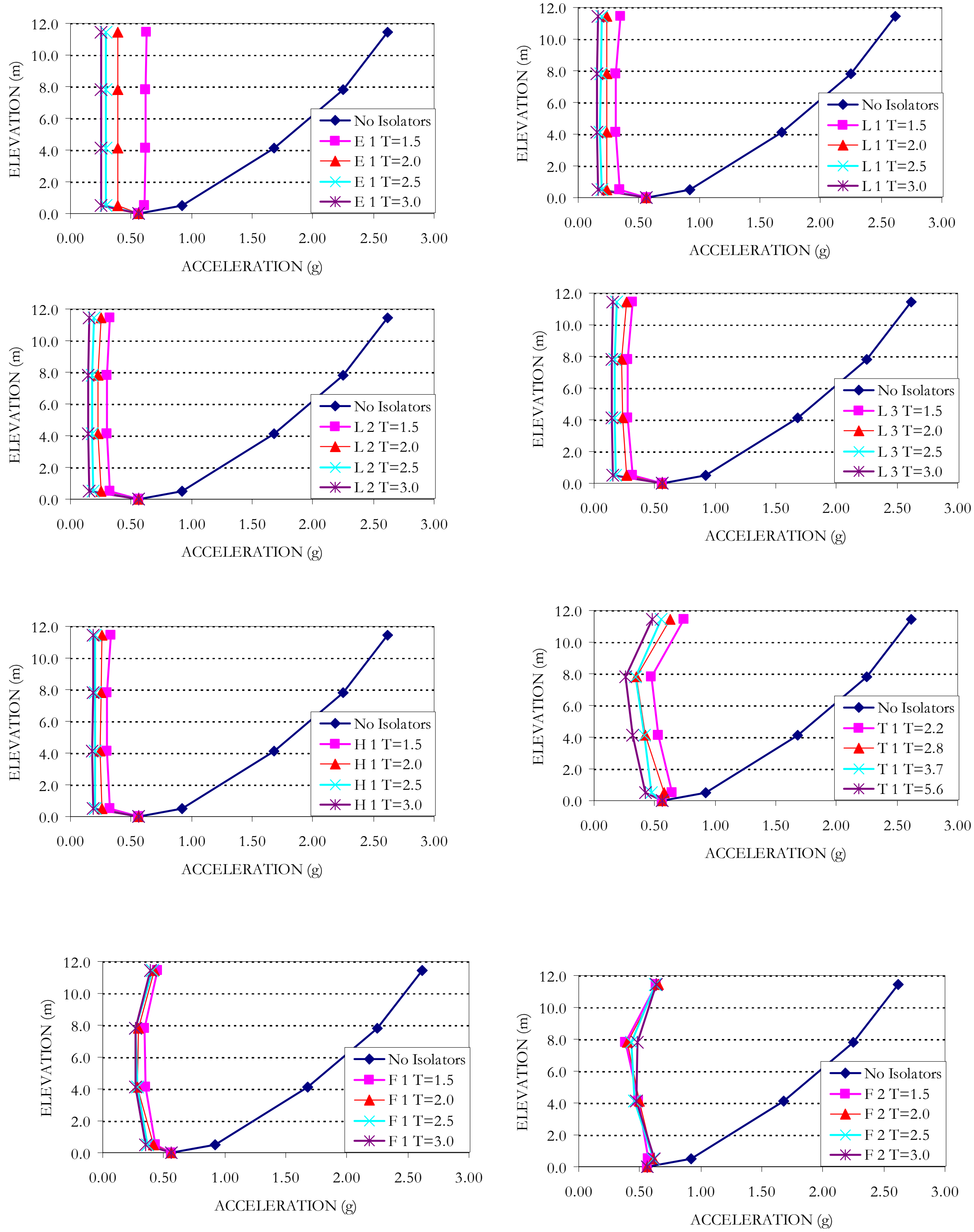


FIGURE 6-24 FLOOR ACCELERATIONS 5 STORY BUILDING T = 0.5 SECONDS

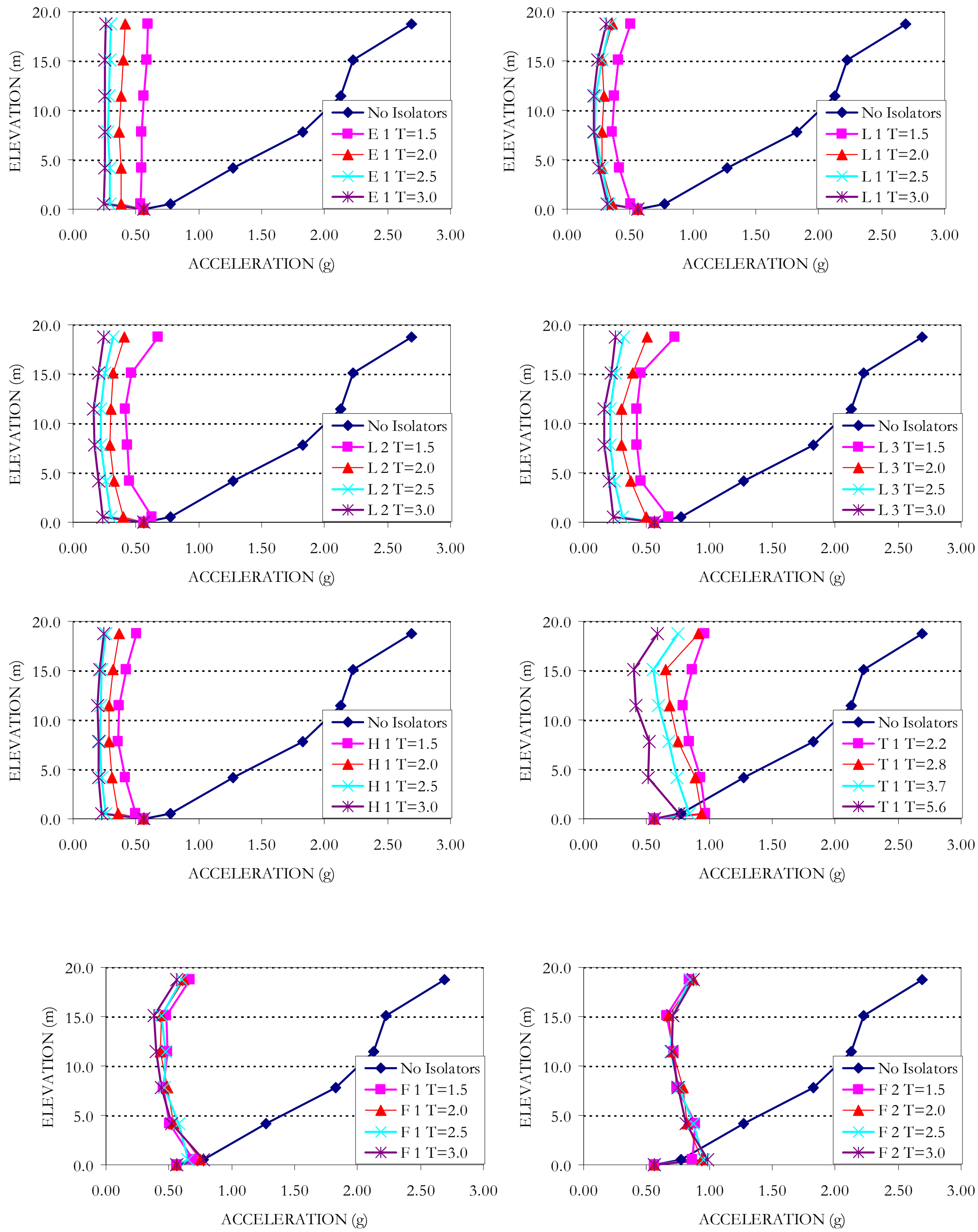
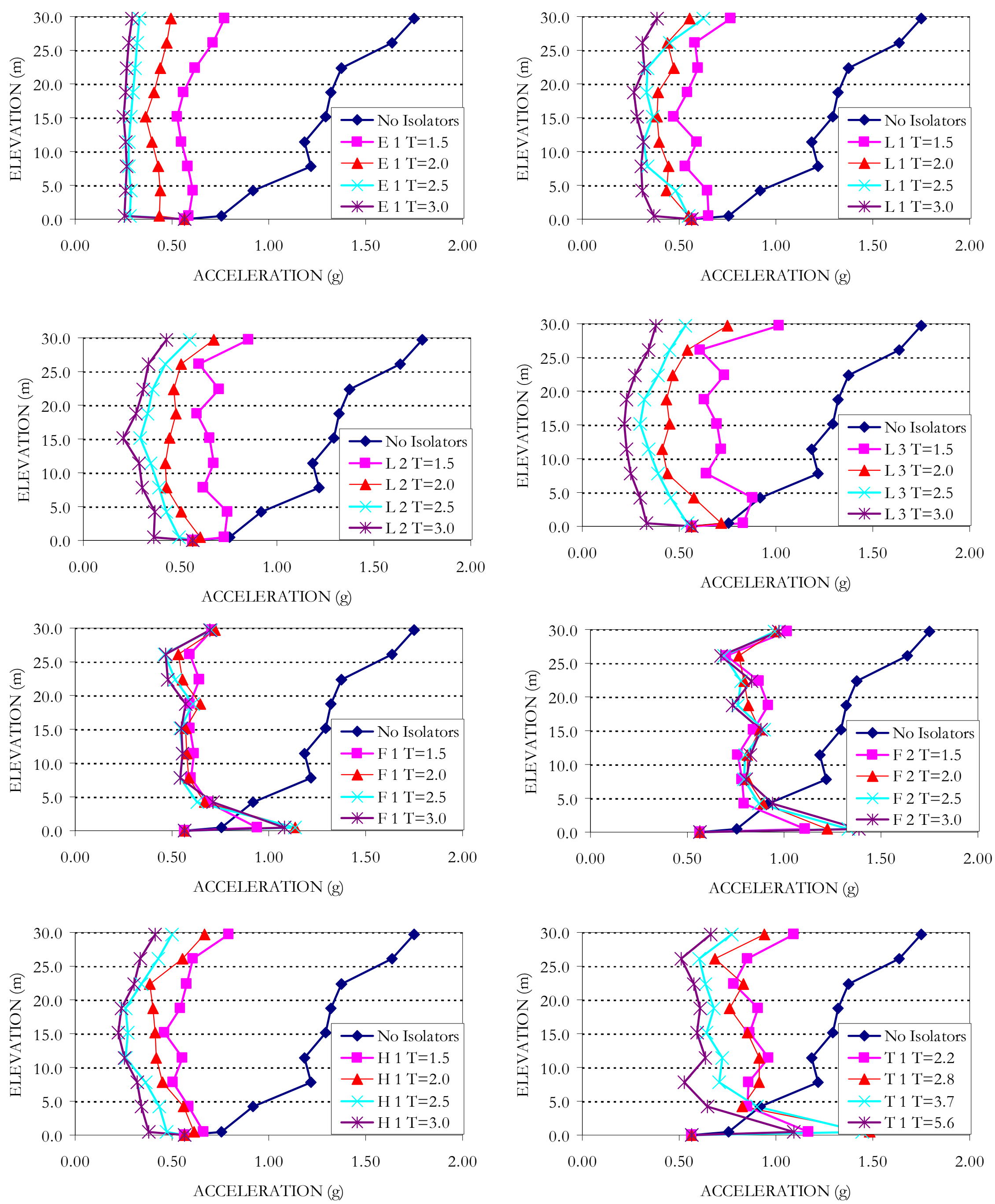


Figure 6-25 Floor Accelerations 8 Story Building T = 1.0 Seconds



6.1.8 OPTIMUM ISOLATION SYSTEMS

The results presented in the previous sections illustrate the wide differences in performance between systems and between different properties of the same system. Different systems have different effects on isolation system displacement, shear coefficient and floor accelerations and no one device is optimum in terms of all possible objectives.

Table 6-4 lists the top 15 systems (of the 32 considered) arranged in ascending order of efficiency for each of three potential performance objectives:

1. Minimum Base Shear Coefficient. The PTFE sliding systems provide the smallest base shear coefficients, equal to the coefficient of friction. These are followed by the LRB with a high yield level (Q_d = 0.10) and 3 second period. However, none of these 4 systems provide a restoring force and so the design displacement is three times the calculated value (UBC provisions). After these four systems, the optimum systems in terms of minimum base shear coefficient are variations of the LRB and FPS systems.

2. Minimum Isolation System Displacement. The FPS systems with a coefficient of friction of 0.12 and relatively short isolated periods are the most efficient at controlling isolation system displacements and the lowest five displacements are all produced by FPS variations. After these are 3 LRB variations and then HDR and FPS. Most of the systems that have minimum displacements have relatively high base shear coefficients and accelerations.

3. Minimum Floor Accelerations. Accelerations are listed for three different building periods and are ordered in Table 6-4 according to the maximum from the three buildings. Some systems will have a higher rank for a particular building period. The elastic isolation systems produce the smallest floor accelerations, followed by variations of LRB and HDR systems. The FPS and PTFE systems do not appear in the optimum 15 systems for floor accelerations.

No system appears within the top 15 of all three categories but some appear in two of three:

1. The FPS systems with a coefficient of friction of 0.12 and a period of 2.5 or 3.0 provide minimum base shear coefficients and displacements. However, floor accelerations are quite high.

2. The LRB with a period of 2 seconds and Q_d = 0.05, 0.075 or 0.10 appear on the list for both minimum displacements and minimum accelerations. The base shear coefficients for these systems are not within the top 15 but are moderate, with a minimum value of 0.203 (compared to 0.06 to 0.198 for the top 15).

3. Five LRB variations and two HDR variations appear in the top 15 for both base shear coefficients and floor accelerations. Of these, the minimum isolated displacement is 258 mm, compared to the range of 103 mm to 213 mm for the top 15 displacements.

These results show that isolation system selection needs to take account of the objectives of isolating and the characteristics of the structure in which the system is to be installed. For most projects a series of parameter studies will need to be performed to select the optimum system.

TABLE 6-4 OPTIMUM ISOLATION SYSTEMS

					Maximum Floor Acceleration (g)		
System	**Variation**	**Period**	**Δ (mm)**	**C**	**T = 0.2 s**	**T = 0.5 s**	**T = 1.0 s**
Minimum Base Shear Coefficient, C							
PTFE	μ=0.06	5.6	1291	**0.060**	0.58	0.89	1.09
PTFE	μ=0.09	3.7	926	**0.090**	0.65	0.99	1.45
PTFE	μ=0.12	2.8	669	**0.120**	0.75	1.02	1.48
LRB	Qd=0.1	3.0	1152	**0.137**	0.15	0.25	0.42
LRB	Qd=0.075	3.0	332	**0.141**	0.15	0.27	0.43
PTFE	μ=0.15	2.2	613	**0.150**	0.83	1.07	1.32
LRB	Qd=0.05	3.0	344	**0.153**	0.16	0.31	0.39
FPS	μ=0.06	3.0	228	**0.162**	0.50	0.83	1.08
LRB	Qd=0.1	2.5	269	**0.163**	0.18	0.33	0.58
LRB	Qd=0.075	2.5	258	**0.167**	0.19	0.33	0.55
FPS	μ=0.12	3.0	130	**0.178**	0.77	1.03	1.38
HDR		3.0	320	**0.179**	0.19	0.25	0.41
LRB	Qd=0.05	2.5	269	**0.180**	0.20	0.34	0.62
FPS	μ=0.06	2.5	199	**0.188**	0.46	0.80	1.13
FPS	μ=0.12	2.5	122	**0.198**	0.77	1.05	1.33
Minimum Isolation System Displacement, Δ							
FPS	μ=0.12	1.5	**103**	0.301	0.75	1.01	1.11
FPS	μ=0.12	2.0	**111**	0.231	0.75	1.07	1.23
FPS	μ=0.12	2.5	**122**	0.198	0.77	1.05	1.33
FPS	μ=0.06	1.5	**124**	0.280	0.53	0.86	0.94
FPS	μ=0.12	3.0	**130**	0.178	0.77	1.03	1.38
LRB	Qd=0.075	1.5	**140**	0.272	0.35	0.70	0.85
LRB	Qd=0.1	1.5	**140**	0.267	0.33	0.74	1.01
LRB	Qd=0.05	1.5	**144**	0.280	0.35	0.53	0.77
HDR		1.5	**148**	0.277	0.33	0.50	0.79
FPS	μ=0.06	2.0	**160**	0.221	0.49	0.83	1.14
HDR		2.0	**177**	0.225	0.26	0.38	0.70
LRB	Qd=0.075	2.0	**195**	0.204	0.27	0.41	0.68
LRB	Qd=0.1	2.0	**197**	0.203	0.28	0.51	0.76
FPS	μ=0.06	2.5	**199**	0.188	0.46	0.80	1.13
LRB	Qd=0.05	2.0	**213**	0.225	0.24	0.36	0.56

					Maximum Floor Acceleration (g)		
System	**Variation**	**Period**	**Δ (mm)**	**C**	**T = 0.2 s**	**T = 0.5 s**	**T = 1.0 s**
Minimum Floor Accelerations, A							
ELASTIC		3.0	528	0.247	**0.25**	**0.26**	**0.29**
ELASTIC		2.5	434	0.279	**0.29**	**0.30**	**0.34**
LRB	Qd=0.05	3.0	344	0.153	**0.16**	**0.31**	**0.39**
HDR		3.0	320	0.179	**0.19**	**0.25**	**0.41**
LRB	Qd=0.1	3.0	1152	0.137	**0.15**	**0.25**	**0.42**
LRB	Qd=0.075	3.0	332	0.141	**0.15**	**0.27**	**0.43**
ELASTIC		2.0	369	0.371	**0.40**	**0.42**	**0.49**
HDR		2.5	269	0.202	**0.20**	**0.27**	**0.51**
LRB	Qd=0.075	2.5	258	0.167	**0.19**	**0.33**	**0.55**
LRB	Qd=0.05	2.0	213	0.225	**0.24**	**0.36**	**0.56**
LRB	Qd=0.1	2.5	269	0.163	**0.18**	**0.33**	**0.58**
LRB	Qd=0.05	2.5	269	0.180	**0.20**	**0.34**	**0.62**
LRB	Qd=0.075	2.0	195	0.204	**0.27**	**0.41**	**0.68**
HDR		2.0	177	0.225	**0.26**	**0.38**	**0.70**
LRB	Qd=0.1	2.0	197	0.203	**0.28**	**0.51**	**0.76**

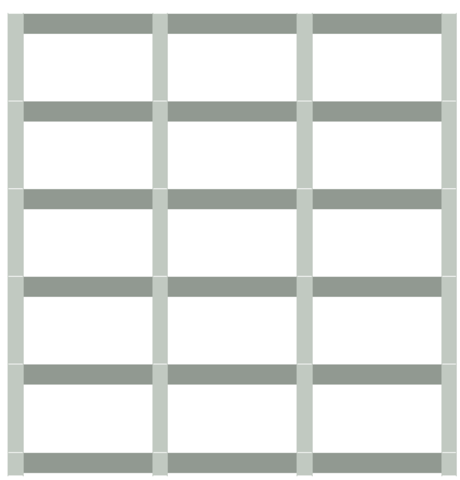
6.2 Problems with the Response Spectrum Method

6.2.1 Underestimation of Overturning

A potentially disturbing aspect of the evaluation in the preceding sections is the large discrepancy in inertia force and acceleration distributions between the response spectrum and the time history methods of analysis. The inertia force distribution defines the overturning moments on a structure and the distributions from the response spectrum analysis produce a smaller overturning moment than these from the time history analysis.

As the shear buildings used above did not produce overturning moments directly from the response spectrum analysis, the example 5 story building was converted to the frame shown in Figure 6-26.

Figure 6-26 Example Frame



Frame elements were selected to produce a period of 0.50 seconds, as used previously.

The response spectrum and time history analyses were repeated for the configurations of (1) No devices, (2) LRB 2 (Q_d = 0.075, 4 effective periods) and (3) FPS 2 (coefficient of friction μ = 0.12, 4 effective periods). For the shear building with these isolation systems the time history analysis produced an effective height of the inertia loads ranging from 0.61H to 0.66H for LRB 2 and 0.70H to 1.09H for FPS 2 (see Figure 6-18). In contrast, the response spectrum analysis produced essentially linear inertia load distributions such that the effective height for both systems was about 0.50H (see Figure 6-16).

The response spectrum and time history analyses were performed using the same process as for the prototype structures and additionally the maximum axial loads in the exterior columns, P, were extracted as a means of calculating the overturning moment. These loads were used to calculate the moment as calculated M = P x L, where L is the distance across the building. From the moment the effective height of the inertia loads can be calculated as H_c = M/V where V is the base shear.

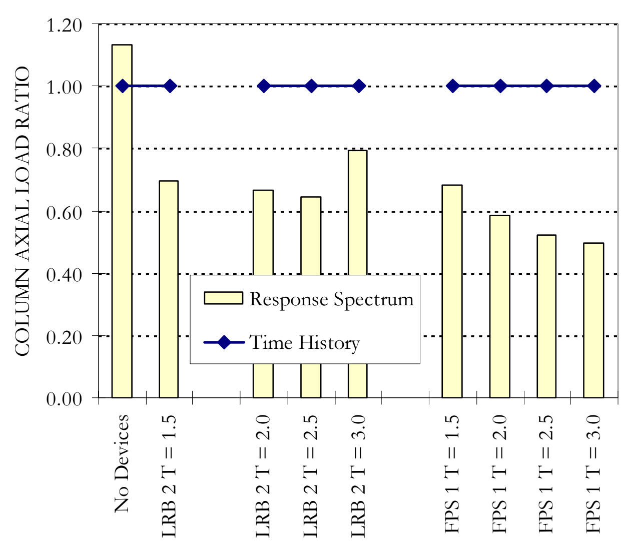
Values so calculated are listed in Table 6-5. These show that for all the isolated systems the response spectrum produces values in a narrow band, from 0.52H to 0.54H whereas for the time history results the values were very system-specific, ranging from 0.65H to 1.06H.

TABLE 6-5 HEIGHT OF INERTIA LOADS

	H_C Response Spectrum	H_C Time History
No Devices	0.72	0.63
LRB 2 T = 1.5	0.53	0.76
LRB 2 T = 2.0	0.52	0.78
LRB 2 T = 2.5	0.52	0.80
LRB 2 T = 3.0	0.52	0.65
FPS 2 T = 1.5	0.54	0.79
FPS 2 T = 2.0	0.53	0.91
FPS 2 T = 2.5	0.53	1.01
FPS 2 T = 3.0	0.53	1.06

The effect of this difference on design can be assessed by comparing column axial loads. The response spectrum values have been normalized by factoring results so as to obtain same base shear as from the equivalent time history analysis. The result ratios are plotted in Figure 6-27. For all isolated configurations the column axial loads are underestimated, in the worst case by a factor of 2.

FIGURE 6-27 AXIAL LOADS IN COLUMNS



6.2.2 REASON FOR UNDERESTIMATION

The problem with the response spectrum analysis based on the effective stiffness procedure is that the modal participation is almost entirely in the fundamental isolated modes with almost zero participation from the higher modes. However, as the time history analysis shows there will be participation from these modes. This has a large influence on the distribution of accelerations and so inertia forces.

The textbooks on base isolation (Skinner, Robinson and McVerry [1993] and Naiem and Kelly [1999]) both discuss how such affects may occur. There appear to be two main effects:

1. For high levels of damping the assumption of uncoupled modes, which forms the basis of the response spectrum method of analysis, does not hold as the isolated and non-isolated modes become increasingly coupled.

2. Probably the more important effect is that for bi-linear systems the initial elastic stiffness will be much more highly coupled with the structural modes than the yielded stiffness. Use of a single effective stiffness ignores this effect. Considering an FPS system with a coefficient of friction of 0.12 for example, the structure will act as a fixed base structure for all the time segments of the response when the base shear does not exceed 0.12 of the weight. The accelerations and inertia forces generated at these times may well exceed the maximum accelerations occurring during the yielded phase.

The books referenced above discuss theoretical techniques for accounting for these effects such as using complex mode shapes for the first and interleaved modes based on the two stiffnesses respectively for the second effect. However, these are not practical to implement within the context of the design office tools we use for response spectrum analysis (ETABS and SAP2000).

The accuracy of the response spectrum method, and means to improve the correlation with the time history method, forms a topic for further research. Pending resolution of these outstanding issues, we should use the time history method of analysis for all our base isolated projects even though code such as UBC and FEMA-273 permit the use of the effective stiffness response spectrum method.

Use of the time history method is not a major problem as we have procedures to implement time history analysis with about the same level of effort as for response spectrum analysis and the main impediment to using this form of analysis, the need for peer review on U.S. projects, is not an issue given that peer review is required for all base isolation projects there anyway.

6.3 EXAMPLE ASSESSMENT OF ISOLATOR PROPERTIES

The limited studies discussed above have shown that there is no one isolation system type, or set of system parameters, which provides optimum performance in all aspects. For projects, it is recommended that a series of studies by performed to tune the system to the structure. Following is an example of how this has been applied to a building project.

For this project a lead rubber system was selected as the isolation type based on a need for relatively high amounts of damping. The LRB properties were selected by assessing performance for a wide range of properties. For this type of bearing the plan size is set by the vertical loads. The stiffness, and so effective period, is varied by changing the height of the bearing, which is accomplished by changing the number of rubber layers. The yield level of the system is varied by modifying the size of the lead cores in the bearing.

For this project the performance was assessed by varying the number of layers from 40 to 60 (changing stiffness by 50%) and by varying the lead core diameter from 115 mm (4.5") to 165 mm (6.5"), changing the yield level by 100%.

A program was set up to cycle through a series of 3D-BASIS analyses. For each variation, the program adjusted the input file properties for stiffness and yield level, performed the analysis and extracted the output response quantities from the output file. From these results the plots in Figures 6-28 and 6-29 were generated. The isolator naming convention is, for example, L40-6, which indicates 40 layers with a 6" lead core.

These plots are used to determine trends in isolator displacements, shear forces and maximum floor accelerations. For this particular structure and seismic input, both the isolator displacement and the base shear coefficient decrease as either the stiffness is decreased or the yield level is increased. However, the maximum floor accelerations increase as the displacements and coefficients decrease and there is a point, in this case then the lead core is increased beyond 6", where the accelerations increase dramatically.

From the result of this type of analysis, isolators can be selected to minimize respective floor accelerations, drifts, base shears of isolator coefficients. As listed in Table 6-6, in this case the minimum accelerations and drifts occur for a tall bearing (60 layers) with a small lead core (4.5"). The isolator displacement can be reduced from 420 mm (16.5") to 350 mm (13.8") and the base shear coefficient from 0.121 to 0.113 by increasing the lead core from 4.5" to 5" (a 23% increase in yield level). This only increases the floor accelerations and drifts by 10% so is probably a worthwhile trade-off.

Minimum isolator displacements are provided by a stiff bearing (44 layers) with a large core (6.5") but there is a small penalty in base shear coefficient and a very large penalty in floor accelerations associated with his. For this project, design should accept isolator displacements of 350 mm to ensure the best performance of the isolated structure.

FIGURE 6-28 DISPLACEMENT VERSUS BASE SHEAR

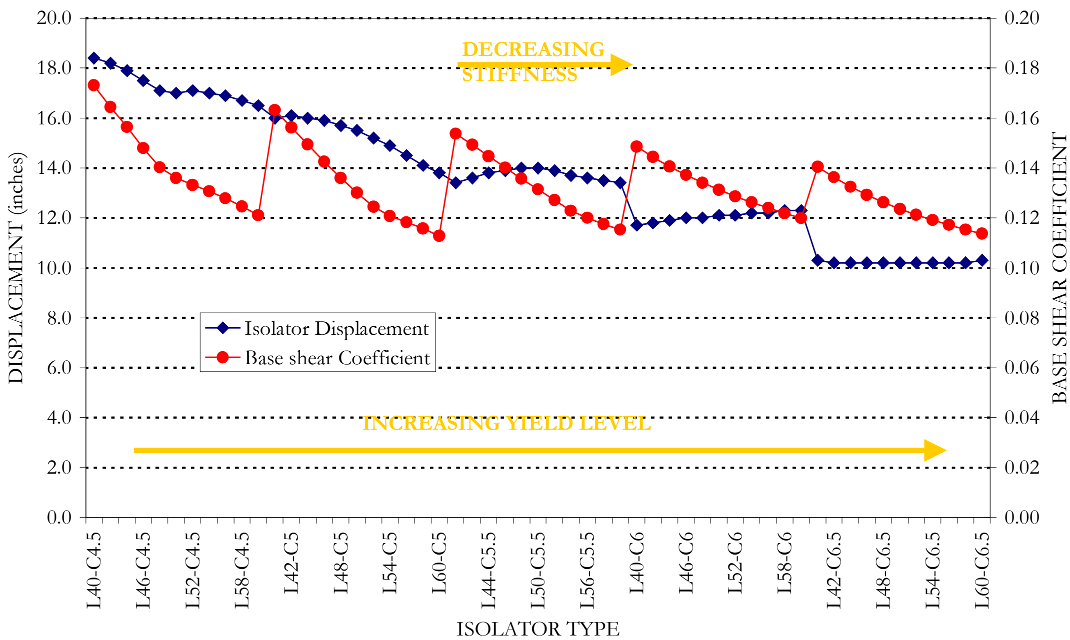


FIGURE 6-29 DISPLACEMENT VERSUS FLOOR ACCELERATION

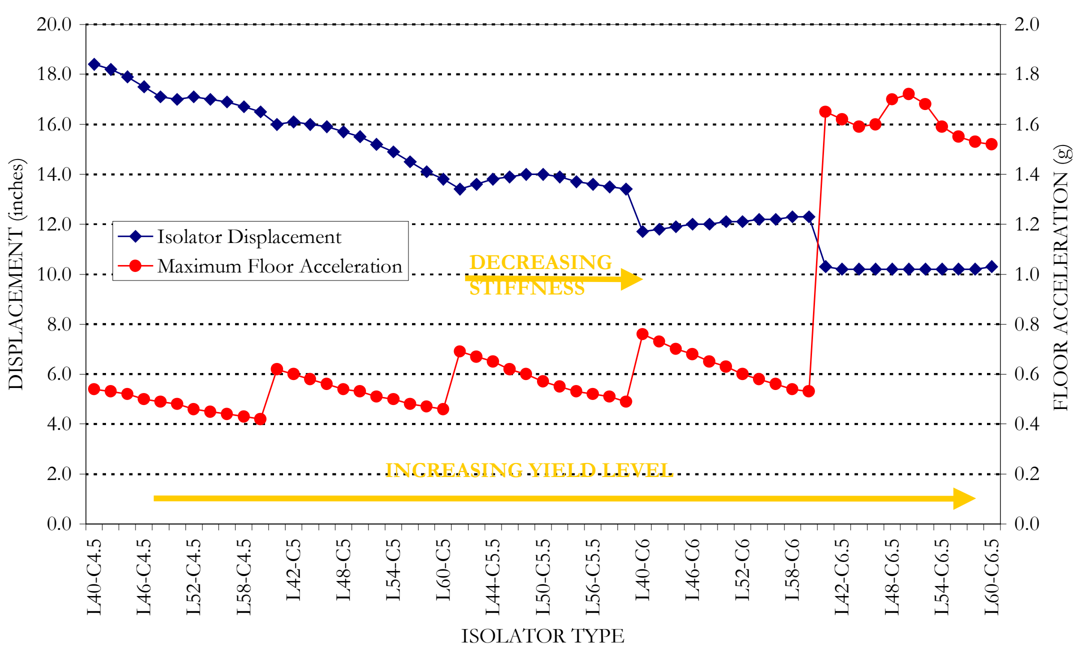
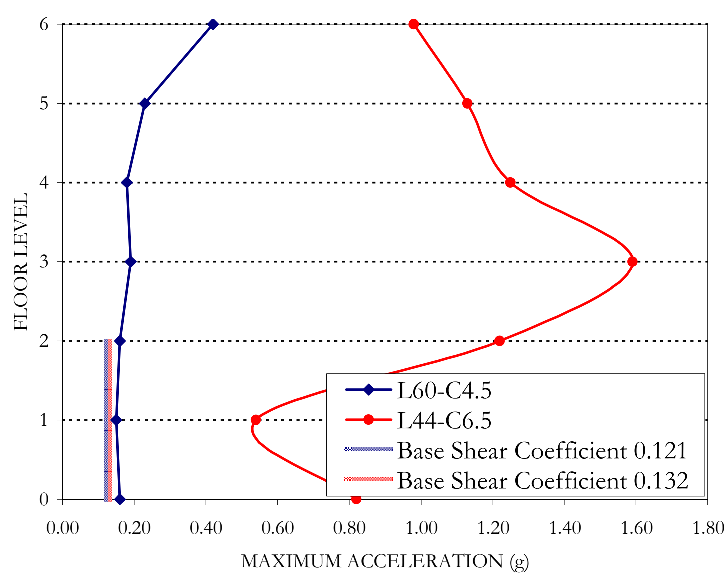


Table 6-6 Optimum Isolator Configuration

	Isolator Displacement mm (Inches)	Base Shear Coefficient (g)	Maximum Floor Acceleration (g)	Maximum Drift
Minimum Floor Acceleration L60-C4.5	420 (16.5)	0.121	0.420	0.0032
Minimum Drift L60-C4.5	420 (16.5)	0.121	0.420	0.0032
Minimum Base Shear Coefficient L60-C5	350 (13.8)	0.113	0.460	0.0035
Minimum Isolator Displacement L44-C6.5	260 (10.2)	0.132	1.590	0.0071

Figure 6-30 Floor Acceleration Profiles



These analyses also illustrate the discussion earlier about the non-uniform nature of the acceleration distribution when determined from the time history method of analysis. Figure 6-30 plots the acceleration profiles for the systems which provide the minimum floor accelerations and minimum isolator displacements, respectively. Even though these systems provide a similar base shear coefficient, the stiff system with high damping provides floor accelerations over three times as high. The shape of the acceleration profile for the latter system exhibits the characteristics of very strong higher mode participation. An analysis which used only effective stiffness would not reflect this effect.

7 ISOLATOR LOCATIONS AND TYPES

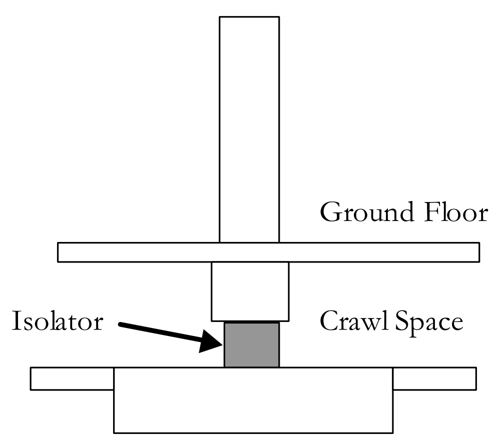
7.1 Selection of Isolation Plane

7.1.1 Buildings

The paramount requirement for installation of a base isolation system is that the building be able to move horizontally relative to the ground, usually at least 100 mm and in some instances up to 1 meter. A plane of separation must be selected to permit this movement. Final selection of the location of this plane depends on the structure but there are a few items to consider in the process. See also Chapter 12 of these guidelines, Structural Design, as there are design consequences of decisions made in the selection of the location of the isolation plane.

The most common configuration is to install a diaphragm immediately above the isolators. This permits earthquake loads to be distributed to the isolators according to their stiffness. For a building without a basement, the isolators are mounted on foundation pads and the structure constructed above them, as shown in Figure 7-1. The crawl space is usually high enough to allow for inspection and possible replacement of the isolators, typically at least 1.2 m to 1.5 m.

Figure 7-1 Building with No Basement



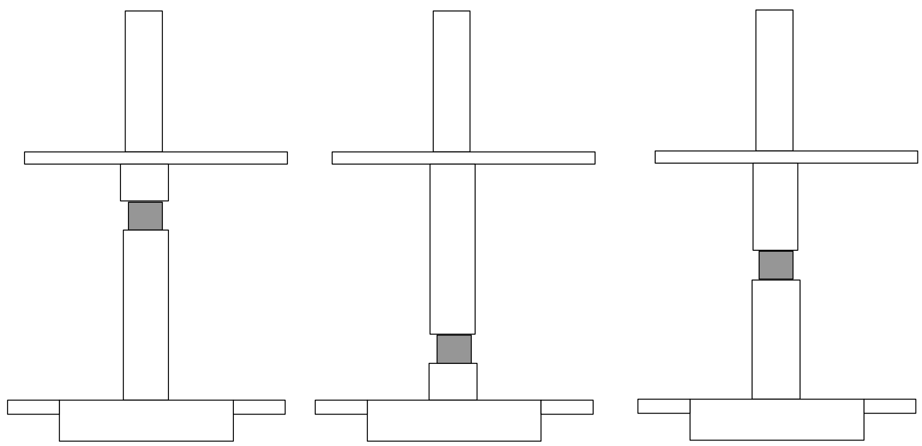
If the building has a basement then the options are to install the isolators at the top, bottom or mid-height of the basements columns and walls, as shown in Figure 7-2. For the options at the top or bottom of the column/wall then the element will need to be designed for the cantilever moment developed from the maximum isolator shear force. This will often require substantial column sizes and may require pilasters in the walls to resist the face loading.

The mid-height location has the advantage of splitting the total moment to the top and bottom of the component. However, as discussed later in Connection Design there will be P-Δ moments in the column/wall immediately above and below the isolator.

The demands on basement structural members can be minimized by careful selection of isolator types and by varying the isolator stiffness. For example, if a LRB system is used then large lead cores may be used in isolators at locations such as wall intersections where there is a high resistance to lateral loads. More vulnerable elements such as interior columns may have isolators with small cores or no cores. As the diaphragm will enforce equal displacements at all isolators this will reduce the forces on the interior columns.

If structural elements below the isolation interface are flexible then they may modify the performance of the isolation system as some displacement will occur in the structural element rather than the isolator. They should be included in the structural model. See discussion on bridge isolation where flexible substructures are common.

FIGURE 7-2 INSTALLATION IN BASEMENT



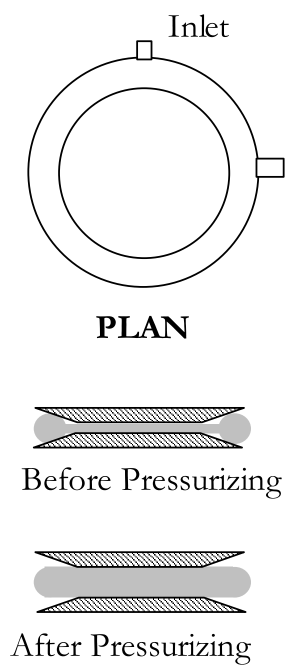
Selection of the isolation plane for the retrofit of existing buildings follows the same process as for new buildings but usually there are more constraints. Also, many of the issues which are resolved during design for a new installation, such as secondary moments, diaphragm action above the isolators and the capacity of the substructure to resist to maximum isolator forces, must be incorporated into the existing building.

Figure 7-4 shows conceptually some of the issues that may be faced in a retrofit installation of any isolation system. These are schematic only as most retrofit projects have unique conditions. You may encounter some of all of these and will most likely also need to deal with other issues:

- The isolators must be installed into the existing structure. The existing structure must be cut away to permit installation. For column installation this will require temporary support for the column loads. For wall structures, it may be possible to cut openings in the wall for the isolators to be installed while the non-separated portion of the wall supports the load. The wall between isolators is removed after installation of the isolators.

- The gravity load must be distributed to the isolators. Usually this is accomplished with flat-jacks, which are hydraulic capsules in the form of a flat double saucer. Thrust plates are placed top and bottom, as shown in Figure 7-3 (adapted from a PSC Freyssinet catalogue). When the jack is inflated hydraulically the upper and lower plates are forced apart. The jacks can be inflated with hydraulic oil but for most isolation projects an epoxy grout is used and the jacks are left in place permanently.

FIGURE 7-3 FLAT JACK

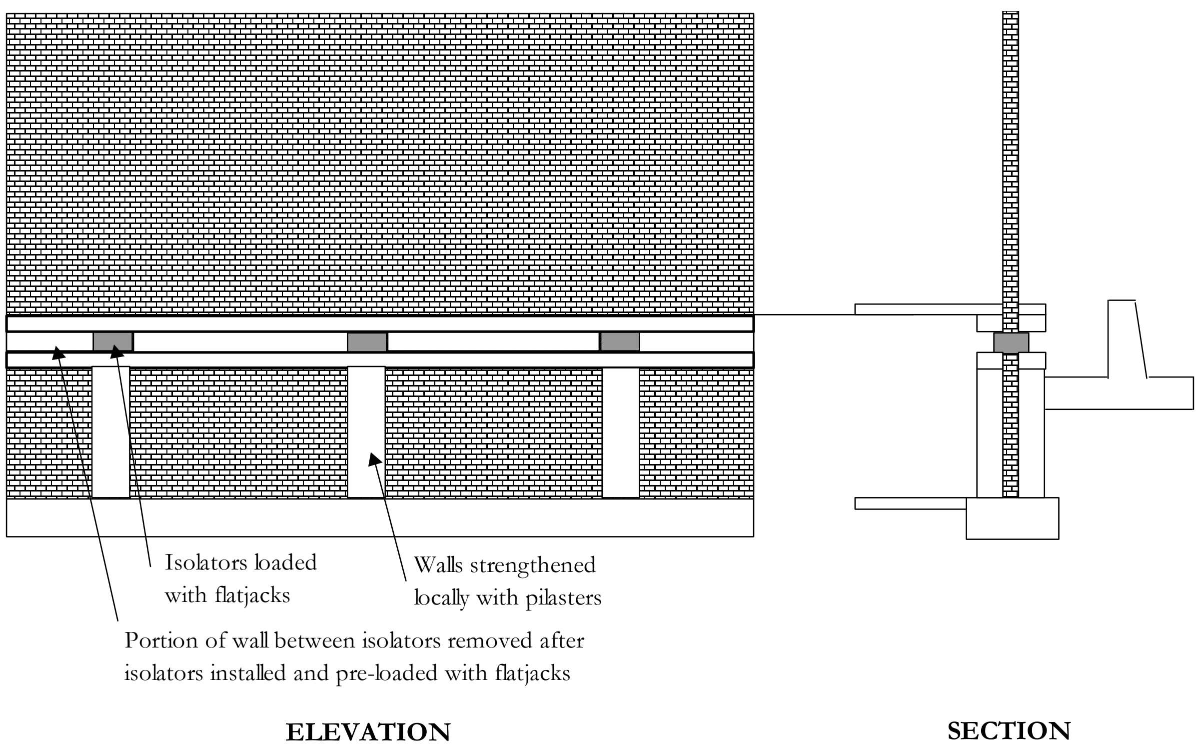


- For installation in wall structures, the walls will be need to be strengthened above and below the isolators to resist primary and secondary moments. Often, precast concrete horizontal needle beams are clamped to each side of the existing wall above and below the isolators. These needle beams are connected using stressed rods.

- The existing wall will usually need strengthening to transfer the bending moment arising from the isolator force to the foundation elements. This may require pilasters.

- The structure above the isolators must be able to move freely by the maximum displacements, usually in the range of 150 mm to 500 mm or more. This will require construction of a moat around the building and may influence selection of the isolation plane as installation at the bottom of the basement will require deep retaining walls to allow movement.

FIGURE 7-4 CONCEPTUAL RETROFIT INSTALLATION



7.1.2 ARCHITECTURAL FEATURES AND SERVICES

Apart from the structural aspects, base isolation requires modifications to architectural features and services to accommodate the movements. Especially important are items which cross the isolation plane, which will include stairs, elevators, water, communications, waste water and power. Provision will also need to be made to ensure that the separation space does not get blocked at some future time.

There are devices available to provide flexible service connections and these can generally be dealt with by the services engineers, who must be advised of the location of the isolation plane and the maximum movements.

Elevators usually cantilever below the isolation plane. The portion below the isolation system will need to have separation all round so that the movement can occur. Stairs may cantilever from above the isolation plane or may be on sliding bearings.

Most isolation projects will have some items such as stairs, shaft walls etc. which require vertical support but must move with the isolators. The most common support for these situations is small sliding bearings. As the vertical reaction is usually small the friction resistance will be negligible compared to the total isolation force.

With sliding bearings, no matter how small the load the displacement will still be equal to the maximum displacements and so even though a small bearing pad is used the size of the slide plate will be as large as for a heavy load.

Most of the problems you may encounter will have been solved on previous isolation projects. Check the files on our previous installations, discuss them with project engineers from earlier projects and consult some of the published case studies from isolation projects world wide.

7.1.3 BRIDGES

As noted in Chapter 4 of these guidelines, the most common location for the isolation plane for bridges is at the top of bents, isolating the superstructure. If the bents are single column bents then the pier will function as a cantilever. Multi-column bents will function as cantilevers under longitudinal loads but will act as frames transversely if the isolators are placed above the top transverse beam.

The weight of the bents themselves is often a high proportion of the total bridge weight and it may be preferable to isolate this portion of the mass as well as the superstructure mass. This could be achieved by placing the isolators at the base rather than at the top of the bent columns. In practice, this is likely to be a problem as there will be large moments, which must be resisted by the bridge superstructure. There may be some bridge configurations where this is practical.

An unusual form of isolation which has been used on the South Rangitikei Viaduct in New Zealand is a rocking isolation system. The 70 m tall twin column piers have a horizontal separation plane near the bottom of each column. When the bridge moves under transverse earthquake loads the piers will rock on one column. Steel torsion bar energy absorbers are used to control the upward displacements and absorb energy under each uplift cycle.

7.1.4 OTHER STRUCTURES

Selection of the isolation plane for other types of structures will follow the same general principles as for buildings and bridges. Isolation reduces the inertia forces in all mass above the isolators and so the general aim is to isolate as much of the weight of possible, which usually means placing the isolators as close to the base as possible.

Exceptions may be where a large mass is supported on a light frame, such as an elevated water tank. It may be possible to install the isolators under the tank at the top of the frame. This will isolate the majority of the mass and will minimize the overturning moments on the isolators, avoiding tension loads in the isolators. However, the frame base must be able to resist the overturning moments from the maximum isolator shear forces applied at the top of the frame.

The structures considered within these guidelines, buildings and bridges, are all relatively heavy and most isolation devices are most suited to large loads. This is because for a given isolation period the displacement is the same regardless of mass and it is difficult to retain stability of small isolators under large displacements. Sliding devices work well under light loads and there has been some development work performed on other low mass devices. Systems based on elastomeric bearings are suitable for loads per device of at least 50 KN and preferably 200 KN. This may restrict options available for structures other than buildings and bridges.

7.2 Selection of Device Type

No one type of device is perfect. If it were, all projects would use the same type of device. Of the types available, following is a summary of their characteristics and advantages and disadvantages. Each project will have specific objectives and constraints and so you will need to select devices that best fit your specific criteria.

7.2.1 Mixing Isolator Types and Sizes

Most projects use a single type of isolator although sliding bearings in particular are often used with lead rubber or high damping rubber bearings. As discussed below, sliding bearings provide good energy dissipation, can resist high compression loads and permit uplift should tension occur but have the disadvantages of sticking friction and not providing a restoring force. If used in parallel with bearing types that do provide a restoring force the advantages of sliding bearings can be used without the disadvantages.

The UBC procedures can be used to determine the ratios of the two types of bearing. A rule of thumb is that the sliding bearings should support no more than 30% of the seismic mass and the LRBs of HDR bearings the remainder. The most common use of sliding bearings is where shear walls provide high overturning forces. A sliding bearing can efficiently resist the high compression and the tension end of the wall can be permitted to uplift.

For most bearing types the plan size required increases as vertical load increases but the height (of LRB and HDR bearings) or radius (of FPS bearings) is constant regardless of vertical load as all bearings will be subjected to the same displacement. Therefore, the bearings can be sized according to the vertical load they support. In practice, usually only a single size or two sizes are used for a particular project. This is for two reasons:

1. For most applications, each different size of bearing requires two prototypes, which are extra bearings used for testing and not used in the finished structure. If the plan size is reduced for some locations with lower loads then the cost savings are often not enough to offset the extra prototype supply and testing costs. If there are less than 20 isolators of a particular size then it is probably more economical to increase them to the next size used.

2. For high seismic zones, a minimum plan size of LRB or HDR isolators is required to ensure stability under maximum lateral displacements. As all bearings have the same displacement, a reduced vertical load may not translate into much reduction, if any, in plan size.

The design procedures can be used to decide whether several sizes of isolator are economically justified. Sort the isolator locations according to maximum vertical loads and then split them into perhaps 2, 3 or more groups, depending on the total number of isolators. Design them using first the same size for all groups and then according to minimum plan size. Check the total volume required for each option, including prototype volume. Price is generally proportional to total volume so this will identify the most economical grouping.

7.2.2 Elastomeric Bearings

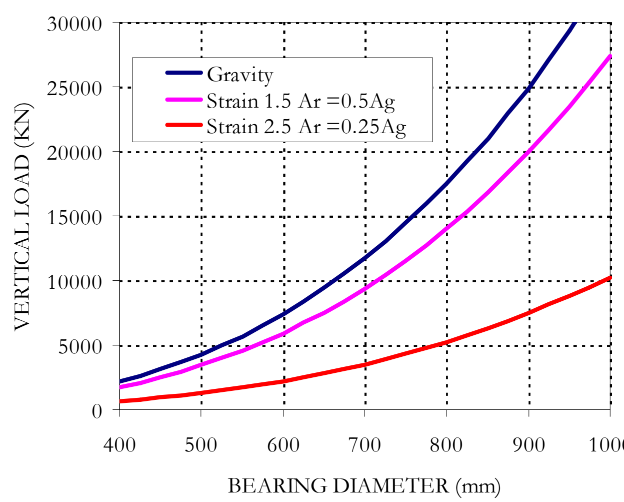
An elastomeric bearing consists of alternating layers of rubber and steel shims bonded together to form a unit. Rubber layers are typically 8 mm to 20 mm thick, separated by 2 mm or 3 mm thick steel shims. The steel shims prevent the rubber layers from bulging and so the unit can support high vertical loads with small vertical deflections (typically 1 mm to 3 mm under full gravity load). The internal shims do not restrict horizontal deformations of the rubber layers in shear and so the bearing is much more flexible under lateral loads than vertical loads, typically by at least two orders of magnitude.

Elastomeric bearings have been used extensively for many years, especially in bridges, and samples have been shown to be functioning well after over 50 years of service. They provide a good means of providing the flexibility required for base isolation.

Elastomeric bearings use either natural rubber or synthetic rubber (such as neoprene), which have little inherent damping, usually 2% to 3% of critical viscous damping. They are also flexible at all strain levels and so do not provide resistance to movement under service loads. Therefore, for isolation they are generally used with special elastomer compounds (high damping rubber bearings) or in combination with other devices (lead rubber bearings).

As discussed later in Chapter 9 of these guidelines, the load capacity of an elastomeric bearing in an undeformed state is a function of the plan dimension and layer thickness. When shear displacements are applied to the bearing the load capacity reduces due to the shear strain applied to the elastomer and to the reduction of effective "footprint" of the bearing. Figure 7-5 provides an example of the load capacity of elastomeric bearings with a medium soft rubber and 10 mm layers.

Figure 7-5 Load Capacity of Elastomeric Bearings



The load capacity is plotted for gravity loads (assuming zero lateral displacements) and for two seismic conditions, the first a moderate displacement producing a shear strain of 150% and an effective area of 0.50 times the gross area. The second is for a very severe seismic displacement, producing a shear strain of 250% and an effective area of only 0.25 times the gross area. This latter case represents the extreme design limits for this type of bearing.

As shown in Figure 7-5, the allowable vertical load reduces rapidly as the seismic displacement increases. This makes the sizing of these isolators complicated in high seismic zones. This is further complicated by the fact that vertical loads on the bearings may increase with increasing displacements, for example, under exterior columns or under shear walls.

7.2.3 High Damping Rubber Bearings

The term high damping rubber bearing is applied to elastomeric bearings where the elastomer used (either natural or synthetic rubber) provides a significant amount of damping, usually from 8% to 15% of critical. This compares to the more "usual" rubber compounds, which provide around 2% damping.

The additional damping is produced by modifying the compounding of the rubber and altering the cross link density of the molecules to provide a hysteresis curve in the rubber. Therefore, the damping provided is hysteretic in nature (displacement dependent). For most HDR compounds the viscous component of damping (velocity dependent) remains relatively small (about 2% to 5% of critical).

The damping provided by the rubber hysteresis can be used in design by adopting the concept of "equivalent viscous damping" calculated from the measured hysteresis area, as in done for LRBs. As for LRBs, the effective damping is a function of strain. For most HDR used to date the effective damping is around 15% at low (25% to 50%) strains reducing to 8%-12% for strains above 100%, although some synthetic compounds can provide 15% or more damping at higher strains.

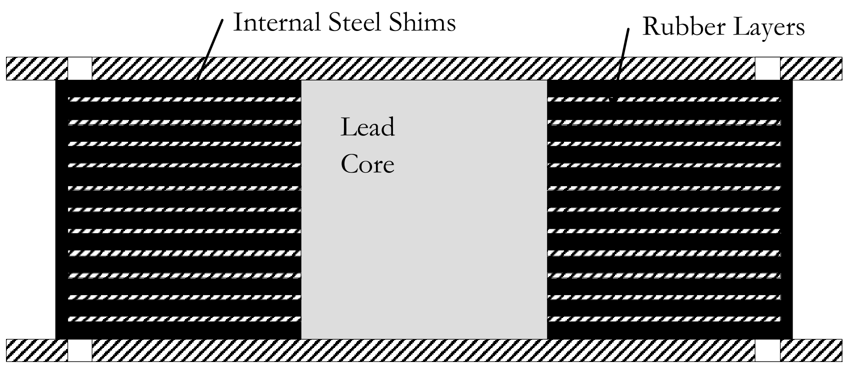
For design, the amount of damping is obtained from tabulated equivalent viscous damping ratios for particular elastomer compounds. The load capacity for these bearings is based on the same formulas used for elastomeric bearings.

7.2.4 Lead Rubber Bearings

A lead-rubber bearing is formed of a lead plug force-fitted into a pre-formed hole in an elastomeric bearing. The lead core provides rigidity under service loads and energy dissipation under high lateral loads. Top and bottom steel plates, thicker than the internal shims, are used to accommodate mounting hardware. The entire bearing is encased in cover rubber to provide environmental protection.

When subjected to low lateral loads (such as minor earthquake, wind or traffic loads) the lead-rubber bearing is stiff both laterally and vertically. The lateral stiffness results from the high elastic stiffness of the lead plug and the vertical rigidity (which remains at all load levels) results from the steel-rubber construction of the bearing.

FIGURE 7-6 LEAD RUBBER BEARING SECTION



At higher load levels the lead yields and the lateral stiffness of the bearing is significantly reduced. This produces the period shift effect characteristic of base isolation. As the bearing is cycled at large displacements, such as during moderate and large earthquakes, the plastic deformation of the lead absorbs energy as hysteretic damping. The equivalent viscous damping produced by this hysteresis is a function of displacement and usually ranges from 15% to 35%.

A major advantage of the lead-rubber bearing is that it combines the functions of rigidity at service load levels, flexibility at earthquake load levels and damping into a single compact unit. These properties make the lead-rubber bearing the most common type of isolator used where high levels of damping are required (in high seismic zones) or for structures where rigidity under services loads is important (for example, bridges). As for HDR bearings, the elastomeric bearing formulas are also applicable for the design of LRBs.

7.2.5 FLAT SLIDER BEARINGS

Sliding bearings provide an elastic-perfectly plastic hysteresis shape with no strain hardening after the applied force exceeds the coefficient of friction times the applied vertical load. This is attractive from a structural design perspective as the total base shear on the structure is limited to the sliding force.

An ideal friction bearing provides a rectangular hysteresis loop, which provides equivalent viscous damping of $2/\pi$ = 63.7% of critical damping, much higher than achieved with LRB's or HDR bearings.

In practice, sliding bearings are not used as the sole isolation component for two reasons:

1. Displacements are unconstrained because of the lack of any centering force. The response will tend to have a bias in one direction and a structure on a sliding system would continue to move in the same direction as earthquake aftershocks occur.

2. A friction bearing will be likely to require a larger force to initiate sliding than the force required to maintain sliding. This is termed static friction, or "stickion". If the sliders are the only component then this initial static friction at zero displacement will produce the governing design force.

The UBC and AASHTO codes require that isolation systems either have a specified restoring force or be configured so as to be capable of accommodating three times the earthquake displacement otherwise required. As maximum design earthquake displacements may be of the order of 400-500 mm this would required sliding systems to be designed for perhaps 1.5m of movement. This may be impractical for detailing movement joints, services, elevators etc.

A hybrid system with elastomeric bearings providing a restoring force in parallel with sliding bearings may often be an economical system. Sliding bearings such as pot bearings using Teflon as a sliding surface can take much higher compressive stresses than elastomeric bearings (60 MPa or more versus 15 MPa or so for elastomeric). Also, the bearings can uplift without disengagement of dowels. Therefore, they are especially suitable at the ends of shear walls and were used at these locations for the Museum of New Zealand.

The most common sliding surface is Teflon on stainless steel. This has a low static coefficient of friction, around 3%. However, the coefficient is a function of both pressure and velocity of sliding. With increasing pressure the coefficient of friction decreases. With increasing velocity the coefficient increases significantly and at earthquake velocities (0.2 to 1 m/sec) the coefficient is generally about 8% to 12%.

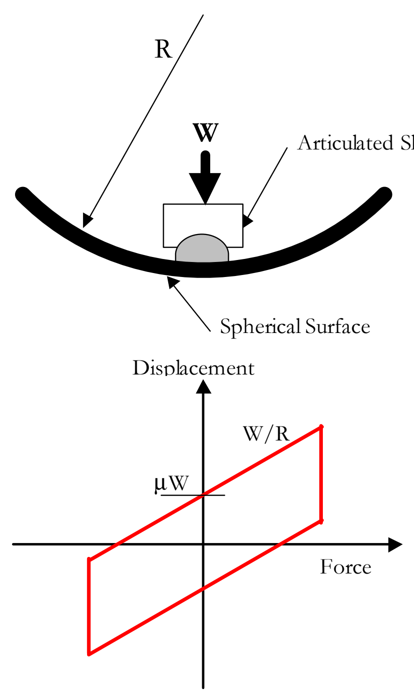
For preliminary design a constant coefficient of friction of about 10% is usually assumed. For detailed analysis, the element model should include the variation with pressure and velocity.

7.2.6 Curved Slider (Friction Pendulum) Bearings

Although a number of curved shapes are possible, the only curved sliding bearing which has been extensive used is a patented device in which the sliding surface is spherical in shape rather than flat, termed the Friction Pendulum System. The schematic characteristics of this device are shown in Figure 7-7.

The isolator provides a resistance to service load by the coefficient of friction, as for a flat slider. Once the coefficient of friction is overcome the articulated slider moves and because of the spherical shape a lateral movement is accompanied with a vertical movement of the mass. This provides a restoring force, as shown in the hysteresis shape in Figure 7-7.

Figure 7-7 Curved Slider Bearing



The bearing properties are defined by the coefficient of friction, the radius of the sphere and the supported weight. The post-sliding stiffness is defined by the geometry and supported weight, as W/R.

The total force resisted by a spherical slider bearing is directly proportional to the supported weight. If all isolators in a project are of the same geometry and friction properties and are subjected to the same displacement then the total force in each individual bearing is a constant times the supported weight. Because of this, the center of stiffness and center of mass of the isolation system will coincide and there will be no torsion moment. Note that this does not mean that there will be no torsion movements at all as there will likely still be eccentricity of mass and stiffness in the building above the isolators.

7.2.7 Ball and Roller Bearings

Although roller bearings are attractive in theory as a simple means of providing flexibility there do not seem to be any practical systems based on ball or roller bearings available. A ball system is under development using a compressible material, which deforms as it rolls providing some resistance to service loads and energy dissipation (the Robinson RoBall). Preliminary results have been presented at conferences and the device appears to have promise, especially for low mass applications. More detail should become available in the near future.

Solid ball and roller bearings constructed of steel or alloys usually have the problem of flattening of the contact surface under time if they are subjected to a high stress, as they would be under buildings and bridges. This appears to have restricted their use. Also, they do not provide either resistance to service loads or damping so would need to be used in parallel with other devices.

7.2.8 Supplemental Dampers

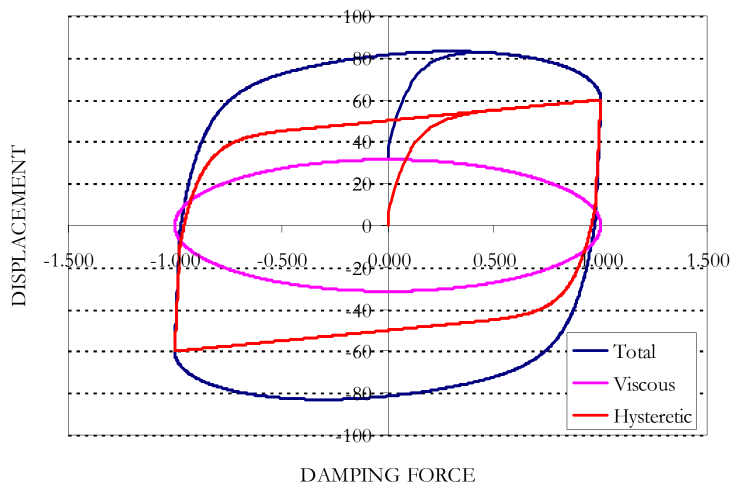
Systems which do not have an inherent restoring force and/or damping, such as elastic bearings, sleeved piles or sliding bearings, may be installed in parallel with dampers. These devices are in the same categories used for in-structure damping, a different form of passive earthquake protection. Supplemental damping may also be used in parallel with damped devices such as LRBs or HDR bearings to control displacements in near fault locations.

External dampers are classified as either hysteretic or viscous. For hysteretic dampers the force is a function of displacement, for example a yielding steel cantilever. For viscous dampers the force is a function of velocity, for example, shock absorbers in an automobile.

For an oscillating system the velocity is out of phase with the displacement and the peak velocity occurs at the zero displacement crossing. Therefore, viscous damping forces are out of phase with the elastic forces in the system and do not add to the total force at the maximum displacement. Conceptually, this is a more attractive form of damping than hysteretic damping.

In practice, if a viscous damper is used in parallel with a hysteretic isolator then there is a large degree of coupling between the two systems, as shown in Figure 7-8. The maximum force in the combined system is higher than it would be for the hysteretic isolator alone. If the viscous damper has a velocity cut-off (a constant force for velocities exceeding a pre-set value) then the coupling is even more pronounced.

FIGURE 7-8 VISCOUS DAMPER IN PARALLEL WITH YIELDING SYSTEM



Practically, it is difficult to achieve high levels of viscous damping in a structure responding to earthquakes. The damping energy is converted to heat and materials exhibiting highly viscous behavior, such as oil, tend to become more viscous with increasing temperature. Therefore, the dampers lose effectiveness as the earthquake amplitude and duration increases unless a large volume of material is used.

These factors have restricted the number of suppliers of viscous dampers suited for earthquake type loads. Hardware tends to be declassified military devices and is expensive.

For either viscous or hysteretic dampers, the damping contributed to the total isolation system is calculated from the total area of the hysteretic loop at a specified displacement level. This loop area is then added to the area from other devices such as lead-rubber bearings. These concepts are the same as used for in-structure damping and energy dissipation – see the HCG Design Guidelines on this topic for further information.

7.2.9 ADVANTAGES AND DISADVANTAGES OF DEVICES

Table 7-1 summarizes the advantages and disadvantages of the most commonly used device types. Note that although disadvantages may apply to a generic type, some manufacturers may have specific procedures to alleviate the disadvantage. For example, static friction is a potential disadvantage of sliding bearings in general but manufactures of devices such as the Friction Pendulum System may be able to produce sliding surfaces that are not subject to this effect.

Some factors listed in Table 7-1 are not disadvantages of the device itself but may be a design disadvantage for some projects. For example, the LRBs and HDR bearings produce primary and secondary (P-Δ) moments which are distributed equally to the top and bottom of the bearing and so these moments will need to be designed for in both the foundation and structure above the isolators.

For sliding systems the total P-Δ moment is the same but the sliding surface can be oriented so that the full moment is resisted by the foundation and none by the structure above (or vice versa).

The advantages and disadvantages listed in Table 7-1 are general and may not be comprehensive. On each project, some characteristics will be more important than others. For these reasons, it is not advisable to rule out specific devices too early in the design development phase. It is usually worthwhile to consider at least a preliminary design for several type of isolation system until it is obvious which system(s) appear to be optimum. It may be advisable to contact manufacturers of devices at the early stage to get assistance and ensure that the most up-to-date information is used.

TABLE 7-1 DEVICE ADVANTAGES AND DISADVANTAGES

	Advantages	**Disadvantages**
Elastomeric	Low in-structure accelerations Low cost	High displacements Low damping No resistance to service loads P-Δ moments top and bottom
High Damping Rubber	Moderate in-structure accelerations Resistance to service loads Moderate to high damping	Strain dependent stiffness and damping Complex analysis Limited choice of stiffness and damping Change in properties with scragging P-Δ moments top and bottom
Lead Rubber	Moderate in-structure accelerations Wide choice of stiffness / damping	Cyclic change in properties P-Δ moments top and bottom
Flat Sliders	Low profile Resistance to service loads High damping P-Δ moments can be top or bottom	High in-structure accelerations Properties a function of pressure and velocity Sticking No restoring force
Curved Sliders	Low profile Resistance to service loads Moderate to high damping P-Δ moments can be top or bottom Reduced torsion response	High in-structure accelerations Properties a function of pressure and velocity Sticking
Roller Bearings	No commercial isolators available.	
Sleeved Piles	May be low cost Effective at providing flexibility	Require suitable application Low damping No resistance to service loads
Hysteretic Dampers	Control displacements Inexpensive	Add force to system
Viscous Dampers	Control displacements Add less force than hysteretic dampers	Expensive Limited availability

8 ENGINEERING PROPERTIES OF ISOLATORS

8.1 SOURCES OF INFORMATION

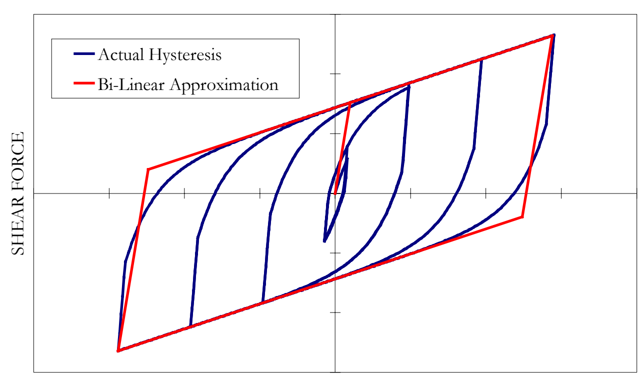
The plain rubber, high damping rubber and lead rubber isolators are all based on elastomeric bearings. The following sections describe the properties of these types of bearing manufactured from natural rubber with industry standard compounding. High damping rubber bearings are manufactured using proprietary compounds and vary from manufacturer to manufacturer. Some examples are provided of high damping rubber but if you wish to use this type of device you should contact manufacturers for stiffness and damping data.

Much of the data in this section has been provided by Skellerup Industries for projects in which we have been involved. We have also used LRBs from Dynamic Isolation Systems, Inc. and the properties of their devices are generally similar to those from Skellerup.

8.2 ENGINEERING PROPERTIES OF LEAD RUBBER BEARINGS

Lead rubber bearings under lateral displacements produce a hysteresis curve which is a combination of the linear-elastic force-displacement relationship of the rubber bearing plus the elastic-perfectly plastic hysteresis of a lead core in shear. The lead core does not produce a perfectly rectangular hysteresis as there is a "shear lag" depending on the effectiveness of the confinement provided by the internal steel shims. This is discussed further in the chapter of design procedures.

FIGURE 8-1 LEAD RUBBER BEARING HYSTERESIS



The resultant hysteresis curve, as shown in Figure 8-1, has a curved transition on unloading and reloading. For design and analysis an equivalent bi-linear approximation is defined such that the area under the hysteresis curve, which defines the damping, is equal to the measured area. It is possible to model the bearing with a continuously softening element but this has only been implemented for two-dimensional models in DRAIN-2D and so is not often used.

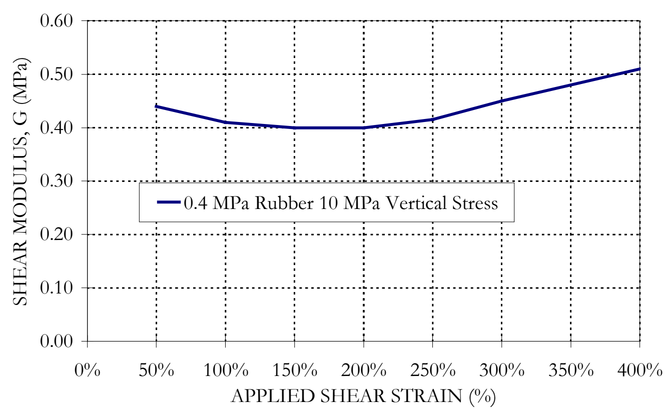
8.2.1 Shear Modulus

Elastomeric and lead rubber bearings are usually manufactured using rubber with a shear modulus at 100% strain ranging from about 0.40 MPa to 1.20 MPa.

Typically, the rubber used for LRBs has only a slight dependence on applied strain, unlike the high damping rubber bearing, which is specifically formulated to have a high dependence on strain. Figure 8-2 shows the variation in shear modulus with shear strain for a rubber with a nominal shear modulus of 0.40 MPa. The shear modulus is about 10% higher than the nominal values for strains of 50%. Above 250% shear strain the rubber stiffens such that at 400% shear strain the modulus is 30% above the nominal value.

As for all elastomeric bearings, the shear modulus has some dependence on vertical load. However, unless the vertical load is very high, the variation with vertical load is low, generally less than 10% and most often less than 5%.

Figure 8-2 : Rubber Shear Modulus



8.2.2 Rubber Damping

Many technical publications on elastomer properties refer to the loss angle. The rubber loss angle, δ, is defined as the phase angle between stress and strain and is used in rubber technology to define the loss factor (or loss tangent) which is defined as the ratio between the loss modulus and the storage modulus:

$$\tan\delta = \frac{G''}{G'}$$

where G" is the out-of-phase shear modulus and G' the in-phase shear modulus. This is a measure of the damping in the material. The materials standards ASTM D2231-94 provides *Standard Practice for Rubber Properties in Forced Vibration* but this is not generally used in structural engineering applications as it relates to smaller strain levels than are used for seismic isolation.

To obtain damping properties for elastomeric bearings, it is usual to use the results from full size bearing tests rather than single rubber layers as specified in ASTM D2231. This is because factors such as flexing of the steel shims affect the total energy loss of the system. Damping is calculated from these tests using the ratio of the area of the hysteresis loop (analogous to the loss modulus) to the elastic strain energy (analogous to the storage modulus).

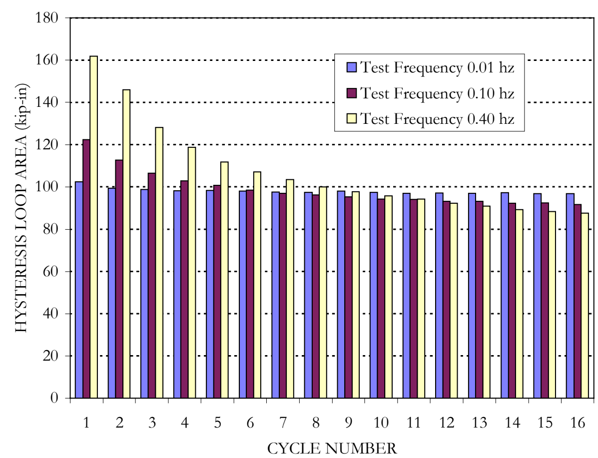
Most lead rubber bearings use a medium to low modulus natural rubber which is not compounded to provide significant viscous damping by hysteresis of the rubber material. All damping is assumed to be provided by the lead cores.

8.2.3 Cyclic Change in Properties

For lead rubber bearings the effective stiffness and damping are a function of both the vertical load and the number of cycles. There is a more pronounced effect on these quantities during the first few cycles compared to elastomeric bearings without lead cores.

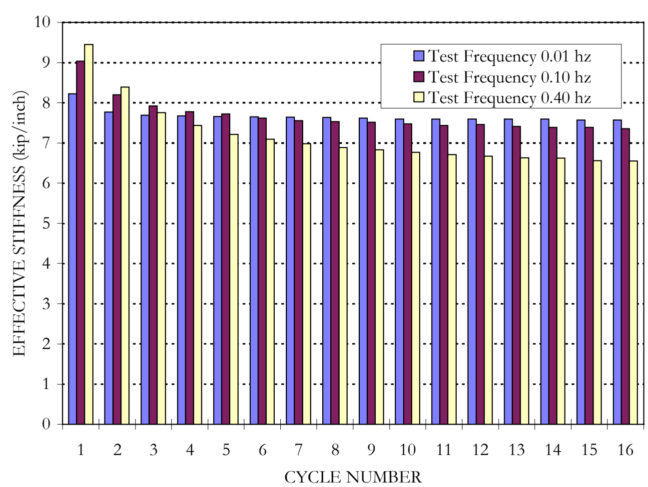
Test results have been published by Skellerup Industries for an extensive series of dynamic tests with varying load levels and shear strains to quantify these effects. Figures 8-3 and 8-4 list the variation in hysteresis and loop area versus cycle number for 380 mm (15") diameter lead rubber bearings. The test results plotted are for 100% shear strain and a vertical load of 950 KN (211 kips), corresponding to a stress of 9 MPa (1.3 ksi).

Figure 8-3: Variation In Hysteresis Loop Area



At slow loading rates there is a relatively small drop in loop area, A_h, and effective stiffness, K_{eff} , with increasing cycles. For faster loading rates, the values at the first cycles are higher but there is a larger drop off. The net effect is that the average values over all cycles are similar for the different loading rates.

Figure 8-4 : Variation In Effective Stiffness



The design procedures and prototype test requirements are such that the design hysteresis loop area and effective stiffness are required to be matched by the average of three cycle tests at the design displacement. This test is considered to best match the likely earthquake demand on the bearings. The maximum reduction in loop area will be about 1% per cycle for the first 10 cycles but then the hysteresis loop stabilizes. The actual maximum reduction in loop area is a function of the dimensions of the isolator and lead core and the properties of the elastomer.

There appears to be a size dependence on the variation in effective stiffness and hysteresis loop area with increasing number of cycles when bearings are tested at the actual expected frequency of response. For practical reasons, there are few test results of large isolation bearings which have been subjected to multiple cycles of the design displacement at the actual expected frequency of loading. One example is the Skellerup Industries tests of the isolation bearings for the Benecia-Martinez bridge. These were large isolators with very large lead cores, approaching the maximum size likely to be used for LRBs. As such, the measured changes with increasing cycles are probably the extreme which might be experienced with this type of isolator.

TABLE 8-1 BENECIA-MARTINEZ ISOLATORS

	Diameter (mm)	Height (mm)	Core Size (mm)	Applied Displacement (mm)	Shear Strain
Type 1	820	332	168	254	139%
Type 2	870	332	184	305	167%
Type 2M	1020	332	188	305	167%
Type 3	1020	349	241	280	163%

Figure 8-5 plots the ratio of hysteresis loop area (EDC = Energy Dissipated per Cycle) measured from each cycle to the requirement minimum loop area in the specification. Values are plotted for each of 15 cycles, which were applied at a frequency of 0.5 hz (corresponding to the isolation period of 2 seconds).

FIGURE 8-5 CYCLIC CHANGE IN LOOP AREA

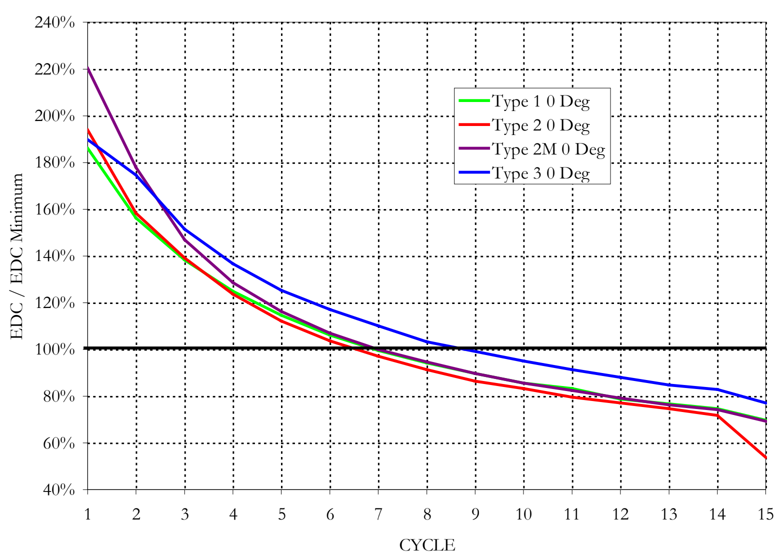
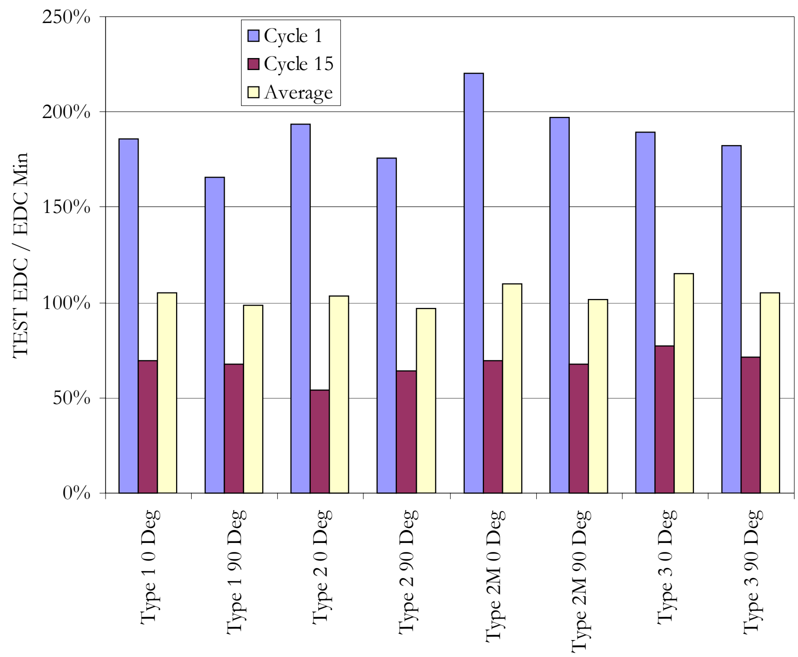


Figure 8-5 shows that the initial loop area is well above the specified minimum and remains higher for between 6 and 8 cycles. By the end of the 15 cycles the EDC has reduced to about 60% of the specified value. Figure 8-5 plots the ratios from the 1st and 15th cycles and the mean ratios over all cycles. This shows that the mean value is quite close to the specified minimum.

The reduction in loop area is apparently caused by heat build up in the lead core and is a transient effect. This is demonstrated by the results in Figure 8-6 as each isolator was tested twice, once at zero degrees and the bearing was then rotated by 90 degrees and the test repeated. As the figure shows, the initial EDC for the Cycle 1 of the second test was similar to the 1st cycle of the preceding test, not the 15th cycle. This indicates that most of the original properties were recovered and it is likely than total recovery would have occurred with a longer interval between tests.

FIGURE 8-6 MEAN CYCLIC CHANGE IN LOOP AREA



Actual earthquakes would rarely impose anything like 15 cycles at the design displacements. Most time history analyses show from 1 to 3 cycles at peak displacements and then a larger number of cycles at smaller displacements. If there are near fault effects there is often only a single cycle at the peak displacement. Figure 8-5 shows that the LRBs will provide at least the design level of damping for this number of cycles.

8.2.4 AGE CHANGE IN PROPERTIES

The rubber tests on compounds used for LRBs show an increase in hardness by up to 3 Shore A after heat aging. This increase in hardness is equivalent to an increase in shear modulus of 10%.

The increase in shear modulus would have a lesser effect on the total bearing stiffness as the lead core yield force is stable with time.

In service, the change in hardness for bearings would be limited to the outside surface since the cover layer prevents diffusion of degradants such as oxygen into the interior. Therefore, average effects would be less than the 10% value. For unprotected natural rubber in service over 100 years (for example, Rail Viaduct in Melbourne, Australia) the deterioration was limited to approximately 1.5 mm (0.06 inches) from the exposed surface.

There is not a great database of information on direct measurement of the change in stiffness properties with time of loaded elastomeric bearings. One example was machine mountings manufactured in 1953 and in service continuously in England. In 1983, after 30 years, two test bearings which had been stored with the machine were tested again and were found to have increased in stiffness by 15.5% and 4.5%. A natural rubber bearing removed from a freeway bridge in Kent showed an increase in shear stiffness of about 10% after 20 years service.

Since the time that the bearings above were manufactured, considerable advances have been made in environmental protection of the bearings. It is predicted that changes in stiffness of the elastomer will be no more than 10% over the design life of the isolators. The net effect on isolator effective stiffness at seismic displacements would be about one-half this value. Damping would not be effected by aging of LRBs as the rubber damping is negligible compared to that provided by the lead core.

8.2.5 Design Compressive Stress

The design procedures used to calculate vertical load capacity are based on a rated load (limiting strain) approach, as incorporated in codes such as AASHTO and BS5400. The effective allowable compressive stress is a function of (1) the ultimate elongation of the rubber (2) the safety factor applied to the ultimate elongation (3) the bearing plan size (4) the bearing shape factor and (5) the applied shear strain (see Chapter 9).

For long term gravity loads (displacement = 0) a factor of 1/3 is applied to the elongation at break. For short term seismic loads (displacement > 0) a factor of 0.75 is applied to the elongation for DBE loads and 1.0 for MCE loads.

Additional rules are used based on experience to ensure that the bearings will perform satisfactorily. For example, it is generally required that the effective bearing area (area of overlap between top and bottom plates) be at least 20% of the gross area at maximum displacement.

The result of this procedure is that the allowable compressive stress is a function of the bearing size and the applied displacement.

Ultimate compressive stresses are calculated by the same procedure as for allowable stresses except that a factor of 1.0 is applied to the elongation at break to obtain the load capacity.

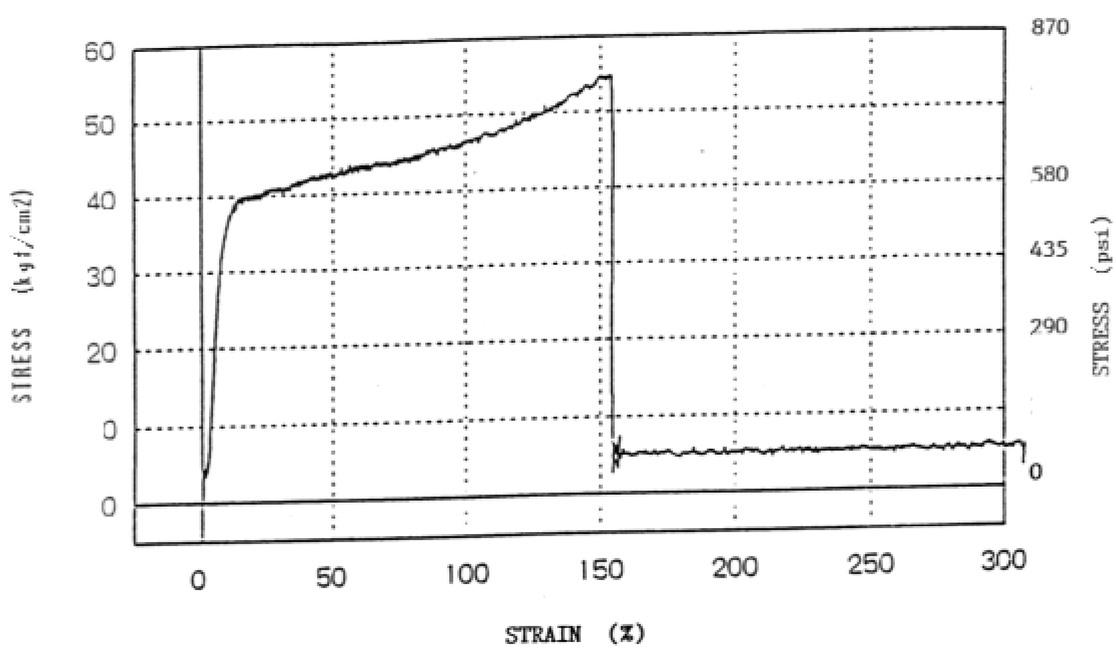
8.2.6 Design Tension Stress

Elastomeric based bearings such as LRBs and HDR bearings have in the past been designed such that tension does not occur. This is because there is little design information for rubber bearings under this type of load. As successive code revisions have increased seismic loads, it has provided very difficult to complete isolation designs such that no tension occurs and some designs do permit tension on the isolators.

Provided high quality control is exercised during manufacture, elastomeric bearings can resist a high tension without failure. Skellerup Industries bearings (without lead cores) have been tested to a tensile strain of 150% at failure, as shown in Figure 8-7. The tension stiffness is approximately elastic to a stress of 4 MPa at a strain of approximately 15%. The stiffness then reduces as cavitation occurs and remains at a low stiffness to a strain of 150%.

The rubber used for the bearing in Figure 8-7 has a shear modulus G = 1.0 MPa. The isolator design procedures permit an ultimate compression stress of 3G, which would permit a tensile stress of 3 MPa (30 kgf/cm²) for this bearing. As shown in Figure 8-7, this level of stress provides an adequate factor of safety before cavitation occurs.

Figure 8-7 Tension Test on Elastomeric Bearing



The tests above were for plain bearings under pure tension. The Skellerup lead rubber bearings for the Berkeley Civic Center were tested under combined shear and tension at close to the design limit

(2.4G versus 3G limit) to a shear strain of 225% (Table 8-2). The bearings were undamaged under these conditions. The maximum uplift displacement was approximately 12 mm (½”).

TABLE 8-2 COMBINED SHEAR AND TENSION TESTS

	Type A	Type B
Diameter (mm)	920	970
Tension Force (KN)	549	608
Tension Stress (MPa)	0.89 (2.4G)	0.88 (2.4G)
Displacement (mm)	686	686
Shear Strain	225%	225%

8.2.7 MAXIMUM SHEAR STRAIN

As discussed above for allowable vertical compressive stresses, the load capacity is calculated based on a total strain formulation where the strain due to compression and the applied shear strain are combined and required to be less than a specified fraction of the elongation at break.

The converse applies for ultimate shear strain, where the maximum shear strain that can be applied depends on the concurrent vertical strain. Although the formulas produce a maximum shear strain based on concurrent vertical loads, empirical limits are also applied to the shear strain based on experimental evidence. Generally, the limiting shear strain is taken as 150% for DBE loads and 250% for MCE loads, unless the design formulas provide a lower limit.

Testing of Skellerup Industries lead rubber bearings at high shear strain levels have shown that failure in lead rubber bearings occurs between 300% and 350% shear strain. The bearings without lead cores can survive imposed shear strains of 400%. Table 8-3 summarizes test results for Skellerup lead rubber bearings (LRB), high damping rubber bearings (HDR) and LRBs without the lead core (R).

TABLE 8-3: HIGH SHEAR TEST RESULTS

BEARING	Type	Diam. D0 (mm)	Vert Stress (MPa)	Max. Disp (mm)	Shear Strain	Effect. Area	Total Strain	
HITEC 150 kip	LRB	450	4.7	335	320%	0.11	925%	(1)
HITEC 500 kip	LRB	620	8.0	434	339%	0.16	795%	(1)
HITEC 750 kip	LRB	620	12.0	381	190%	0.24	614%	(2)
PEL	HDR	450	8.8	305	340%	0.22	762%	(2)
JQT Tension	R	350	-5.5	84	155%	Vert	Strain	(3)
JQT Shear/Tens	R	350	-2.7	140	250%	0.61	363%	(4)
JQT Shear/Comp.	R	500	2.5	250	250%	0.40	282%	(5)
	R	500	7.5	250	250%	0.40	347%	
	R	500	10.0	400	400%	0.11	854%	
	R	500	15.0	250	250%	0.40	444%	
	R	500	20.0	250	250%	0.40	509%	

NOTES TO TABLE 8-1:

(1) The 150 kip and 500 kip HITEC specimens were tested to failure.

(2) The 500 kip HITEC bearing was cycled to 15" displacement but the equipment was not sufficient to perform the failure test. The 18" bearing in the PEL test was cycled to 340% strain without failure.

(3) The Japanese Tension test was to failure under pure tension at an ultimate tensile strain of 155%.

(4) The Japanese combined shear/tension test was for 5 cycles at 150% and 5 cycles at 250% shear strain. Failure did not occur.

(5) Failure did not occur in any of the Japanese combined shear/compression tests

(6) LRB indicates Lead Rubber Bearing, HDR indicates high damping rubber bearings, R indicates LRB without lead core.

8.2.8 BOND STRENGTH

The bond strength defines the adhesion between the rubber layers and the internal steel shims. Specifications typically require that the adhesive strength between the rubber and steel plates be at least 40 lb/inch when measured in the 90° peel test specified by ASTM D429, Method B. Failure is required to be 100% rubber tear. All compounds we use for LRBs should meet this requirement.

8.2.9 VERTICAL DEFLECTIONS

The initial vertical deflections under gravity loads are calculated from standard design procedures for elastomeric bearings. For bearings with a large shape factor the effects of bulk modulus are important and are included in the calculation of the vertical stiffness on which deflection calculations are based. Elastomeric bearings are stiff under vertical loads and typical deflections under dead plus live load are usually of the order of 1 mm to 3 mm (0.04 to 0.10 inches).

8.2.9.1 Long Term Vertical Deflection

Creep is defined as the increase in deformation with time under a constant force and so is the difference between short term and long term deflection. In rubber, creep consists of both physical creep (due to molecular chain slippage) and chemical creep (due to molecular chain breakage). For structural bearings the physical deformation is dominant. Chemical effects, for example oxidation, are minimal since the bulk of the bearing prevents easy diffusion of chemicals into the interior. Therefore, chemical effects can be ignored.

Natural rubber generally offers the greatest resistance to creep compared to all other rubbers. The actual values depend on the type and amount of filler as well as the vulcanization system used.

Creep usually does not exceed more than 20% of the initial deformation in the first few weeks under load and at most a further 10% increase in deformation after a period of many years. The maximum long term deflections for design purposes are conservatively taken to be 1.5 times the short term values.

A detailed case study has been made of a set of bearings over a 15 year period. The building, Albany Court in London, was supported on 13 bearings of capacity from 540 KN to 1800 KN (120 kips to 400 kips). Creep was less than 20% of the original deflection after 15 years.

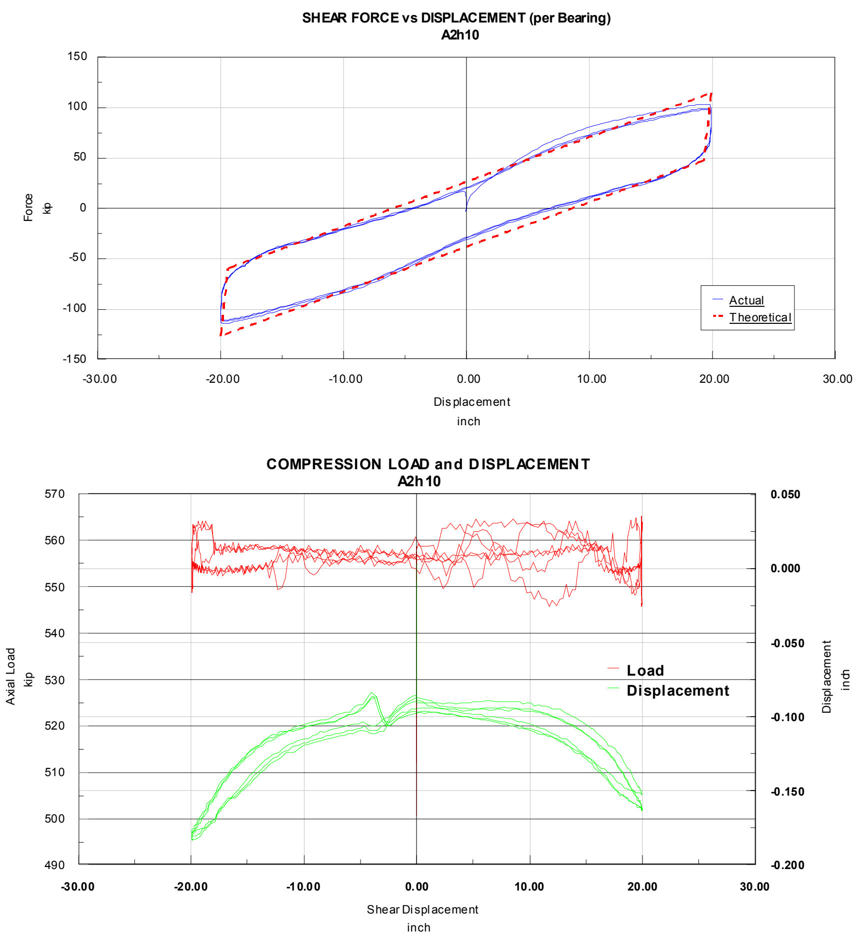
8.2.9.2 Vertical Deflection under Lateral Load

Under lateral loads there will be some additional vertical deflections as the bearing displaces laterally. Generally, this deformation is relatively small. Figure 8-8 is an example of a combined compression shear test in which vertical deformations were measured. This is from the Berkeley Civic Center test series referenced above for tension loads.

The bearing is displaced to a shear displacement of 508 mm (20") under a vertical load of 2500 KN (550 kips). The initial vertical deflection when the load is applied is 2.5 mm (0.1") which increases to a maximum of approximately 4.6 mm (0.18") at the 508 mm lateral displacement. The most severe total vertical deflection measured was 12.4 mm (0.49") at a lateral displacement of 686 mm (27").

Based on these results, allowance in design should be made for about 15 mm (0.6") vertical downward movement at maximum displacements.

FIGURE 8-8 COMBINED COMPRESSION AND SHEAR TEST



8.2.10 WIND DISPLACEMENT

For LRBs, resistance to wind loads is provided by the elastic stiffness of the lead cores. Typical wind displacements for projects have ranged from 3.5 mm (0.14") under a wind load of 0.01W to 11 mm (0.43") under a load of 0.03W. The cores are usually sized to have a yield level at least 50% higher than the maximum design wind force.

8.2.11 COMPARISON OF TEST PROPERTIES WITH THEORY

The discussions above, and the design procedures in Chapter 9, are based on theoretical formulations for LRB design. A summary of test results from nine projects in Table 8-4 compares the theoretical values with what can be achieved in practice:

TABLE 8-4 SKELLERUP INDUSTRIES LRB TEST RESULTS

Project	Plan Size mm	Δ mm	Design K_{EFF} KN/mm	Design EDC KN-mm	Test K_{EFF} KN/mm	Test EDC KN-mm	K_{TEST} / K_{DESIGN}	EDC_{TEST} / EDC_{DESIGN}
Hutt Health	662	264	1.42	115720	1.41	143425	99%	124%
Taiwan C347	875	58.2	11.51	640	12.02	741	104%	116%
Taiwan C258	420 365 1100 1000	158 104 113 125	0.91 0.91 9.51 11	220 90 1240 1790	0.922 1.008 10.336 11.406	286 117 1712 2381	101% 111% 109% 104%	130% 130% 138% 133%
Arik Bridge	686 686	76 76	6.00 6.85	511 407	5.04 6.00	542 397	84% 88%	106% 98%
Three Mile	686 686 686	206 185 261	12.69 15.09 12.57	127102 136246 202082	14.17 14.65 12.11	184823 167792 266662	112% 97% 96%	145% 123% 132%
Maritime Museum	500	169	1.27	81870	1.20	97425	94%	119%
Benecia Martinez	813 864 1016 1016	254 305 305 279	13.28 14.14 19.63 28.76	166764 233058 240030 369989	13.97 14.32 20.44 29.44	201625 303581 346672 554355	105% 101% 104% 102%	121% 130% 144% 150%
St John's	1219	406	13.95	500748	13.43	482803	96%	96%
Berkeley Civic	914 965	508 508	6.81 7.45	240259 241059	7.16 7.89	294894 307124	105% 106%	123% 127%

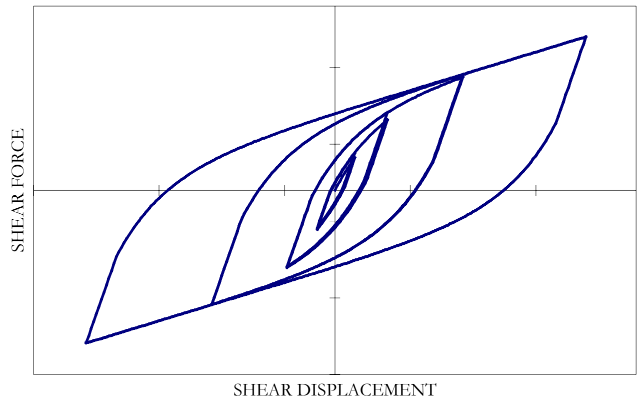
Of the 19 isolators tested in Table 8-4, the effective stiffness in 13 was within ±5% of the design value and a further 3 were within ±10%. One test produced a stiffness 12% above the design value and two were respectively –12% and –16% lower than the design values. These last two were for the same project (Arik Bridge) and were the result of a rubber shear modulus lower than specified. Specifications generally require the stiffness to be within ±10% for the total system and allow ±15% variation for individual bearings.

The hysteresis loop area (EDC) exceeded the design value for 17 of the 19 tests. Three tests were lower than the design value, by a maximum of 4%. Specifications generally require the EDC to be at least 90% of the design value, with no upper limit.

8.3 Engineering Properties of High Damping Rubber Isolators

High damping rubber bearings are made of specially compounded elastomers which provide equivalent damping in the range of 10% to 20%. The elastomer provides hysteretic behavior as shown in Figure 8-9.

FIGURE 8-9 HIGH DAMPING RUBBER HYSTERESIS

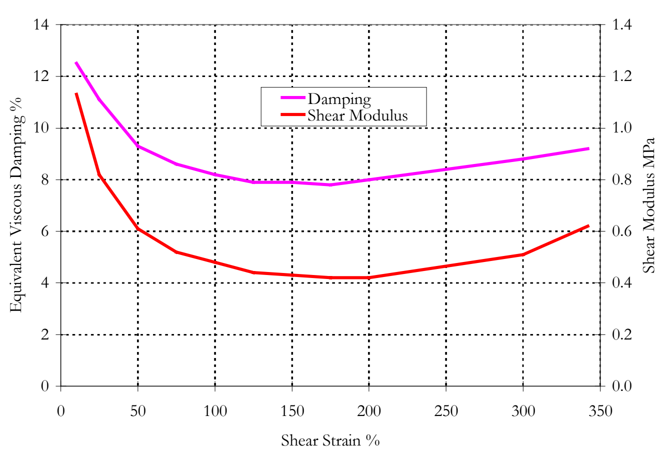


Whereas the properties of lead rubber bearings have remained relatively constant over the last few years, there have been continuous advances in the development of high damping rubber compounds. These compounds are specific to manufacturers as they are a function of both the rubber compounding and the curing process.

Although the technical literature contains much general information on HDR, there is not a lot of technical data specific enough to enable a design to be completed. The information in these sections relates to a specific compound developed by Skellerup Industries and used for the Missouri Botanical Gardens project. This can be used for a preliminary design. In terms of currently available compounds it is not a particularly high damping formulation, so design using these properties should be easily attainable. If you wish to use HDR, the best approach is probably to issue performance based specifications to qualified manufacturers to get final analysis properties.

8.3.1 Shear Modulus

FIGURE 8-10 HDR SHEAR MODULUS AND DAMPING



The shear modulus of a HDR bearing is a function of the applied shear strain as shown in Figure 8-10. At low strain levels, less than 10%, the shear modulus is 1.2 MPa or more. As the shear strain increases the shear modulus reduces, in this case reaching a minimum value of 0.4 MPa for shear strains between 150% and 200%. As the shear strain continues to increase the shear modulus increases again, for this compound increasing by 50% to 0.6 MPa at a strain of 340%.

The initial high shear modulus is a

characteristic of HDR and allows the bearings to resist service loads such as wind without excessive movement.

The increase in shear stiffness as strains increases beyond about 200% can be helpful in controlling displacements at the MCE level of load, which may cause strains of this magnitude. However, they have the disadvantage of increasing force levels and complicating the analysis of an isolated structure on HDR bearings.

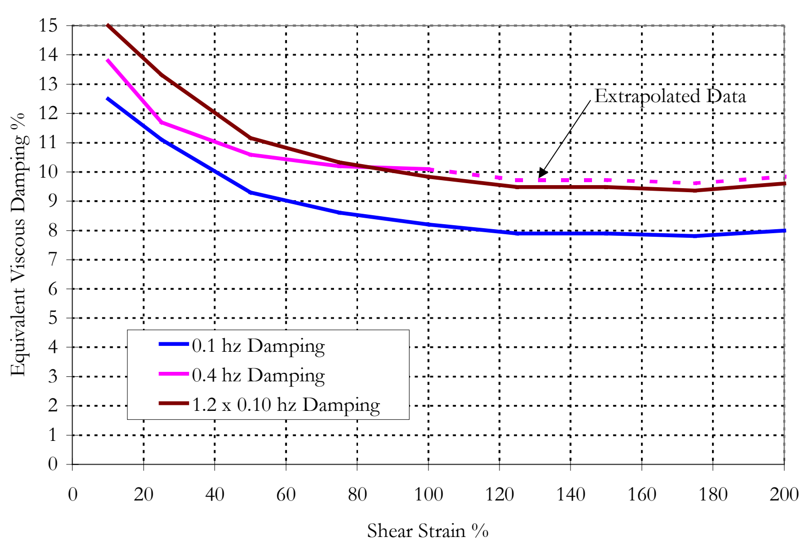
8.3.2 Damping

Although the majority of the damping provided by HDR bearings is hysteretic in nature there is also a viscous component which is frequency dependent. These viscous effects may increase the total damping by up to 20% and, if quantified, can be used in design.

Viscous damping is difficult to measure across a full range of displacements as the power requirements increase as the displacements increase for a constant loading frequency. For this reason, viscous damping effects are usually quantified up to moderate displacement levels and the results used to develop a formula to extrapolate to higher displacements.

Figure 8-11 shows the equivalent viscous damping for a load frequency of 0.1 hz, a slow loading rate at which viscous effects can be assumed to be negligible. For strains up to 100%, the tests used to develop these results were also performed at a loading rate of 0.4 hz (period 2.5 seconds), an average frequency at which an isolation system is designed to operate.

FIGURE 8-11 VISCOUS DAMPING EFFECTS IN HDR



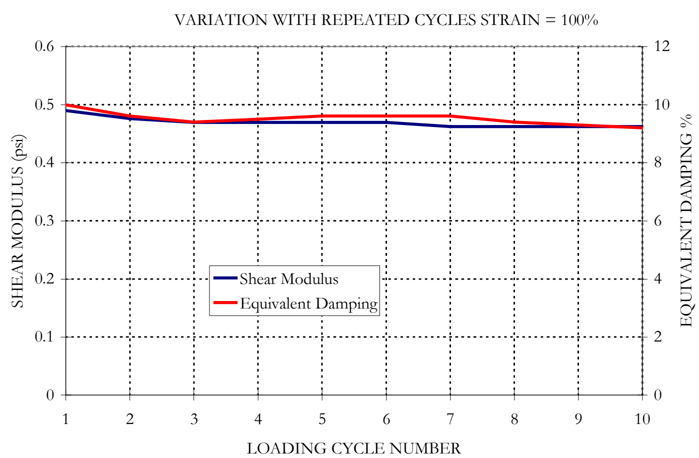
The damping at 0.4 hz was higher than that at 0.1 hz by a factor which increased with strain. At 25% the factor was 1.05 and at 100% the factor was 1.23. The frequency dependency indicates the presence of viscous (velocity dependent) damping in the elastomer. The velocity increases proportionately to the frequency and so the high frequency test gives rise to higher viscous damping forces. The tests at various strain levels are performed at the same frequencies and so the velocity increases with strain. The velocities are four times as high at 100% strain as at 25%. This is why the factor between the 0.4 hz and 0.1 hz damping increases.

The added viscous damping adds approximately 20% to the total damping for strains of 100% or greater, for this compound increasing the damping from 8% to 9.6%.

8.3.3 Cyclic Change in Properties

The properties of a HDR bearing will change under the first few cycles of loading because of a process known as "scragging". When a HDR bearing is subjected to one or more cycles of large amplitude displacement the molecular structure is changed. This results in more stable hysteresis curves at strain levels lower to that at which the elastomer was scragged. Partial recovery of unscragged properties is likely. The extent of this recovery is dependent on the compound.

Figure 8-12 Cyclic Change in Properties for Scragged HDR



When HDR bearings are specified the specifications should required one to three scragging cycles at a displacement equal to the maximum test displacement. You should request information from each manufacturer as to scragging effects on a particular compound to enable you to decide on just how many scragging cycles are needed.

Once a HDR bearing has been scragged the properties are very stable with increased number of cycles, as shown in Figure 8-12.

8.3.4 Age Change in Properties

Although most HDR compounds have a more limited service record than other natural rubber formulations the same additives to resist environmental degradation are used as for other elastomers and there is no reason to suspect that they will have a shorter design life. However, as the compounds are so specific to particular manufacturers you should request data from potential suppliers. The specifications will require the same accelerated (heat) aging tests as for lead rubber bearings.

8.3.5 Design Compressive Stress

HDR bearings are generally designed using the same formulas as for LRBs and so the comments in the sections on LRBs also apply.

8.3.6 Maximum Shear Strain

The maximum shear strains for LRBs usually have an empirical limit which may restrict the shear strain to a lesser value than permitted by the design formulas. These limits are related to performance of the lead core and so do not apply to HDR bearings. The maximum shear strain is based on the limiting strain formulas and may approach 300% for MCE loads, compared to a 250% limit for LRBs.

The higher shear strain limits for HDR bearings may result in a smaller plan size and lower profile than a LRB, for a smaller total volume. However, this also depends on the levels of damping as the displacements may differ between the two systems.

8.3.7 Bond Strength

The bond strength requirements are the same as for LRBs previously.

8.3.8 Vertical Deflections

The vertical stiffness, and so deflections under vertical loads, are governed by the same formulas as for LRBs and so will provide similar deflections for similar construction although the specific elastomer properties may cause more differences.

8.3.8.1 Long Term Vertical Deflections

HDR bearings are cured differently from LRBs and have higher creep displacements. The compression set (after 22 hours at 158°F) may be as high as 50%, compared to less than 20% for low damping rubber compounds. This may cause an increase in long term deflections and you should seek advice from the supplier on this design aspect.

8.3.9 Wind Displacements

HDR bearings generally rely on the initially high shear modulus to resist wind loads and do not require a supplemental wind restraint. The wind displacement can be calculated using compound-specific plots of shear modulus versus shear strain. This may require an iterative procedure to solve for a particular lateral wind load. There have been no reported instances of undue wind movements in buildings isolated with HDR bearings.

8.4 ENGINEERING PROPERTIES OF SLIDING TYPE ISOLATORS

Most specifications for sliding bearings require that the sliding surface be a self-lubricating polytetrafluroethylene (PTFE) surface sliding across a smooth, hard, non-corrosive mating surface such as stainless steel.

There are two types of sliding isolators that we might use:

1. Curved slider bearings (the Friction Pendulum System) providing the total isolation system.

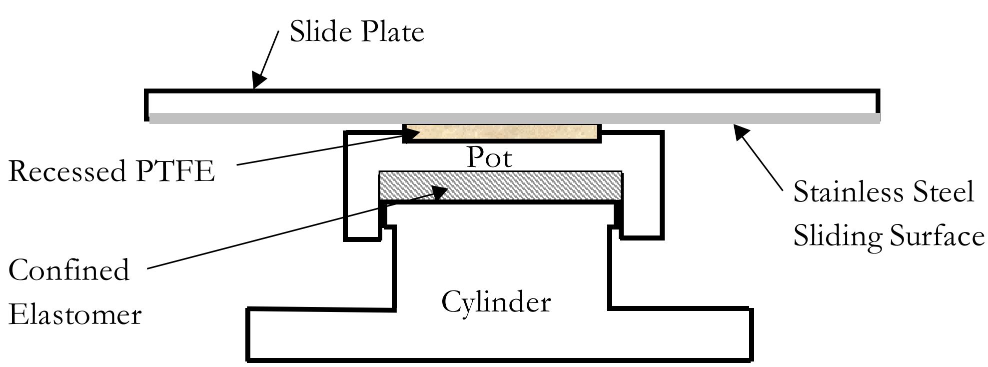
2. Sliding bearings in parallel with other devices, usually with HDR or LRB.

The former application uses proprietary products and the detailed information on the sliding surfaces, construction etc. will be provided by the supplier. The information supplied should provide the information described here for other sliding devices.

Most applications where we have used sliding bearings to provide part of the isolation system have been based on "pot" bearings, a commercially available bearing type which has long been used for non-seismic bridge bearings. For light loads, such as under stairs, a simpler sliding bearing can be constructed by bonding the PTFE to an elastomeric layer.

Pot-type bearings have a layer of PTFE bonded to the base of the "pot" sliding on a stainless steel surface. The "pot" portion of the bearing consists of a steel piston, inside a steel cylinder, bearing on a confined rubber layer. The pot allows rotations of typically up to at least 0.20 radians. Figure 8-13 shows a schematic section of the bearing.

FIGURE 8-13 SECTION THROUGH POT BEARINGS



The pot bearing in Figure 8-13 is oriented with the slide plate on top. The bearing can also be oriented with a reversed orientation and the slide plate at the bottom. The option of the slide plate on the top has the advantage that debris will not settle on the stainless steel slide surface but the disadvantage that under lateral displacements the eccentricity will cause secondary moments in the structure above the isolation plane. With the slide plate on the bottom the moments due to eccentricity will be induced in the foundation below the isolator which will often be better able to resist these moments. In this case, either wipers or a protective skirt may be required to prevent debris settling on the slide plate.

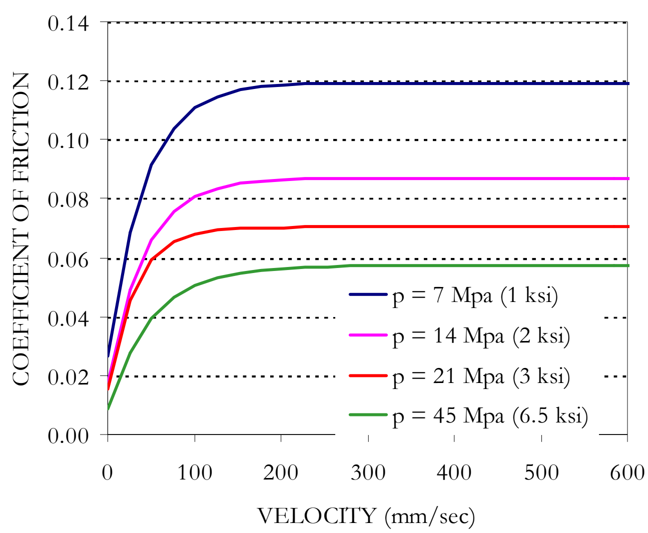
The reason for selecting pot bearings rather than simply PTFE bonded to a steel plate is that in many locations some rotational capability is required to ensure that during earthquake displacements the load is evenly distributed to the PTFE surface. This may be achieved by bonding the PTFE to a layer of rubber or other elastomer. However, the advantage of a pot bearing is that the elastomer is confined and so will not bulge or extrude under high vertical pressures. In this condition, the allowable pressure on the rubber is at least equal to that on the PTFE and so more compact bearings can be used than would otherwise be required.

Methods of protecting the sliding surface should be considered as part of supply. Pot bearings which have been installed on isolation projects previously have used wipers to clear debris from the sliding surface before it can damage the PTFE/stainless steel interface.

8.4.1 Dynamic Friction Coefficient

The coefficient of friction of PTFE depends on a number of factors, of which the most important are the sliding surface, the pressure on the PTFE and the velocity of movement.

Figure 8-14: Coefficient Of Friction For Slider Bearings



Data reported here is that developed for pot bearings based on tests of Hercules bearings which were used for the Museum of New Zealand project. For this project tests showed that the minimum dynamic coefficient for a velocity $<$ 25 mm/sec (1 in/sec) ranged from 2.5% to 8% depending on pressure. These results were from a wide range of bearing sizes and pressures. The mean coefficient of friction at low speeds was 5% at pressures less than 13.8 MPa (2 ksi) decreasing to 2% at pressures exceeding 69 MPa (10 ksi).

The high load capacity test equipment used for the full scale bearings was not suitable for high velocity tests and so the maximum dynamic friction coefficient was obtained from two sources:

1. A series of tests were performed at the University of Auckland, New Zealand (UA), using bearing sizes of 10 mm, 25 mm and 50 mm (3/8", 1" and 2") diameter. The effect of dynamic coefficient friction versus size was determined from these tests.

2. Additional data was obtained from State University of Buffalo tests performed on 254 mm (10”) bearings using the same materials (Technical Report NCEER-88-0038). This data confirmed the results from the UA tests.

Figure 8-14 plots the coefficient of friction for 254 mm (10") bearings. The test results from the UA series of tests showed some size dependence, as the maximum dynamic coefficient of friction for velocities greater than 500 mm/sec (20 inch/sec) was approximately 40% higher for 50 mm (2 inch) diameter bearings compared to 254 mm (10 inch) bearings.

8.4.2 Static Friction Coefficient

On initiation of motion, the coefficient of friction exhibits a static or breakaway value, μ_B, which is typically greater than the minimum coefficient of sliding friction. This is sometimes termed *static friction* or *stiction*. Table 8-5 lists measured values of the maximum and minimum static friction coefficient for bearing tests from 1000 KN (220 kips) to 36,500 KN (8,100 kips). These values are plotted in Figure 8-15 with a power "best fit" curve. As for the dynamic coefficient, the friction is a function of the vertical stress on the bearing.

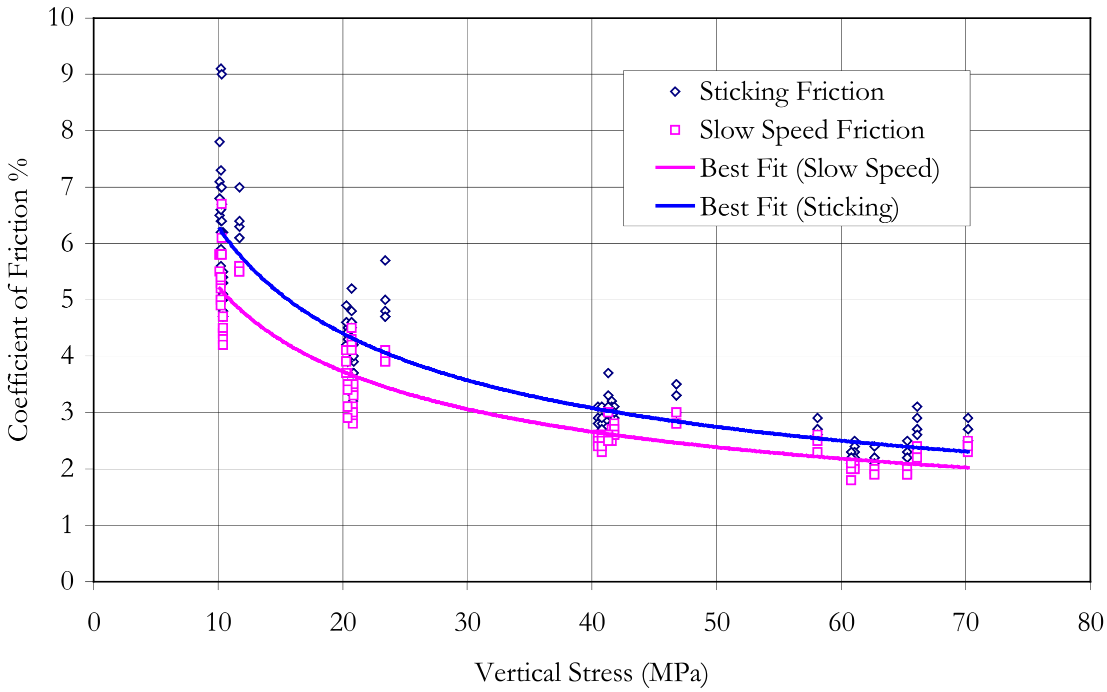
At low stresses (10 MPa) the static coefficient of friction is about 5% and the maximum sticking coefficient almost two times as high (9%). At high pressures (70 MPa) the static coefficient is approximately 2% and the sticking coefficient up to 3%.

The ratio of the maximum to minimum depends on the loading history. A test to ultimate limit state overload invariably causes a high friction result immediately after.

Table 8-5 : Minimum/Maximum Static Friction

Type	Vertical Load (kips)	Vertical Stress (ksi)	Sticking Friction Coefficient %	Minimum Speed Friction Coefficient %
36500KN 8100 kip	2028 4056 8111 11333	1.51 3.01 6.03 8.42	4.92 3.40 3.00 2.70	4.30 2.96 2.60 2.44
23500KN 5200 kip	1306 2611 5222 7833	1.51 3.03 6.06 9.09	5.48 3.90 3.00 2.22	4.58 3.40 2.68 1.94
7800KN 3900 kip	989 1978 3956 5933	1.48 2.96 5.91 8.86	6.28 4.36 2.62 2.34	5.10 3.50 2.42 2.04
15400KN 3400 kip	856 1711 3422 5133	1.46 2.94 5.87 8.81	7.00 4.54 2.90 2.22	5.74 3.94 2.52 1.96
9100KN 2000 kip	506 1011 2022 3033	1.70 3.39 6.78 10.17	6.38 4.98 3.32 2.66	5.54 3.96 2.92 2.40
3000KN 670 kip	167 333 667 1067	1.48 2.96 5.91 9.46	7.18 3.90 2.88 2.30	5.52 3.02 2.50 1.92
1000KN 220 kip	56 111 222 356	1.49 3.00 5.99 9.58	7.16 4.74 3.22 2.80	6.04 4.24 2.72 2.26

FIGURE 8-15: STATIC AND STICKING FRICTION



8.4.3 EFFECT OF STATIC FRICTION ON PERFORMANCE

An isolation system which was formed of a hybrid of flat sliding and high damping rubber bearings was studied extensively to assess the effect of static friction on the forces transmitted into the superstructure.

The sliding friction element in ANSR-L has a sliding force which is a function of the velocity and pressure on the element. The coefficient of friction is continually updated during the time history analysis as either of these parameters change.

The element also has a sticking factor where the initial coefficient of friction is factored by a sticking factor which reduces exponentially over a specified travel distance. For this project the response with a sticking factor of 2.0 was assessed.

Figure 8-16 shows the effect of the sticking factor on the time history of friction force. The structure has a weight of 400,000 KN on the sliding bearings, a static coefficient of friction of 4% and a maximum coefficient of friction of 10%. The maximum coefficient of friction produces a sliding force of 40,000 KN. Because the sticking initially occurred at the lower coefficient of friction, the sticking factor of 2.0 increased the maximum force by a lesser factor, increasing the force by 50% to 60,000 KN.

Figure 8-17 shows the force-displacement function over the same time period shown in Figure 8-16.

The isolation system comprises sliding bearings supporting 35% of the seismic weight and high damping rubber bearings supporting the remaining 65%. Figure 8-18 plots the hysteresis curves for each of these isolator components and the total hysteresis for the combined system.

The effects of the static breakaway friction are dissipated over relatively small displacements and after displacements of 50 mm or more the effects are negligible. The HDR bearings provide a force which increases with increasing displacement and so the sticking force is not as high as the force which occurs at maximum displacement when the maximum HDR force is added to the friction force at maximum velocity.

This type of evaluation can be used on projects which contain sliding bearings as one component to determine the maximum weight which can be supported on sliders such that the breakaway friction does not govern maximum forces.

FIGURE 8-16 TIME HISTORY WITH STICKING

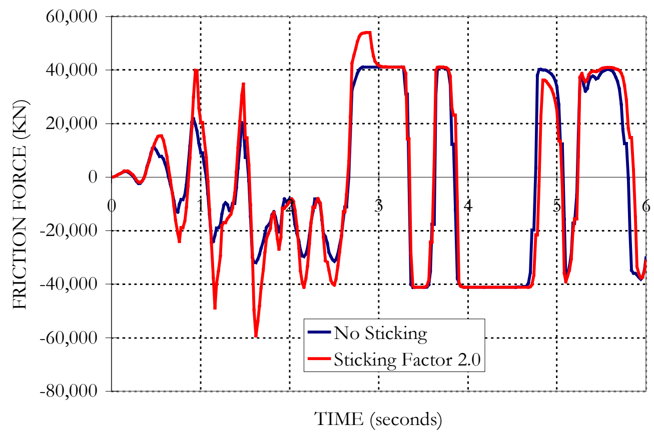


FIGURE 8-17 HYSTERESIS WITH STICKING

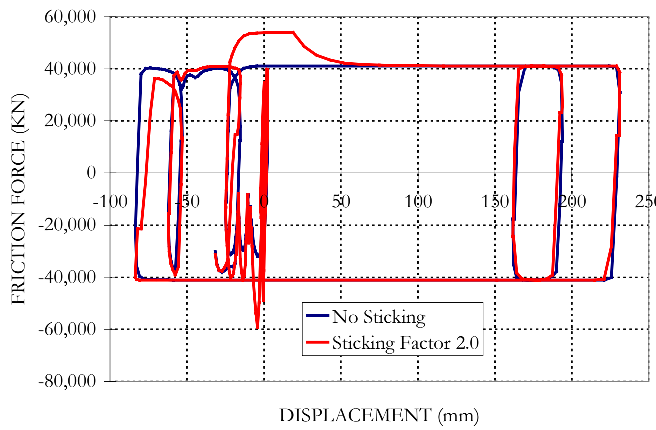
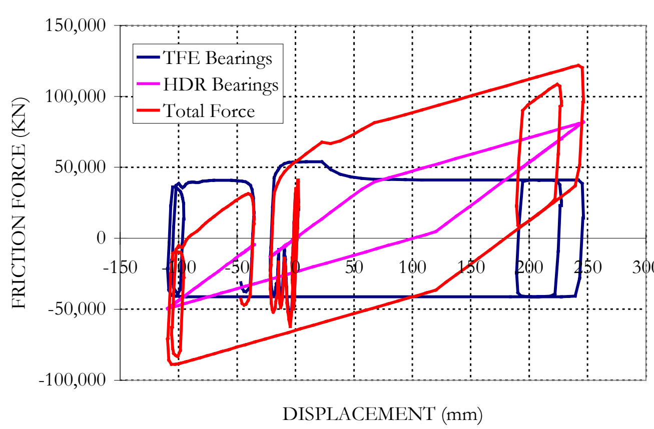


FIGURE 8-18 COMBINED HYSTERESIS WITH STICKING



8.4.4 CHECK ON RESTORING FORCE

The UBC requires systems without a restoring force to be designed for a displacement equal to three times the design displacement. This has a large impact on P-Δ forces, the size of the separation gap and the cost of separating services and components. Wherever possible, systems should be designed to provide a restoring force.

The definition of a restoring force is that the force at the design displacement is at least 0.025W greater then the force at one-half the design displacement.

Calculations for the restoring force for the example described above are listed in Table 8-6. This design just achieves the UBC definition of a system containing a restoring force. As this system had 35% of the weight on sliders, an upper limit of 30% should be used for preliminary design to ensure that the restoring force definition is achieved.

TABLE 8-6 CALCULATION OF RESTORING FORCE

Design Displacement Force at Design Displacement, F_{DD}	245 mm 121,915 KN = 0.107W
½ Design Displacement Force at ½ Design Displacement, $F_{0.5DD}$	122 mm 93,859 KN = 0.082W
$F_{DD} - F_{0.5DD} = 0.107W - 0.082W$	$0.025W \geq 0.025$ W Ok

The restoring force requirement is absolute, not earthquake specific, and so may cause problems in low seismic zone when total forces, as a fraction of seismic weight, are low. For such zones it may not be possible to use sliders as part of the isolation system and still comply with the UBC requirement for a restoring force. However, in low seismic zones it may be practical to design and detail for three times the computed seismic displacement anyway.

8.4.5 AGE CHANGE IN PROPERTIES

PTFE is about the best material known to man for corrosion resistance, which is why there is difficulty in etching and bonding it.

For base isolation use, the PTFE is dry/unlubricated and any changes over the design life will be minor. Tests confirm little change in friction over several thousand cycles such as occurs in a bridge with daily and seasonal movements due to thermal stresses.

8.4.6 Cyclic Change in Properties

As the PTFE slides on the stainless steel surface under high pressure and velocity there is some flaking of the PTFE and these flakes are deposited on the stainless steel surface. As the total travel distance increases (over 2 meters) a thin film of PTFE will build up on the stainless steel. This will result is some reduction of the coefficient of friction. A maximum MCE displacement would be about 12 m (assuming 10 cycles, 1200 mm travel per cycle). At the frequency of an isolation system μ would probably decrease about 10% over this travel.

Extreme testing performed by Hercules at the University of Sydney measured a heat build up in TFE of about 250°F after 250 cycles at ± 100 mm amplitude and 0.8 hz frequency. After 100 cycles of this load (approximately 40 m travel) the coefficient of friction had reduced to approximately one-half the original value.

8.4.7 Design Compressive Stress

Typical allowable bearing stresses for service loads is are 45 MPa (6.5 ksi) for virgin PTFE and 60 MPa (8.7 ksi) for glass filled PTFE.

8.4.8 Ultimate Compressive Stress

The ultimate bearing stress is 68 MPa (9.85) ksi for virgin TFE and 90 MPa (13.0 ksi) for glass filled TFE.

8.5 Design Life Of Isolators

Most isolation systems are based on PFTE and natural rubber bearings which have a long record of excellent in-service performance. As part of prototype testing, rubber tests including ozone testing and high temperature tests to simulate accelerated aging are performed to ensure the environmental resistance and longevity of the system.

All steel components of the elastomeric based bearings are encased in a protective cover rubber except for the load plates. These plates are usually coated with a protective paint. The protective coating system adopted for the Museum of New Zealand, which had a specified 150 year design life, was a deposited metal paint system.

8.6 FIRE RESISTANCE

Isolation bearings are generally required to achieve a fire rating equivalent to that required for the vertical load carrying assemblies.

We have used Holmes Fire and Safety to provide services for the New Zealand Parliament Building isolators and the Museum of New Zealand.

One of two approaches can be used to provide acceptable fire rating with the method used decided on depending on specific project needs:

1. Design surrounding flexible protective "skirts" for the bearings as was done for Parliament Buildings.

2. Rate the resistance of rubber itself based on vertical load, dimensions and fire loading to determine whether it is more economical to provide the fire rating by providing extra cover rubber to the bearing.

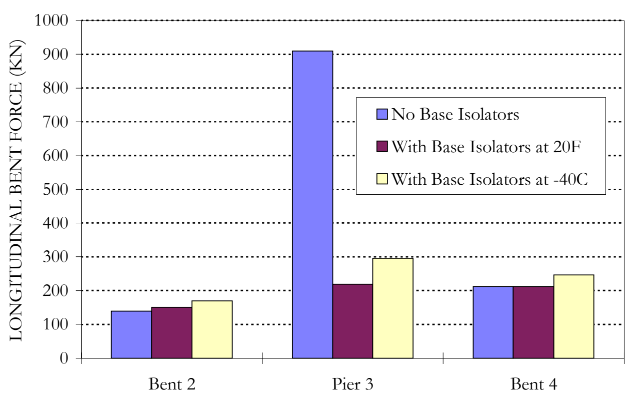
8.7 EFFECTS OF TEMPERATURE ON PERFORMANCE

Elastomeric bearings are usually compounded from natural rubber and so are subjected to temperature constraints typical to this material. The upper operating range of service temperature for natural rubber, without special compounding, is 60°C (140°F) and so the upper limit of the design temperatures for most projects will not cause any problems.

The stiffness of natural rubber is a function of temperature but within the range of -20°C to 60°C (-5°F to 140°F) the effect is slight and not significant in terms of isolation performance. Below -20°C the stiffness gradually increases as the temperature is lowered until at about –40°C (-40°F) it is double the value at 20°C (68°F). The variation in stiffness is reversible as temperature is increased.

As an example of the assessment of the effect of extreme low temperatures, the base isolation properties for a bridge project in a cold region were calculated assuming the shear modulus was increased by a factor of 2. Figure 8-19 illustrates the effect on maximum pier and bent forces in the longitudinal direction.

FIGURE 8-19 EFFECT OF LOW TEMPERATURES



The Pier 3 longitudinal force increased by 25%. The transverse Pier 3 force increased by 18%. Bent 2 and 4 forces were essentially unchanged as the force was determined by the elastic stiffness which is only a weak function of rubber shear modulus.

For this project, if a design earthquake occurred at temperatures below -20°C the pier forces could increase, with a maximum increase of about 25% at the extreme low temperature. The probability of a design level earthquake occurring while temperatures are below -20°C are probably low, although this depends on the temperature distribution at the bridge site. If the probability was considered significant, and the increased forces could lead to substructure damage, the isolators could be modified to ensure that the forces at minimum temperature did not exceed target values.

8.8 Temperature Range for Installation

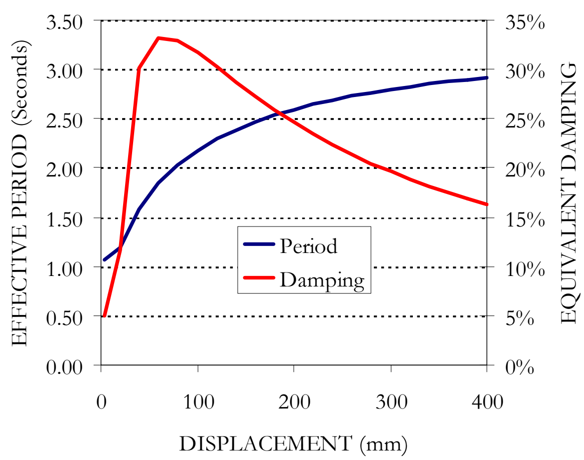
For bridge isolation projects, base isolators are designed to resist the maximum seismic displacements plus the total R+S+T (creep, shrinkage and thermal) displacements. Therefore, the temperature of installation does not matter from a technical perspective.

For aesthetic reasons it is desirable to install the bearings at as close to mean temperature as possible so that the bearings are not in a deformed configuration for most of their service life.

9 ISOLATION SYSTEM DESIGN

9.1 DESIGN PROCEDURE

FIGURE 9-1 ISOLATOR PERFORMANCE



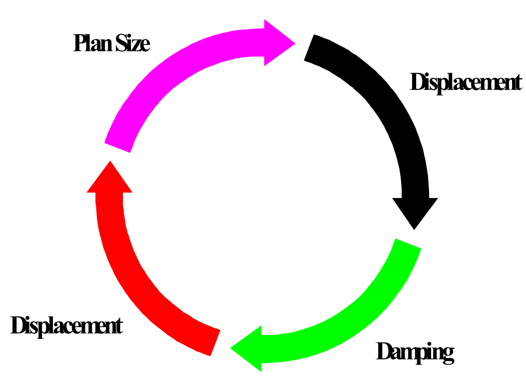
Performance requirements for the structural system and limitations on total movements can used to define the optimum effective period and level of damping. Unfortunately, the selection of hardware to supply these parameters is not simple.

Most isolation systems produce hysteretic damping. Both the effective period and damping are a function of displacement, as shown in Figure 9-1 for a lead rubber bearing.

Because of this displacement dependence, the design process is iterative, as shown schematically in Figure 9-2 for elastomeric bearing isolation systems (lead rubber and high damping rubber). A further complication arises for these types of bearing in that, as well as period and damping, the minimum plan size of the bearing is also a function of displacement.

The iterative process involves:

FIGURE 9-2 : ITERATIVE PROCEDURE FOR DESIGN



1. At each isolator locations, select a bearing plan size based on vertical load and assume a displacement at the target period and damping.

2. Calculate the effective stiffness, period and equivalent viscous damping at the assumed displacement.

3. From the seismic load parameters, calculate the actual displacement for this stiffness and damping.

4. Calculate revised damping for the actual displacement. Repeat step 3 if

necessary.

5. Check and adjust the minimum plan size required to support vertical loads at this displacement if necessary.

These steps are repeated until convergence is attained. The procedure can be automated using spreadsheet macros but there is no guarantee that convergence will be achieved as there are limits on effective periods and damping using practical isolators. Generally, the higher the vertical load on an elastomeric bearing the easier it will be to achieve long effective periods.

Because of the complexity of hardware design, and empirical aspects of design for most types of isolators, it is usual to obtain assistance from manufacturers. As base isolation technology has evolved, manufacturers have realized that structural engineers do not have the skills to design hardware and so will provide this assistance.

There are codes available (U.S, British and Australian) which provide design rules for devices used in isolation, such as elastomeric bearings and Teflon sliding bearings. However, most of these codes are for non-seismic bridge applications and need to be adapted to use for seismic isolation applications.

9.2 Implementation of the Design Procedure

Earlier versions of HCG spreadsheets attempted to perform a complete isolation system design using vertical load and seismic load data by automating the isolator plan size and height calculations. As most projects have constraints, it proved to be more efficient for the design engineer to retain control over these dimensions. The spreadsheets we use now are set up to evaluate system performance and factors of safety based on user selected isolator details. Although this procedure requires the engineer to have more knowledge of the base isolation design process, it allows the system to be closely tailored to project requirements.

The example provided in this section is based on design to 1997 UBC requirements, using the spreadsheet UBCTemplate.XLS. The spreadsheet templates based on other codes follow the same general principles. In the spreadsheets, cells colored red indicate user-specified input. The workbook contains a number of sheets with the design performed within the sheet *Design*.

9.2.1 Material Definition

The material definitions are contained on the sheet *Design Data*, as shown in Figure 9-3. This is the basic information used for the design process. The range of properties available for rubber is restricted and some properties are related to others, for example, the ultimate elongation, material constant and elastic modulus are all a function of the shear modulus. Information on available rubbers is provided later in this chapter and you should also check with manufacturers, especially for high damping rubber formulations.

As for the rubber, the PTFE properties used for sliding bearings are supplier-specific. The values listed in Figure 9-3 are typical of the material we have used on past projects but other properties are available.

High damping rubber is the most variable of the isolator materials as each manufacturer has specific properties for both stiffness and damping. The design procedure is based on tabulated values of the shear modulus and equivalent damping, as listed in Figure 9-3. The damping values tabulated may include viscous damping effects if appropriate.

Default HDR properties listed are for a relatively low damping rubber formulation and so any design based on these properties should be easily achievable from a number of manufacturers. As such, they should be conservative for preliminary design.

FIGURE 9-3 MATERIAL PROPERTIES USED FOR DESIGN

DESIGN PROPERTIES				
Units	1	0.001		
Elastomer Properties	**KN,mm**	**MPa**	**KN/cm^2**	**ksi**
Shear Modulus	0.0004	0.4	0.040	0.057
Ultimate Elongation	6.5	6.5	6.5	6.5
Material Constant, k	0.87	0.87	0.87	0.87
Elastic Modulus, E	0.00135	1.35	0.14	0.19
Bulk Modulus	1.5	1500	150	215
Damping	0.05	0.05	0.05	0.05
Lead Yield Strength	0.008	8.00	0.8	1.15
Teflon Coeff of Friction	0.1	0.10	0.1	0.1
Gravity	9810	9810	981	386.4
TFE Properties				
Vertical Stiffness	5000	5000000	500000	28222
Lateral Stiffness	2000	2000000	200000	11289
Coeff of Friction - Lo Vel	0.04	0.04	0.04	0.04
Coeff of Friction - Hi Vel	0.1	0.1	0.1	0.1
Coefficient a	0.9	0.9	0.9	0.9

TEST DATA HDR Bearings		
Shear Strain %	Shear Modulus MPa	Equivalent Damping %
10	1.21	12.72
25	0.79	11.28
50	0.57	10.00
75	0.48	8.96
100	0.43	8.48
125	0.40	8.56
150	0.38	8.88
175	0.37	9.36
200	0.35	9.36

9.2.2 PROJECT DEFINITION

The project definition section of the spreadsheet is as shown in Figure 9-4. The information provided defines the seismic loads and the structural data required in terms of the UBC requirements for evaluating performance:

1. The design units may be metric (KN,mm) or U.S units (kip,in). The example used here is in metric units.

2. The seismic information is extracted from UBC tables for the particular site. This requires the zone, soil type and fault information and the isolated lateral force coefficient, R_I.

3. The response modification coefficient, R, and importance factor, I, for an equivalent fixed base building are required as they form a limitation on base shear. Note that, for base isolated structures, the importance factor is assumed to be unity for all structures.

4. Building dimensions are required to estimate the torsional contribution to the total isolation system displacement.

The project definition information is specific to a project and, once set, does not need to be changed as different isolation systems are assessed and design progresses.

FIGURE 9-4 PROJECT DEFINITION

PROJECT:	UBC Design Example	
Units:	KN,mm	
Seismic Zone Factor, Z	0.4	Table 16-I
Soil Profile Type	SC	Table 16-J
Seismic Coefficient, C_A	0.400	Table 16-Q
Seismic Coefficient, C_V	**0.672**	Table 16-R
Near-Source Factor N_a	1.000	Table 16-S
Near-Source Factor N_v	1.200	Table 16-T
MCE Shaking Intensity $M_M Z N_a$	0.484	
MCE Shaking Intensity $M_M Z N_v$	0.581	
Seismic Source Type	A	Table 16-U
Distance to Known Source (km)	10.0	
MCE Response Coefficient, M_M	1.21	Table A-16-D
Lateral Force Coefficient, R_I	2.0	Table A-16-E
Fixed Base Lateral Force Coefficient, R	5.5	Table 16-N
Importance Factor, I	1.0	Table 16-K
Seismic Coefficient, C_{AM}	0.484	Table A-16-F
Seismic Coefficient, C_{VM}	**0.813**	Table A-16-G
Eccentricity, e	0.31	
Shortest Building Dimension, b	5.03	
Longest Building Dimension, d	6.23	
Dimension to Extreme Isolator, y	3.1	
$D_{TD}/D_D = D_{TM}/D_M$	1.182	

9.2.3 ISOLATOR TYPES AND LOAD DATA

The isolator types and load data are defined as shown in Figure 9-4. This stage assumes that you have decided on the type of isolator at each location. See Chapter 7 of these Guidelines for assistance on selecting the device types and the number of variations in type. For most projects, there will be some iteration as the performance of different types and layouts is assessed.

1. The types of isolators which can be included in this spreadsheet are lead rubber bearings (LRB), high damping rubber bearings (HDR), elastomeric bearings (ELAST, equivalent to an LRB with no lead core), flat sliding bearings (TFE) and curved sliding bearings (FPS).

2. For each isolator type, define the vertical load conditions. The average DL + SLL is used to assess seismic performance. The maximum and minimum load combinations are used to assess the isolator capacity.

3. The total wind load on the isolators may be provided as it may form a lower limit on the design shear forces.

4. Most building projects will not have a non-seismic displacement or rotation, these are more common on bridge projects. If they do apply, enter them on this sheet. Be aware that high rotations will severely limit the capacity of the elastomeric types of isolator (LRB, HDR and ELAST). If you have high rotations in some locations, consider using the pot-bearing type of slider there.

For most projects, the data in this section will be changed as you assess variation of isolation system. Often, the isolator type will be varied and sometimes variation of the number of each type of isolator will be considered.

FIGURE 9-5 ISOLATOR TYPES & LOAD DATA

BEARING TYPES AND LOAD DATA

	LRB-A	LRB-B	TFE-C	Total
Location				
Type (LRB, HDR, ELAST,TFE,FPS)	LRB	LRB	TFE	
Number of Bearings	12	12	6	30
Number of Prototypes	2	2	2	
Average DL + SLL	800	1200	500	
Maximum DL + LL	1100	1600	250	
Maximum DL + SLL + EQ	1600	2400	500	
Minimum DL - EQ	0	0	0	
Seismic Weight	9600	14400	3000	27000
Total Wind Load				620
Non-Seismic Displacement				
Non-Seismic Rotation (rad)				
Seismic Rotation (rad)				

9.2.4 Isolator Dimensions

The spreadsheet provides specific design for the elastomeric types of isolator (LRB, HDR and ELAST). For other types (TFE and FPS) the design procedure uses the properties specified on this sheet and the *Design Data* sheet but does not provide design details. For these types of bearings you will need to provide the load and design conditions to manufacturers for detailed design.

The isolators are defined by the plan size and rubber layer configuration (elastomeric based isolators) plus lead core size (lead rubber bearings) or radius of curvature (curved slider bearings). For curved slider bearings the radius defines the post-yielded slope of the isolation system and the period of response. You should be able to select an appropriate starting value of this from the project performance specifications and fine tune it by trial and error.

1. The minimum plan dimensions for the elastomeric isolators are those required for the maximum gravity loads. As described in the following section, the performance assessment includes the factor of safety under each load condition. The gravity factor of safety (F.S.), at zero displacement, should be at least 3 for both the strain and buckling limit states. This is based on the AASHTO requirement of providing a factor of 3 on the ultimate elongation at break. A starting point for the design procedure is to set a plan dimension such that this factor of safety is achieved.

2. As discussed earlier, the design process is iterative because the plan dimension is also a function of the maximum displacement. For moderate to high seismic zones the plan size based on a F.S. of 3 will likely need to be increased as the design progresses. An increment of 50 mm in plan sizes is generally used with sizes in the sequence of 570 mm, 620 mm etc. This is based on mold sizes in 50 mm increments plus 20 mm side cover rubber.

3. The rubber layer thickness is generally a constant at 10 mm. This thickness provides good confinement for the lead core and is sufficiently thin to provide a high load capacity. If vertical loads are critical the load thickness may be reduced to 8 mm or even 6 mm although you should check with manufacturers for these thin layers. Thinner layers add to the isolator height, and also cost, as more internal shims are required. The layer thickness should not usually exceed 10 mm for LRBs but thicker layers may be used for elastomeric or HDR bearings. The load capacity drops off rapidly as the layer thickness increases.

4. The number of layers defines the flexibility of the system. This needs to be set so that the isolated period is in the range required and so that the maximum shear strain is not excessive. This is set by trial and error.

5. The size of the lead core for LRBs defines the amount of damping in the system. The ratio of Q_D/W is displayed for guidance. This ratio usually ranges from 3% in low seismic zones to 10% or more in high seismic zones. Usually the softer the soil the higher the yield level for a given seismic zone. As for the number of rubber layers, the core is sized by trial and error.

6. Available plan shapes are Circular and Square (plus Rectangular for bridges). Most building projects use circular bearings as it is considered that these are more suitable for loading from all

horizontal directions. Square and rectangular bearings are more often used for bridges as these shapes may be more space efficient.

FIGURE 9-6 ISOLATOR DIMENSIONS

BEARING DIMENSIONS

	LRB-A	LRB-B	TFE-C
Plan Dimension (Radius for FPS)	670	770	
Depth (R only)			
Layer Thickness	10	10	
Number of Layers	20	20	
Lead Core Size	90	110	
Shape (S = Square, C = Circ)	C	C	
Total Height	301	301	
Weight (kg)	411	547	

Qd/W
6.75%

The procedure for fine tuning dimensions is to set initial values, activate the macro to solve for the isolation performance and change the configuration to achieve the improvements you need. At each step, the effect of the change is evaluated by assessing the isolation system performance, as described below.

9.2.5 ISOLATOR PERFORMANCE

The workbook contains a macro which solves for isolation performance once all dimensions and properties have been set. This is not automatic, you must activate the button once you have made changes. If your changes affect the performance of the isolation system you will see a message DBE (or MCE) NOT CONVERGED. Run the macro to get an updated performance summary.

As you make changes, you need to check two things, (1) the status of the isolation bearings to safely support the loads and (2) the performance of the isolation system.

The isolation bearing status for all elastomeric based isolators is summarized by the factors of safety, as shown in Figure 9-7. Although generally factors of safety exceeding 1.0 indicate satisfactory performance, experience has shown that some more severe restrictions should be imposed during the design process. This conservatism in design is recommended as it will increase the probability of successful prototype tests.

FIGURE 9-7 PERFORMANCE SUMMARY

PERFORMANCE SUMMARY

	LRB-A	LRB-B	TFE-C	DBE	MCE
Gravity Strain F.S.	12.26	12.91			
Buckling F.S	7.88	9.54			
DBE Strain F.S	2.10	2.28			
Buckling F.S	2.50	3.37			
MCE Strain F.S	1.38	1.62			
Buckling F.S	1.52	2.33			
Reduced Area / Gross Area	28.2%	36.6%			
Maximum Shear Strain	196%	196%			
Effective Period T_D T_M				2.08	2.17
Displacement D_D D_M				240.8	331.8
Total Displacements D_{TD} D_{TM}				284.5	392.1
Force Coefficient V_b / W				0.225	0.284
Force Coefficient V_s / W				0.112	
1.5 x Yield Force / W				0.101	
Wind Force / W				0.023	
Fixed Base V at T_D				0.070	
Design Base ShearCoefficient				0.112	
Damping β_{eff}				18.39%	14.70%
Damping Coefficients B_D B_M				1.44	1.32

- The gravity factor of safety should exceed 3.0 for both strain and buckling. For high seismic zones it will generally be at least 6.0 as performance is governed by seismic limit states.

- The DBE factor of safety should be at least 1.5 and preferably 2.0 for both strain and buckling.

- The MBE factor of safety should be at least 1.25 and preferably 1.5 for both strain and buckling.

- The ratio of reduced area to gross area should not go below 25% and should preferably be at least 30%.

- The maximum shear strain should not exceed 250% and preferably be less than 200%.

The limit states are governed by both the plan size and the number of rubber layers. You need to adjust both these parameters to achieve a design within the limitations above. At each change, you also need to check whether the seismic performance is achieved.

The performance of the isolated structure is summarized for the DBE and MCE in the final two columns in Figure 9-7. Performance indicators to assess are:

- The isolated period. Most isolation systems have an effective period in the range of 1.50 to 2.50 seconds for DBE, with the longer periods tending to be used for high seismic zones. It may not be possible to achieve a period near the upper limit if the isolators have light loads.

- The displacements and total displacements. The displacements are estimated values at the center of mass and the total displacements, which include an allowance for torsion. The latter values, at MCE loads, define the separation required around the building.

- The force coefficient V_b/W is the maximum base shear force that will be transmitted through the isolation system to the structure above. This is the base shear for elastic performance but is necessarily the design base shear.

- The design base shear coefficient is defined by UBC as the maximum of four cases:

  1. The elastic base shear reduced by the isolated response modification factor $V_S = V_B/R_I$.

  2. The yield force of the isolation system factored by 1.5.

  3. The base shear corresponding to the wind load.

  4. The coefficient required for a fixed base structure with a period equal to the isolated period.

For this example, the first condition governs. The designer should generally aim for this situation as the isolation system will be used most efficiently if this limit applies.

The performance summary also lists the equivalent viscous damping of the total isolation system and the associated damping reduction factor, B. Design should always aim for at least 10% damping at both levels of earthquake and preferably 15%.

The design worksheet also provides details of the calculations used to obtain this performance summary. Figure 9-8 shows the calculations for the MCE level. In this example, the TFE slider bearing provides much higher damping than the LRBs on an individual bearing basis. However, as only about 10% of the total seismic weight is supported on sliders the contribution to total system damping of the sliders is not large.

FIGURE 9-8 PERFORMANCE AT MCE LEVEL

Seismic Performance : Maximum Capable Earthquake

	LRB-A	LRB-B	TFE-C	MCE
Number of Isolators	12	12	6	30
Elastic Stiffness, Ku	5.56	7.49	0.00	
Yielded Stiffness, Kr*	0.63	0.84	0.00	
Yield Displacement, Dy	10.46	11.55	0.00	
Characteristic Strength, Qd	50.89	76.03	50.00	
Seismic Displacement, Dm				331.80
Bearing Force = Qd+DmKr*	259.4	354.9	50.000	
Effective Stiffness = F/Dm	0.782	1.070	0.151	23.124
Seismic Weight				27000
Seismic Mass = W/9810				2.752
Effective Period = $2\pi\sqrt{(M/K)}$				2.17
$Ah = 4Q_D(\Delta_m-\Delta_y)$	65415	97388	66359	2351797
$\beta = (1/2\pi)(A_h/K_e\Delta^2)$	12.10%	13.16%	63.66%	14.70%
B Factor				1.32
SA = Cvd/BT				0.28
SD = $(g/4\pi^2)$*CvdTd/Bd				331.81
Check Convergence = Sd/Δm				1.00

There is quite an art to the selection of final isolation design parameters. For example, in this case damping could be increased by increasing lead core core sizes in the LRBs. The core sizes cannot be increased much, however, as the yield force will increase. As shown in Figure 9-7, the design base shear force will be governed by the $1.5F_Y$ condition if core sizes are increased. Therefore, the extra damping may actually result in an increase in structural design forces.

9.2.6 PROPERTIES FOR ANALYSIS

The workbook provides a plot of the hysteresis curves for each of the isolator types as designed for displacements up to the MCE total displacement level. These plots (Figure 9-9) show the bi-linear properties to be used for system evaluation.

The properties used to develop the hysteresis loop are also listed in a format suitable for the ETABS program, as shown in Figure 9-10. The use of these properties is discussed later in these guidelines.

FIGURE 9-9 HYSTERESIS OF ISOLATORS

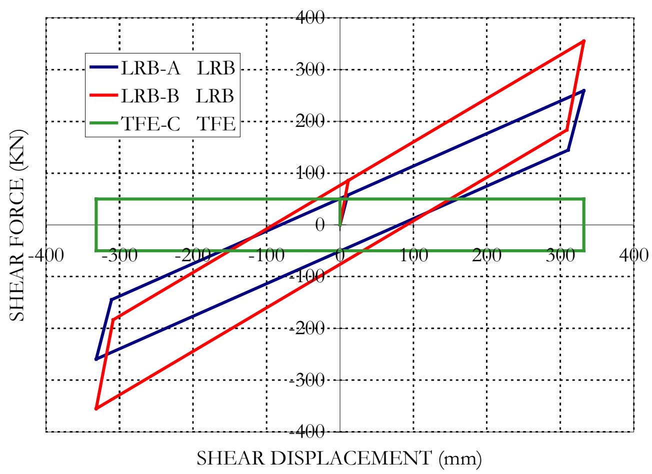


FIGURE 9-10 ANALYSIS PROPERTIES FOR ETABS

ETABS Spring Properties

	LRB-A	LRB-B	TFE-C	
	LRB	LRB	TFE	
First Data Line:				
ID	1	2	3	Identification Number
ITYPE	ISOLATOR1	ISOLATOR1	ISOLATOR2	Biaxial Hysteretic/Linear/Friction
KE2	0.78	1.07	0.15	Spring Effective Stiffness along Axis 2
KE3	0.78	1.07	0.15	Spring Effective Stiffness along Axis 3
DE2	0.071	0.081	0.587	Spring Effective Damping Ratio along Axis 2
DE3	0.071	0.081	0.587	Spring Effective Damping Ratio along Axis 3
Second Data Line:				
K1	709.4	1145.0	5000.0	Spring Stiffness along Axis 1 (Axial)
K2	5.49	7.41	2000.00	Initial Spring Stiffness along Axis 2
K3	5.49	7.41	2000.00	Initial Spring Stiffness along Axis 3
FY2/K11/CFF2	57.47	85.59	0.10	Yield Force Along Axis 2
FY3/K22/CFF3	57.47	85.59	0.10	Yield Force Along Axis 3
RK2/K33/CFS2	0.11	0.11	0.04	Post-Yield stiffness ratio along Axis 2
RK3/CFS3	0.11	0.11	0.04	Post-Yield stiffness ratio along Axis 3
A2			0.90	Coefficient controlling friction Axis 2
A3			0.90	Coefficient controlling friction Axis 3
R2			0.000	Radius of Contact 2 direction
R3			0.000	Radius of Contact 3 direction

9.3 DESIGN EQUATIONS FOR RUBBER AND LEAD RUBBER BEARINGS

9.3.1 CODES

The vertical load capacity of elastomeric isolation bearings has traditionally been based on a limiting strain formulation as implemented in the British codes BS 5400 and BE 1/76. These codes were intended for non-seismic applications where lateral forces are from sources such as traffic loads and thermal movements in bridges.

The U.S. AASHTO bridge code provides rules for vertical load capacity of elastomeric bearings subjected to earthquake induced displacements. This code adjusts the factors of safety from the British codes to be more appropriate for short duration, infrequently occurring loads.

9.3.2 EMPIRICAL DATA

For lead-rubber bearings some of the procedures are based on empirical data, in particular the effective yield stress of the lead core and the elastic (unloading) stiffness. The values reported here are those used by Skellerup Industries, based on an extensive database of test results for this type of bearing assembled from projects from 1978 to the present. The values used have been shown to give an accurate estimate of force levels and hysteresis loop areas. Other manufacturers will be able to provide similar data for their bearings.

9.3.3 Definitions

A_b	=	Bonded area of rubber
A_g	=	Gross area of bearing, including side cover
A_h	=	Area of hysteresis loop
		(Also termed EDC = energy dissipated per cycle)
A_{pl}	=	Area of Lead core
A_r	=	Reduced rubber area
B	=	Overall plan dimension of bearing
B_b	=	Bonded plan dimension of bearing
E	=	Elastic modulus of rubber
	=	3.3 to 4.0 G depending on hardness
E_b	=	Buckling Modulus
E_c	=	Effective Compressive Modulus
E_∞	=	Bulk Modulus
f	=	Factor applied to elongation for load capacity
	=	1 / (Factor of Safety)
F_m	=	Force in bearing at specified displacement
g	=	Acceleration due to gravity
G_γ	=	Shear modulus of rubber (at shear strain γ)
H_r	=	Height free to buckle
I	=	Moment of Inertia of Bearing
k	=	Material constant (0.65 to 0.85 depending on hardness)
K_d	=	Yielded stiffness of lead rubber bearing $= K_r$
K_{eff}	=	Effective Stiffness
K_r	=	Lateral stiffness after yield
K_u	=	Elastic Lateral stiffness
K_v	=	Vertical stiffness of bearing
K_{vi}	=	Vertical stiffness of layer i
n	=	Number of rubber layers
p	=	Bonded perimeter
P	=	Applied vertical load
P_{cr}	=	Buckling Load
P_γ	=	Maximum rated vertical load
Q_d	=	Characteristic strength
		(Force intercept at zero displacement)
S_i	=	Shape factor for layer i
t_i	=	Rubber layer thickness
t_{sc}	=	Thickness of side cover

t_{sh}	=	Thickness of internal shims
T_{pl}	=	Thickness of mounting plates
T_r	=	Total rubber thickness
W	=	Total seismic weight
Δ	=	Applied lateral displacement
Δ_m	=	Maximum applied displacement
Δ_y	=	Yield displacement of lead rubber bearing
β	=	Equivalent viscous damping
ε_u	=	Minimum elongation at break of rubber
ε_c	=	Compressive Strain
ε_{sc}	=	Shear strain from applied vertical loads
ε_{sh}	=	Shear strain from applied lateral displacement
ε_{sr}	=	Shear strain from applied rotation
ε_u	=	Minimum elongation at break of rubber
θ	=	Applied rotation
σ_y	=	Lead yield stress

9.3.4 Range of Rubber Properties

Rubber compounds used for isolation are generally in the hardness range of 37 to 60, with properties as listed in Table 9-1. As compounding is a continuous process intermediate values from those listed are available. As seismic demands have increased over the last 10 years the softer rubbers tend to be used more often. The lowest stiffness rubber has a shear modulus G of about 0.40 MPa although some manufacturers may be able to supply rubber with G as low as 0.30 MPa.

There is uncertainty about the appropriate value to use for the bulk modulus, K_∞, with quoted values ranging from 1000 to 2000 MPa. The 1999 AASHTO Guide Specifications provide a value of 1500 MPa and this is recommended for design.

Table 9-1 Vulcanized Natural Rubber Compounds

Hardness IRHD±2	Young's Modulus E (MPa)	Shear Modulus G (MPa)	Material Constant k	Elongation at Break Min, %
37	1.35	0.40	0.87	650
40	1.50	0.45	0.85	600
45	1.80	0.54	0.80	600
50	2.20	0.64	0.73	500
55	3.25	0.81	0.64	500
60	4.45	1.06	0.57	400

9.3.5 Vertical Stiffness And Load Capacity

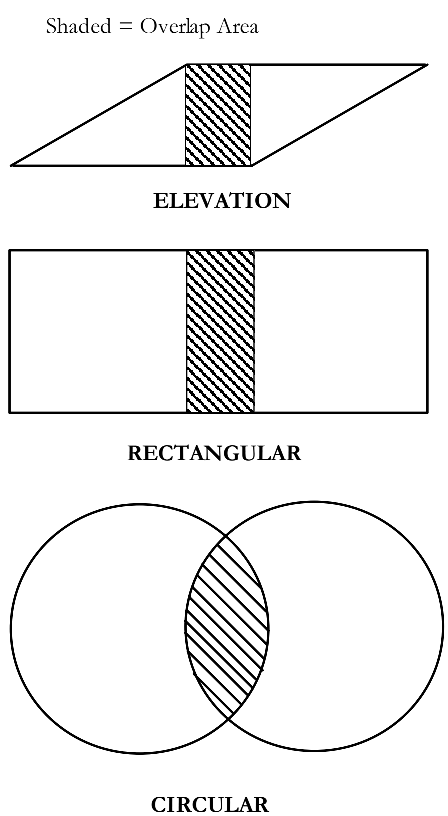
The dominant parameter influencing the vertical stiffness, and the vertical load capacity, of an elastomeric bearing is the shape factor. The shape factor of an internal layer, S_i, is defined as the loaded surface area divided by the total free to bulge area:

$$S_i = \frac{B}{4t_i} \qquad \text{for square and circular bearings}$$

for lead rubber bearings, which have a hole for the lead core,

$$S_i = \frac{A_b - A_{pl}}{\pi B_b t_i}$$

FIGURE 9-11 EFFECTIVE COMPRESSION AREA



9.3.6 Vertical Stiffness

The vertical stiffness of an internal layer is calculated as:

$$K_{vi} = \frac{E_c A_r}{t_i}$$

where the compressive modulus, E_c, is a function of the shape factor and material constant as follows:

$$E_c = E\left[1 + 2kS_i^2\right]$$

In the equation for vertical stiffness, a reduced area of rubber, A_r, is calculated based on the overlapping areas between the top and bottom of the bearing at a displacement, Δ, as follows (see Figure 9-11):

$$A_r = A_b\left(1 - \frac{\Delta}{B_b}\right) \qquad \text{for square bearings}$$

$$A_r = 0.5\left\{B^2 \sin^{-1}\left(\frac{\varsigma}{B_b}\right) - \Delta\varsigma\right\}$$

where for circular bearings

$$\varsigma = \sqrt{\left(B_b{}^2 - \Delta^2\right)}$$

When the effective compressive modulus, E_c, is large compared to the bulk modulus E_∞ then the vertical deformation due to the bulk modulus is included by dividing E_c by 1 + (E_c /E_∞) to calculate the vertical stiffness.

Bulk modulus effects are used to when the vertical stiffness is used to calculate vertical deformations in the bearing but not the shear strains due to vertical load. The 1999 AASHTO Guide Specifications are an exception to this – see below.

9.3.7 Compressive Rated Load Capacity

The vertical load capacity is calculated by summing the total shear strain in the elastomer from all sources. The total strain is then limited to the ultimate elongation at break of the elastomer divided by the factor of safety appropriate to the load condition.

The shear strain from vertical loads, ε_{sc} , is calculated as

$$\varepsilon_{sc} = 6 S_i \varepsilon_c$$

where

$$\varepsilon_c = \frac{P}{K_{vi} t_i}$$

If the bearing is subjected to applied rotations the shear strain due to this is

$$\varepsilon_{sr} = \frac{B_b{}^2 \theta}{2 t_i T_r}$$

The shear strain due to lateral loads is

$$\varepsilon_{sh} = \frac{\Delta}{T_r}$$

For service loads such as dead and live load the limiting strain criteria are based on AASHTO 14.5.1P

$$f\varepsilon_u \geq \varepsilon_{sc} \qquad \text{where} \quad f = 1/3 \quad \text{(Factor of safety 3)}$$

And for ultimate loads which include earthquake displacements

$$f\varepsilon_u \geq \varepsilon_{sc} + \varepsilon_{sh} \quad \text{where } f = 0.75 \text{ (Factor of safety 1.33)}$$

Combining these equations, the maximum vertical load, P_γ, at displacement Δ can be calculated from:

$$P_\gamma = \frac{K_{vi}\, t_i \left(f\varepsilon_u - \varepsilon_{sh}\right)}{6S_i}$$

Codes used for buildings and other non-bridge structures (e.g. UBC) do not provide specific requirements for calculating elastomeric bearing load capacity. Generally, the total strain formulation from AASHTO is used with the exception that the Maximum Capable Earthquake displacement is designed using f = 1.0.

9.3.7.1 AASHTO 1999 Requirements

The 1999 AASHTO Guide Specifications generally follow these same formulations but make two adjustments:

1. The total strain is a constant value for each load combination, rather than a function of ultimate elongation. Using the notation of this section, AASHTO defines a strain due to non-seismic deformations, $\varepsilon_{s,s}$ and a strain due to seismic displacements, $\varepsilon_{s,eq}$. The limits are then:

$$\varepsilon_{sc} \leq 2.5$$

$$\varepsilon_{sc} + \varepsilon_{s,s} + \varepsilon_{sr} \leq 5.0$$

$$\varepsilon_{sc} + \varepsilon_{s,eq} + 0.5\varepsilon_{sr} \leq 5.5$$

2. The shear strain due to compression, ε_{sc}, is a function of the maximum shape factor:

$$\varepsilon_{sc} = \frac{3SP}{2A_r G(1+2kS^2)}$$

For S ≤ 15, or

$$\varepsilon_{sc} = \frac{3P(1+8GkS^2/E_\infty)}{4GkSA_r}$$

for S > 15.

The equation for S ≤ 15 is a re-arranged form of the equations above with the approximation that E = 4G. The formula for S > 15 has approximated $(1+2kS^2) \approx 2kS^2$ and adjusted the vertical stiffness for the bulk modulus effects.

There is no universal agreement regarding the inclusion of bulk modulus effects in load capacity calculations and at this stage it is recommended that the 1999 AASHTO formulas be used only if the specifications specifically require this (see Section 4.3 for discussion).

9.3.8 Tensile Rated Load Capacity

For bearings under tension loads, the stiffness in tension depends upon the shape of the unit, as in compression, and is approximately the same as the compression stiffness. Therefore, the same equations are used as for compressive loads except that the strains are the sum of absolute values.

When rubber is subjected to a hydrostatic tension of the order of 3G, cavitation may occur. This will drastically reduce the stiffness. Although rubbers with very poor tear strength may rupture catastrophically once cavitation occurs, immediate failure does not generally take place. However, the subsequent strength of the component and its stiffness may be effected. Therefore, the isolator design should ensure that tensile stresses do not exceed 3G under any load conditions.

9.3.9 Bucking Load Capacity

For bearings with a high rubber thickness relative to the plan dimension the elastic buckling load may become critical. The buckling load is calculated using the Haringx formula as follows:

The moment of inertia, I is calculated as

$$I = \frac{B_b^{\ 4}}{12} \qquad \text{for square bearings}$$

$$I = \frac{\pi B_b^{\ 4}}{64} \qquad \text{for circular bearings}$$

The height of the bearing free to buckle, that is the distance between mounting plates, is

$$H_r = (nt_i) + (n-1)t_{sh}$$

An effective buckling modulus of elasticity is defined as a function of the elastic modulus and the shape factor of the inner layers:

$$E_b = E(1 + 0.742 S_i^2)$$

Constants T, R and Q are calculated as:

$$T = E_b I \frac{H_r}{T_r}$$

$$R = K_r H_r$$

$$Q = \frac{\pi}{H_r}$$

From which the buckling load at zero displacement is:

$$P_{cr}^0 = \frac{R}{2}\left[\sqrt{1 + \frac{4TQ^2}{R}} - 1\right]$$

For an applied shear displacement the critical buckling load at zero displacement is reduced according to the effective "footprint" of the bearing in a similar fashion to the strain limited load:

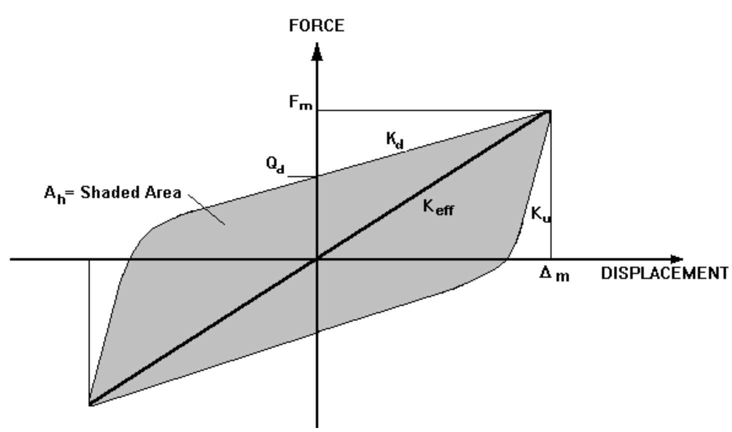
$$P_{cr}^{\gamma} = P_{cr}^0 \frac{A_r}{A_g}$$

The allowable vertical load on the bearing is the smaller of the rated load, P, or the buckling load.

9.3.10 Lateral Stiffness and Hysteresis Parameters For Bearing

Lead rubber bearings, and elastomeric bearings constructed of high damping rubber, have a nonlinear force deflection relationship. This relationship, termed the hysteresis loop, defines the effective stiffness (average stiffness at a specified displacement) and the hysteretic damping provided by the system. A typical hysteresis for a lead-rubber bearing is as shown in Figure 9-12.

Figure 9-12 : Lead Rubber Bearing Hysteresis



For design and analysis this shape is usually represented as a bilinear curve with an elastic (or unloading) stiffness of K_u and a yielded (or post-elastic) stiffness of K_d. The post-elastic stiffness K_d is equal to the stiffness of the elastomeric bearing alone, K_r. The force intercept at zero displacement is termed Q_d, the characteristic strength, where:

$$\mathrm{Q_d} = \sigma_y \mathrm{A_{pl}}$$

The theoretical yield level of lead, σ_y, is 10.5 MPa (1.5 ksi) but the apparent yield level is generally assumed to be 7 MPa to 8.5 MPa (1.0 to 1.22 ksi), depending on the vertical load and lead core confinement.

The post-elastic stiffness, K_d, is equal to the shear stiffness of the elastomeric bearing alone:

$$\mathrm{K_r} = \frac{\mathrm{G}_\gamma \mathrm{A_r}}{\mathrm{T_r}}$$

The shear modulus, G_γ, for a high damping rubber bearing is a function of the shear strain γ, but is assumed independent of strain for a lead-rubber bearing manufactured from natural rubber and with standard cure.

The elastic (or unloading) stiffness is defined as:

$$\mathrm{K_u} = \mathrm{K_r} \qquad \text{for elastomeric bearings}$$

$$\mathrm{K_u} = 6.5\mathrm{K_r}\left(1 + \frac{12\mathrm{A_{pl}}}{\mathrm{A_r}}\right)$$

for lead-rubber bearings

$$\mathrm{K_u} = 25\mathrm{K_r}$$

For lead rubber bearings, the first formula for K_u was developed empirically in the 1980's to provide approximately the correct stiffness for the initial portion of the unloading cycle and to provide a calculated hysteresis loop area which corresponded to the measured areas.

The bearings used to develop the original equations generally used 12.7 mm (½") rubber layers and doweled connections. By the standard of bearings now used, they were poorly confined. Test results from more recent projects has shown that the latter formula for K_u provides a more realistic estimate for most LRBs.

The shear force in the bearing at a specified displacement is:

$$\mathrm{F_m} = \mathrm{Q_d} + K_r \Delta$$

from which an average, or effective, stiffness can be calculated as:

$$K_{eff} = \frac{F_m}{\Delta}$$

The sum of the effective stiffness of all bearings allows the period of response to be calculated as:

$$T_e = 2\pi\sqrt{\frac{W}{g\Sigma K_{eff}}}$$

Seismic response is a function of period and damping. High damping and lead rubber bearings provide hysteretic damping. For high damping rubber bearings, the hysteresis loop area is measured from tests for strain levels, γ, and the equivalent viscous damping β calculated as given below. For lead rubber bearings the hysteresis area is calculated at displacement level Δ_m as:

$$A_h = 4Q_d\left(\Delta_m - \Delta_y\right)$$

from which the equivalent viscous damping is calculated as:

$$\beta = \frac{1}{2\pi}\left(\frac{A_h}{K_{eff}\Delta^2}\right)$$

The isolator displacement can be calculated from the effective period, equivalent viscous damping and spectral acceleration as:

$$\Delta_m = \frac{S_a T_e^{\,2}}{4\pi^2 B}$$

where S_a is the spectral acceleration at the effective period T_e and B is the damping factor, a function of β which is obtained from AASHTO or the UBC.

The formula for Δ_m includes T_e and B, both of which are themselves a function of Δ_m. Therefore, the solution for maximum displacement includes an iterative procedure.

9.3.11 Lead Core Confinement

The effect of inserting a lead core into an elastomeric bearing is to add an elastic-perfectly plastic component to the hysteresis loop as measured for the elastomeric bearings. The lead core will have an apparent yield level which is a function of the theoretical yield level of lead, 10.5 MPa, (1.58 ksi) and the degree of confinement of the lead. As the confinement of the lead increases the hysteresis of the lead core will move more towards an elasto-plastic system as shown in Figure 9-13.

Confinement is provided to the lead core by three mechanisms:

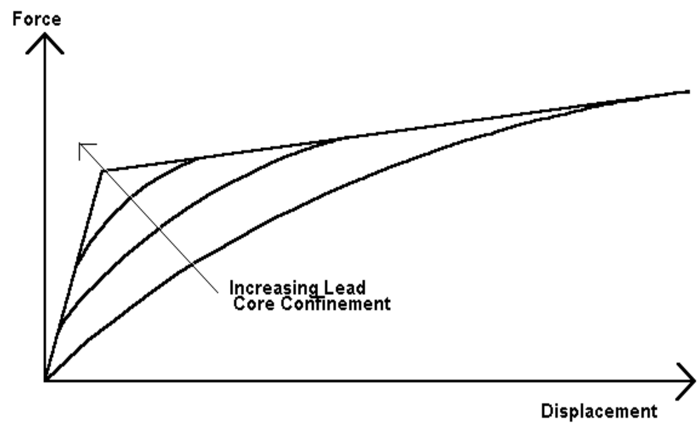
1 The internal shims restraining the lead from bulging into the rubber layers.,

2 Confining plates at the top and bottom of the lead core.

3 Vertical compressive loads on the bearings.

The confinement provided by internal shims is increased by decreasing the layer thickness, which increases the number of shims providing confinement. Earlier lead rubber bearings used doweled and then bolted top and bottom mounting plates. Current practice is to use bonded mounting plates, which provides more effective confinement than either of the two earlier methods.

FIGURE 9-13: EFFECT OF LEAD CONFINEMENT



The degree of confinement required also increases as the size of the lead core increases. Smaller diameter cores, approximately B/6, tend to have a higher apparent yield level than cores near the maximum diameter of B/3.

The effective stiffness and loop area both reduce with the number of cycles. The effective stiffness is essentially independent of axial load level but the loop area varies proportional to vertical load. During an earthquake some bearings will have decreased loop area when earthquake induced loads act upwards. However, at the same time instant other bearings will have increased compressive loads due to earthquake effects and so an increased loop area. The net effect will be little change in total hysteresis area based on an average dead load.

Lead core confinement is a complex mechanism as the lead is flowing plastically during seismic deformations and the elastomer must be considered to be a compressible solid. These features preclude explicit calculations of confinement forces. Manufacturers generally rely on their databases of prototype and test results plus manufacturing experience to ensure that the isolators have adequate confinement for each particular application. This is then demonstrated by prototype tests.

For long term loads, lead will creep and the maximum force in the core will be less than the yield force under suddenly applied loads. For structures such as bridges where non-seismic displacements are applied to the bearings this property will affect the maximum force transmitted due to creep, shrinkage and temperature effects.

Tests at slow loading rates have shown that for loads applied over hours rather than seconds the stress relaxation in the lead is such that the maximum force in the lead will be about one-quarter the force for rapid loading rates. Therefore, for slowly applied loads the maximum lead core force is assumed to be $F = 0.25\ Q_d$.

9.3.12 Design Procedure

As noted above, the solution of seismic performance requires an iterative procedure. The design is performed using the Excel spreadsheet described earlier in this chapter. Initial bearing plan sizes are determined, based on maintaining a factor of safety of 3 under maximum vertical loads in the undeformed configuration.

The number of rubber layers and the lead core sizes are then set by a trial-and-error procedure to achieve the required seismic performance. As the damping is a function of displacement, this requires an iterative procedure that is implemented as a macro in the design workbook:

1. A displacement is assumed, using the total rubber thickness as a starting point.

2. The effective stiffness of the bearing at this displacement is calculated.

3. The effective period is calculated using the total seismic mass and the effective period.

4. The equivalent viscous damping is calculated from the area of the hysteresis loop. For HDR, the damping and shear modulus are interpolated from tabulated values of these quantities versus shear strain.

5. The damping factor, B, is calculated for the equivalent viscous damping.

6. The spectral displacement is calculated from the acceleration response spectrum at the effective period, modified by the damping factor B.

7. This displacement is compared with the displacement assumed in Step 1. above. If the difference exceeds a preset tolerance, the calculated displacement defines a new starting displacement and the procedure is repeated until convergence is achieved.

The seismic performance is evaluated for both the design level and the maximum seismic events.

9.4 Sliding and Pendulum Systems

Both flat and curved sliding systems can be designed using the same procedure as outlined above but is generally simpler in that the device properties are not a function of dimensions. The isolation system properties are defined by two parameters:

1. The characteristic strength, defined as μW where μ is the coefficient of friction for the sliding surface and W is the total seismic weight.

2. The post-yielded stiffness is defined as zero for a flat slider or W/R for a spherical slider.

The steps above are then used to iterate to solve for the isolated displacement.

9.5 OTHER SYSTEMS

The design procedure can be used for any type of isolation system which can be approximated with a softening bi-linear hysteresis loop, that is, the yielded stiffness is less than the elastic stiffness, or for which tabulated values of damping and stiffness versus displacement are available.

For devices that do not have these characteristics, special design procedures may need to be developed.

10 EVALUATING PERFORMANCE

10.1 STRUCTURAL ANALYSIS

The design procedure described in the previous chapter is based on the response of the isolation system alone, without accounting for the flexibility of the structure itself. The flexibility of the structure above the bearings (or the substructure below bridge bearings) will modify the response because some of the displacement will take place in the structure. The amount of variation from the assumed response will depend on the flexibility of the structure or substructure relative to the isolation system.

It is possible to modify the design procedure to take account of substructure flexibility for bridge structures. In this modification, the stiffness of the bearings is calculated as the combined stiffness of the isolators and the bent acting in series. However, most design offices have computers with structural analysis programs and it is generally more efficient to include structural flexibility at the analysis phase rather than as part of the design process.

TABLE 10-1 ANALYSIS OF ISOLATED STRUCTURES

1. SINGLE DEGREE OF FREEDOM NONLINEAR Isolation System Design
2. PLANE FRAME / PLANE WALL 2D BRIDGE MODELS NONLINEAR Design Level vs. Damage Studies Effect of Bent Flexibility
3. THREE DIMENSIONAL LINEAR Member Design Forces
4. THREE DIMENSIONAL LINEAR SUPERSTRUCTURE NONLINEAR ISOLATORS Isolation System Properties
5. THREE DIMENSIONAL NONLINEAR SUPERSTRUCTURE NONLINEAR ISOLATORS Isolation System and Structure Performance

There is a hierarchy of analysis procedures, as listed in Table 10-1 in order of complexity. Each procedure has its role in the design and evaluation process. The analysis options are not mutually exclusive and in fact all methods are usually performed in sequence up to the most complex procedure appropriate for a project. In this way, each procedure provides benchmark results to assess the reasonableness of the results from the next, more complex procedure.

10.2 Single degree-of-freedom Model

A simple model is often used at the preliminary design phase, especially for hybrid systems. The superstructure is assumed rigid and the total weight modeled as a single mass. A number of elements in parallel are then used to model the isolators. For example, all elastomeric bearings are represented as a single elastic element, the lead cores as an elasto-plastic element and the sliding bearings as a single friction element. Viscous damping would be included in the elastic element but not the yielding elements. This model provides maximum isolator displacements and forces and acts as a check on the design procedure used.

This model will produce results equivalent to those produced by the design procedure but with more accurate results in two areas:

1. The input for this model is a series of time histories and so this procedure the differences in results that will be produced by time history analysis compared to the response spectrum analysis method used in the design procedure.

2. The mass can be excited by two components of earthquake simultaneously, which will provide displacements and base shear forces that incorporate the interaction of isolator yield in the two directions.

The ANSRL program is most commonly used for this type of analysis although it is possible to also use 3D-BASIS or ETABS.

10.3 Two Dimensional Nonlinear Model

Two-dimensional models of a single representative frame or shear wall from the building are an effective way of assessing the effects of superstructure / substructure response and yielding. The structural elements are represented as bilinear yielding elements and the isolators as for the single degree of freedom model.

For bridge structures, two separate 2D models are usually developed, one to model longitudinal response and the other to model transverse response.

The DRAIN2D2 program is used for these analyses. This type of analysis is rarely used for buildings as computer hardware is such that three-dimensional non-linear analyses are practical in almost all cases. It is still used for bridges, and in fact is implemented in the BRIDGE spreadsheet, as the final analyses for bridges can be performed separately for the longitudinal and transverse directions.

10.4 Three dimensional Equivalent Linear Model

A linear elastic model using a building analysis program such as ETABS (or STRUDL for bridges) is sufficient for final design for some structures. A response spectrum analysis is performed to obtain earthquake response. In this type of analysis the isolators are represented as short column or bearing elements with properties selected to provide the effective stiffness of the isolators. Damping is incorporated by reducing the response spectrum in the range of isolated periods by the B Factor.

As described earlier, there are some doubts about a possible under-estimation of the overturning moments for most non-linear isolation systems if this procedure is used. Pending resolution of this issue, it is recommended that a time history analysis always be performed on this model. Note that in ETABS and other programs the same model can be used for both response spectrum and time history analyses.

10.5 Three Dimensional Model - Elastic Superstructure, Yielding Isolators

This type of model is appropriate where little yielding is expected above or below the isolators. Some programs, for example 3D-BASIS, represent the building as a "super-element" where the full linear elastic model of the fixed base structure is used to reduce the superstructure to an element with three degrees of freedom per floor. This type of analysis provides isolator displacements directly and load vectors of superstructure forces. The critical load vectors are applied back to the linear elastic model to obtain design forces for the superstructure.

As the isolators are modeled as yielding elements the response spectrum method cannot be used and so a time history analysis must be performed. ETABS has non-linear isolator elements and it is recommended that this option be used for all structures.

10.6 Fully Nonlinear Three Dimensional Model

Full non-linear structural models have become more practical as computer hardware in design offices has improved although they remain time consuming and are generally only practical for special structures. The Museum of New Zealand nonlinear model with 2250 degrees of freedom and over 1500 yielding elements was analyzed on MS-DOS 486 computers and provided full details of isolator forces and deformations, structural plastic rotations, drifts, floor accelerations and in-structure response spectra.

Our offices use the ANSR-L program for this type of analysis. See the separate Users Manual for Performance Based Design for a full description of non-linear analysis.

10.7 Device Modeling

For nonlinear analysis the yield function of the devices is modeled explicitly. The form of this function for a particular element depends on the device modeled:

- HDR bearings are modeled as either a linear elastic model with viscous damping included or with the hysteretic loop directly specified.

- LRBs are modeled as either two separate components (rubber elastic, lead core elasto-plastic) or as a single bi-linear element.

- For sliding bearings, an elastic-perfectly plastic element with a high initial stiffness and a yield level which is a function of vertical pressure and velocity. If uplift can occur this is combined with a gap element so that the shear force is zero when uplift occurs.

The modeling must be such that damping is not included twice, as viscous and hysteretic. This is why LRBs are often better modeled as two components. Element damping is applied to the rubber component, which has some associated viscous damping, but not to the lead component.

For the Museum of New Zealand, a series of Teflon material tests were used to develop the dependence on pressure and velocity of the coefficient of friction. This was used to calibrate an ANSRL model. The model was then used to correlate the results of shaking table tests for a concrete block mounted on Teflon pads.

10.8 ETABS Analysis For Buildings

Versions 6 and above of the ETABS program have the capability of modeling a base isolated building supported on a variety of devices. The ETABS manual provides some guidance for developing an isolated model.

The Holmes Consulting Group isolation design spreadsheet produces a sheet giving properties to use for the ETABS analysis (see Figure 9-10). This section describes the basis for the calculation of these properties and procedures to analyze the isolated building using ETABS. These notes were developed at a time when Version 6 was used in all offices. As we progressively migrate all offices to Version 7, these notes will be updated. However, the same general concepts will apply.

10.8.1 Isolation System Properties

The devices used in HCG isolation system designs will generally be one or more of lead-rubber bearings (LRB), elastomeric bearings (ELAST), high damping rubber bearings (HDR) or Teflon flat or curved sliding bearings (PTFE and FPS). These are modeled as springs in ETABS. The appropriate spring types are as follows:

Lead Rubber Bearings (LRB)

LRBs are modeled as an equivalent bi-linear hysteresis loop with properties calculated from the lead yield stress and the elastomeric bearing stiffness. This is modeled as type ISOLATOR1 in ETABS.

Elastomeric Bearings (ELAST)

Plain elastomeric bearings are modeled as type LINEAR.

Sliding Bearings (PTFE and FPS)

Sliding bearings are modeled as type ISOLATOR2 with properties for the coefficient of friction at slow and fast velocities as developed from tests.

The coefficient of friction is a function of pressure on the bearing as well as velocity. ETABS incorporates the velocity dependence but not the pressure dependence. Recommended values for Teflon are:

Vertical Pressure on PTFE	Friction Coefficient at Low Velocity	Friction Coefficient at High Velocity	Coefficient Controlling Variation
< 5 MPa	0.04	0.14	0.9
5 - 15 MPa	0.03	0.12	0.9
> 15 MPa	0.03	0.10	0.9

For curved slider bearings (FPS) the radius of curvature is also specified. If you are using this type of isolator, consult the supplier for appropriate friction values.

High Damping Rubber Bearings (HDR)

Although the elastomer used for these bearings is termed "high damping" the major energy dissipation mechanism of the elastomer is hysteretic rather than viscous, that is, the force deflection curves form a nonlinear hysteresis. The UBC code provides a procedure for converting the area under the hysteresis loop to an equivalent viscous damping ratio to be used for equivalent linear analysis.

For nonlinear analysis it is more accurate to model the force deflection curve directly and so include the hysteretic damping implicitly. This avoids the approximations in converting a hysteresis area to viscous damping.

The second data line for the ETABS input file contains the bilinear properties for a nonlinear time history analysis. The procedure for deriving these properties is based on the following methodology:

- The effective stiffness at the design displacement is known from the design procedure and the stiffness properties of the elastomer. This provides the force in the bearing at the design displacement.
- An equivalent "yield" strain is defined in the elastomer. This defines a "yield" displacement.
- From the elastomer shear modulus at the assumed "yield" strain the yield force can be calculated and the hysteresis loop constructed.
- The area of this hysteresis loop is computed and, from this and the effective stiffness, the equivalent viscous damping is calculated.

- If necessary, the assumed "yield" strain is adjusted until the equivalent viscous damping at maximum displacement equals the damping provided by the elastomer at that strain level.

The spreadsheet provides details of these calculations.

10.8.2 Procedures for Analysis

The ETABS model can be analyzed using a number of procedures, in increasing order of complexity:

1. Equivalent static loads.
2. Linear response spectrum analysis.
3. Linear Time History Analysis.
4. Nonlinear Time History Analysis.

The UBC provides requirements on the minimum level of analysis required depending on building type and seismicity:

The *equivalent static analysis* is limited to small, regular buildings and would almost never be sufficient for New Zealand applications.

A *linear response spectrum analysis* is the most common type of analysis used. This is sufficient for almost all isolation systems based on LRB and/or HDR bearings.

The response spectrum analysis procedure uses the effective stiffness of the bearings, defined as the force in the bearing divided by the displacement. Therefore, it is iterative in that, if the analysis produces a displacement which varies from that assumed to calculate stiffness properties, the effective stiffness must be adjusted and the analysis repeated.

In practice, the single mass approximation used for system design usually gives a good estimate of displacement and multiple analyses are not required. However, if the ETABS analyses produce center of mass displacements above the isolators that are significantly different from the design procedure values (variation more than about ±10%) then the properties should be recalculated.

The effective stiffness at a specified displacement, Δ, can be calculated from the data on the second line of the ISOLATOR1 input as:

$$KE2 = \frac{FY2(1-RK2)}{\Delta} + K2.RK2$$

The effective damping can be calculated as:

$$DE2 = \frac{2.FY2.(1-RK2)(\Delta - \frac{FY2}{K2})}{\pi.KE2.\Delta^2}$$

This is the total damping - as discussed below, this must be reduced by the structural damping, typically 0.05, specified in ETABS.

Linear time history analysis provides little more information than the response spectrum analysis for a much greater degree of effort and so is rarely used.

Nonlinear time history analysis is required for (1) systems on very soft soil (2) systems without a restoring force (e.g. sliding systems) (3) velocity dependent systems and (4) systems with limited displacement capability.

In practice, nonlinear time history has been used in many projects even where not explicitly required by the UBC. This is largely because most isolated projects have been especially valuable or complex buildings. As discussed earlier, there are some concerns about the accuracy of equivalent stiffness analysis results. Time history analysis should always be used.

10.8.3 Input Response Spectra

The response spectrum analysis calculates the response of each mode from the spectral ordinate at that period. For the isolated modes the damping must include the equivalent viscous damping of LRB and HDR bearings. Therefore, a series of response spectra must be input covering the full range of damping values for all modes. These spectra are calculated by dividing the spectral values by the B factor for each damping factor, as specified in the UBC:

	Equivalent Viscous Damping						
	<2%	5%	10%	20%	30%	40%	>50%
B	0.8	1.0	1.2	1.5	1.7	1.9	2.0

The time history solution applies the modal damping to the response calculated for each mode during the explicit integration. Therefore, the input time history does not need to be modified to reflect damping.

10.8.4 Damping

The ETABS program is relatively straightforward for modeling stiffness properties of the isolators, both for effective stiffness analysis and nonlinear time history analysis. However, the manner in which damping is applied is more complex.

The aim for most analyses is to use 5% damping for the structural modes, as is assumed in the codes, but to use only the damping provided by the isolation system in the isolated modes. The procedure used to implement this in each type of analysis is as described below.

Response Spectrum Analysis

For the response spectrum analysis, a value of 0.05 is specified for DAMP in the *Lateral Dynamic Spectrum Data* section. This applies damping of 5% to all modes, including the isolated modes. To avoid including this damping twice, the value of both DE2 and DE3 in the *Spring Properties* section is reduced by 0.05. The spreadsheet calculations include this reduction.

Linear Time History Analysis

The linear time history analysis uses the effective stiffness and damping values as for the response spectrum analysis and so the same procedure for specifying damping as used above is applicable, that is, reducing DE2 and DE3 by 0.05.

An alternative method for specifying damping is available in time history analysis by providing data lines to override the modal damping value specified. In this procedure, NDAMP is specified as 3 in the *Lateral Dynamic Time History Data* section and modes 1 to 3 are specified to have 0.0 damping.

Nonlinear Time History Analysis

For nonlinear time history analysis the hysteretic damping is modeled explicitly and the values of DE2 and DE3 are not used. The only procedure available to avoid "doubling up" on damping is to specify viscous damping as 0.0 in the first three modes, the second method listed above for linear time history analysis. This slightly underestimates total damping.

The procedures used to specify damping for the different analysis types are generally based on an assumption of hysteretic damping only in the isolators, with no viscous damping. This is a conservative approach. From tests on these bearings at different frequencies, the damping may be increased by about 20% by viscous effects. In some types of analysis, this increase can be incorporated by increasing the size of the hysteresis loop.

10.9 Concurrency Effects

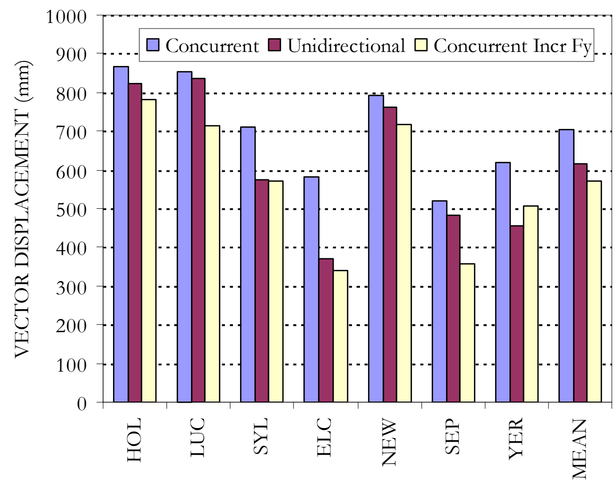
The design procedure for isolation systems is based on a single degree of freedom approximation which assumes a constant direction of earthquake loads. The evaluation of the structural system requires that earthquake motions be applied concurrently along both horizontal axes. For the response spectrum method of analysis UBC requires that the spectrum be applied 100% along one direction and 30% of the ground motion along the orthogonal axis. The time history method of analysis requires that two horizontal components of each earthquake record be applied simultaneously.

The yield function for bi-linear systems such as lead-rubber bearings is based on a circular interaction formula:

$$\frac{\sqrt{V_{ax}^2 + V_{ay}^2}}{V_y} = 1.0$$

where V_{ax} and V_{ay} are the applied shears along the two horizontal axes and V_y is the yield strength of the isolation system. If concurrent shear forces are being applied along each axis then the effective yield level along either axis will be less than the design value based on non-concurrent seismic loads. For the case where equal shear forces are applied in both directions simultaneously, the shear force along each axis will be equal to $V_y/\sqrt{2}$.

FIGURE 10-1 DISPLACEMENTS WITH CONCURRENT LOADS



The reduced yield force along a particular direction will result in the equivalent viscous damping being less than the value calculated from the design procedure. This will produce a performance different from that calculated. In some circumstances, it may be desirable to increase the yield level so that under concurrent action the response will closer match that calculated from non-concurrency.

As an example of the effects of concurrency, an isolated building was analyzed for seven near fault earthquake records, each with two horizontal components scaled by the same factor.

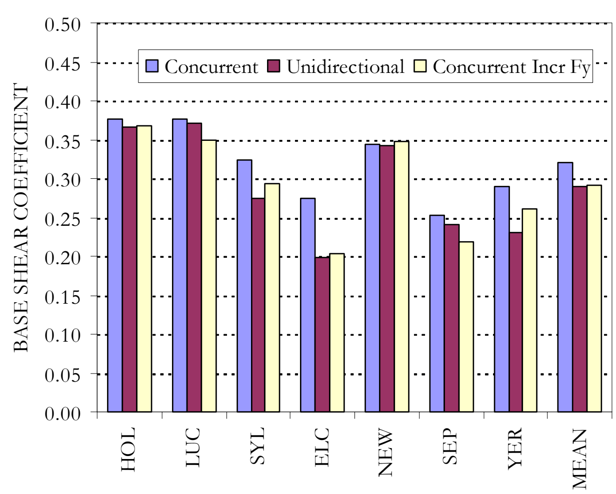
Maximum displacements and base shear coefficients were obtained for three cases:

1. The isolation system as designed, maximum vector response when both components were applied simultaneously.

2. The isolation system as designed, maximum vector response of the components applied individually.

3. The isolation system with the yield level increased by $\sqrt{2}$, then evaluated as for 1. above, the maximum vector response with both components applied

FIGURE 10-2 SHEARS WITH CONCURRENT LOADS



simultaneously.

Figures 10-1 and 10-2 plot the resulting displacements and shear coefficients respectively for the three cases for each earthquake. Also plotted are the mean results, which would be used for design if 7 earthquakes were used for analysis.

The results show that there is a consistent effect of concurrent versus non-concurrent applications of the two earthquake components in that the concurrent components always produced higher displacements and higher shear forces than the non-concurrent case. However, the difference was very much a function of the earthquake records. Displacements were higher by from 2% to 57% and shears higher by from 1% to 38%. The average displacement was 15% higher, the average shear 11% higher.

Increasing the yield strength of the isolation system generally, but not always, reduced concurrent displacements and shears to values less than the non-concurrent values with the lower yield strength. In this example, an increase in yield level would reduce the mean displacement to less than the non-concurrent value and reduce the base shear to only slightly more than the non-concurrent value.

	Mean Displacement (mm)	**Mean Base Shear Coefficient**
Concurrent	706	0.320
Unidirectional	615	0.289
Concurrent with Increased F_y	570	0.292

Although in this example an increase in yield level was effective in counteracting concurrency effects the results were not consistent enough to demonstrate that this will always be an effective strategy. It is recommended that you assess the effects of concurrency on a project specific basis and test whether an increase in yield level is justified.

Note that an increase in yield level may require higher design forces for the structure above the isolation system if design is governed by the requirement for design base shear being at least 1.5 times the yield level of the system. If design is governed by this criterion then you may want to accept higher displacements and shears from concurrency effects rather than re-design for a higher yield level.

11 CONNECTION DESIGN

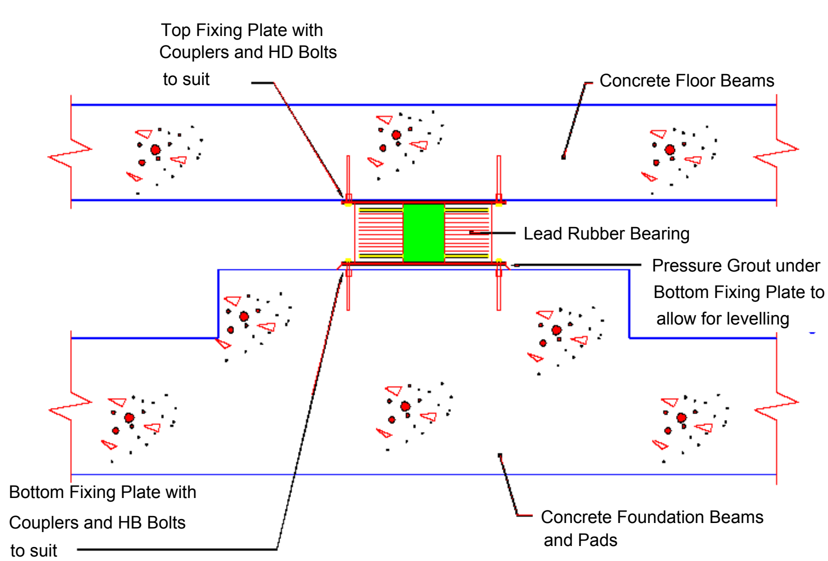
11.1 ELASTOMERIC BASED ISOLATORS

Early seismic isolation bearings used load plates bolted to a steel plate bonded internally in the bearing. Manufacturing technology has now improved such that the majority of seismic isolation bearings are manufactured with flange plates, or load plates, bonded to the bearing top and bottom during manufacture. These plates are larger in plan dimension than the isolator and are used to connect the bearing to the foundation below and the structure above.

The load plates may be circular, square or rectangular, depending on project requirements. The amount of overhang depends on the bolt sizes and the seismic displacement. The bolts must be located far enough from the isolator such that they do not damage the cover rubber during maximum seismic displacements. Square load plates allow a smaller plan dimension than circular plates and so are often used for this reason.

The isolators are installed between the foundation and the structure, as shown conceptually in Figure 11-1. The connection design must ensure that the maximum forces are safely transferred from the foundation through the bearing to the structure above.

FIGURE 11-1 TYPICAL INSTALLATION IN NEW BUILDING

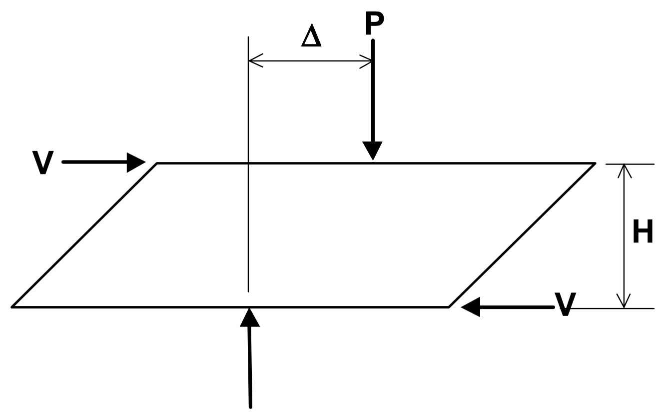


11.1.1 Design Basis

The connection of the isolation bearing to a structure must transmit shear forces, vertical loads and bending moments. Bending moments are due to primary (VH) and secondary (PΔ) effects. Design for shear is relatively straightforward. Design for bending moments is complicated by the unknown shape of the compressive block, especially under extreme displacements.

It is recognized that the design approach used here is simplistic and not a true representation of the actual stress conditions at the connection interface. However, the procedure has been shown to be conservative by prototype testing which has used less bolts, and thinner plates, than would be required by the application of this procedure.

Figure 11-2: Forces on Bearing in Deformed Shape



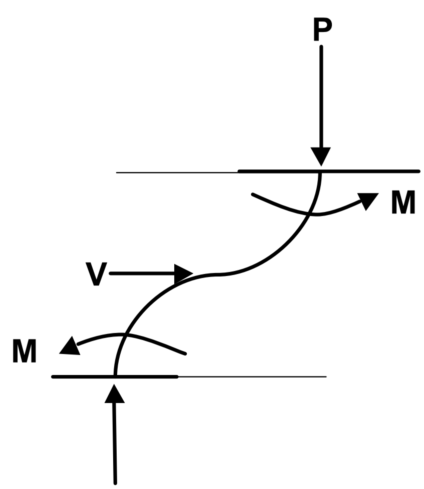
Bearing design includes the mounting plate and mounting bolts. The design basis depends on project specifications, but generally is either AASHTO allowable stress values, with a 4/3 stress increase factor for seismic loads, or AISC requirements.

11.1.2 Design Actions

Connections are designed for two conditions, (1) maximum vertical load and (2) minimum vertical load, each of which is concurrent with the maximum earthquake displacement and shear force.

The bearing is bolted to the structure top and bottom and so acts as a fixed end column for obtaining design moments. Figure 11-2 shows the forces on the bearing. Figure 11-3 shows how the actions may be calculated as an equivalent column on the centerline of the bearing.

Figure 11-3: Equivalent Column Forces



The total moment due to applied shear forces, VH, plus eccentricity, PΔ, is resisted by equal moments at the top and bottom of the isolator. These design moments are equal to:

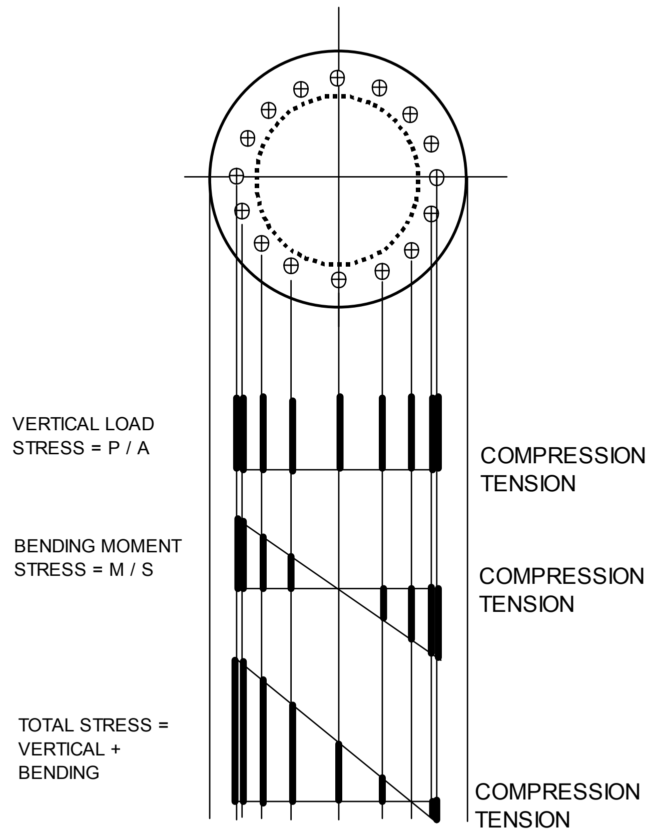
$$M = \frac{1}{2}(VH + P\Delta)$$

11.1.3 CONNECTION BOLT DESIGN

The design procedure adopted for the mounting plate connection is based on the simplified condition shown in Figure 11-4, where the total axial load and moment is resisted by the bolt group. In Figure 11-4, the area used to calculate P/A is the total area of all bolts and the section modulus used to calculate M/S is the section modulus of all bolts. Figure 11-4 is for a circular load plate. A similar approach is used for other shapes.

It is recognized that in reality the compression forces will be resisted by compressive stresses in the plate rather than by the bolts. However, the bearing stiffness to calculate the modular ratio, and so the neutral axis position, is unknown. This is why the bolt group assumption is made. This assumption is conservative as it underestimates the actual section modulus and so is an upper bound of bolt tension.

FIGURE 11-4: ASSUMED BOLT FORCE DISTRIBUTION



The procedure for bolt design is:

1. Calculate the shear force per bolt as V/n, where n is the number of bolts.
2. Calculate the axial load per bolt as P/A
3. Calculate the tension per bolt due to the moment as M/S where S is the section modulus of the bolt group.
4. Calculate the net tension per bolt as P/A – M/S

5. Check the bolt for combined shear plus tension.

This is done for maximum and minimum vertical loads.

11.1.4 Load Plate Design

For a circular load plate, the assumed force distribution on which load plate calculations are based is shown in Figure 11-5. Bending is assumed to be critical in an outstanding segment on the tension side of the bearing. The chord defining the segment is assumed to be tangent to the side of the bearing.

This segment is loaded by three bolts in the example displayed in Figure 5-11.

Conservatively, it is assumed that all bolts (three in this example) have the maximum tension force and also that all three bolts have the lever arm of the furthest bolt.

The design procedure adopted for a square load plate connection is based on the condition shown in Figure 11-6. Loading is assumed along the direction of the diagonal as this is the most critical for the bolt layout used with this type of load plate.

Figure 11-6: Circular Load Plate

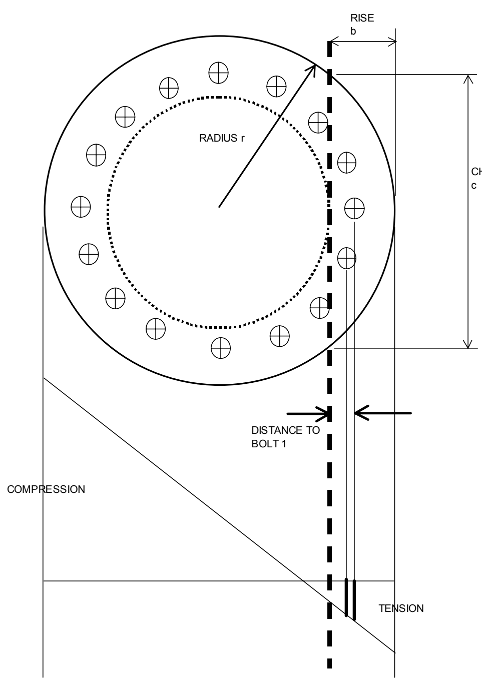
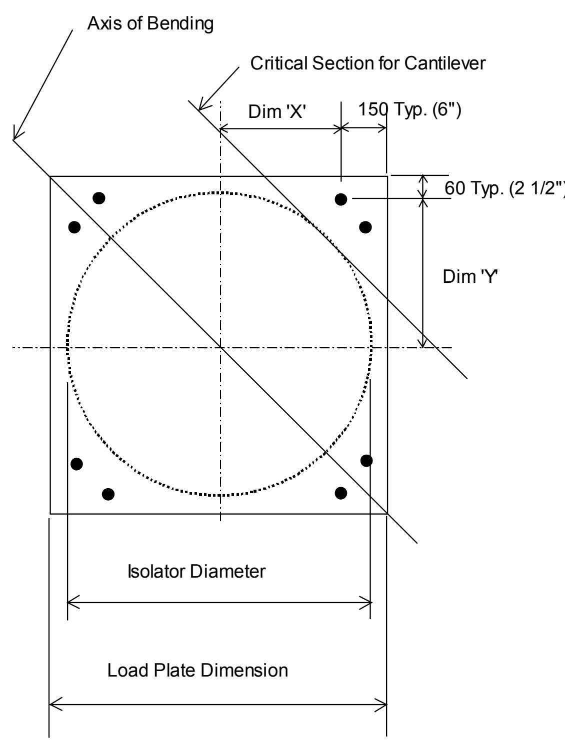


Figure 11-5: Square Load Plate



11.2 Sliding Isolators

For sliding isolators the eccentricity of the load at maximum displacements depends on the orientation of the bearing. If the slide plate is at the top then under maximum displacements the vertical load will apply a P-Δ moment which must be resisted by the structural system above the isolators. If the slide plate is at the bottom then the eccentricity will load the foundation below the isolator but will not cause moments above. For this reason, most seismic applications of sliding isolators are more effective if the slide plate in located at the bottom.

Most sliding bearings are designed with a low coefficient of friction at the sliding interface. In theory, they do not require any shear connection. The friction at the interface of the bearing to the structure above and to the foundation below, usually steel on concrete, will have a higher coefficient of friction and so slip will not occur. In practice, bolts are usually used at each corner of the slide plate and the sliding component is bolted to the structure above, also usually with four bolts.

Most types of sliding bearings, such as pot bearings and friction pendulum bearings, are proprietary items and the supplier will provide connection hardware as part of supply.

11.3 Installation Examples

Figure 11-7 to 11-12 are example installation details taken from isolation projects which we have completed. These include both new and retrofit projects. Although these details cannot be used directly for other projects, they should provide you with an indication of installation methods and the extent of strengthening associated with installation.

FIGURE 11-7 EXAMPLE INSTALLATION : NEW CONSTRUCTION

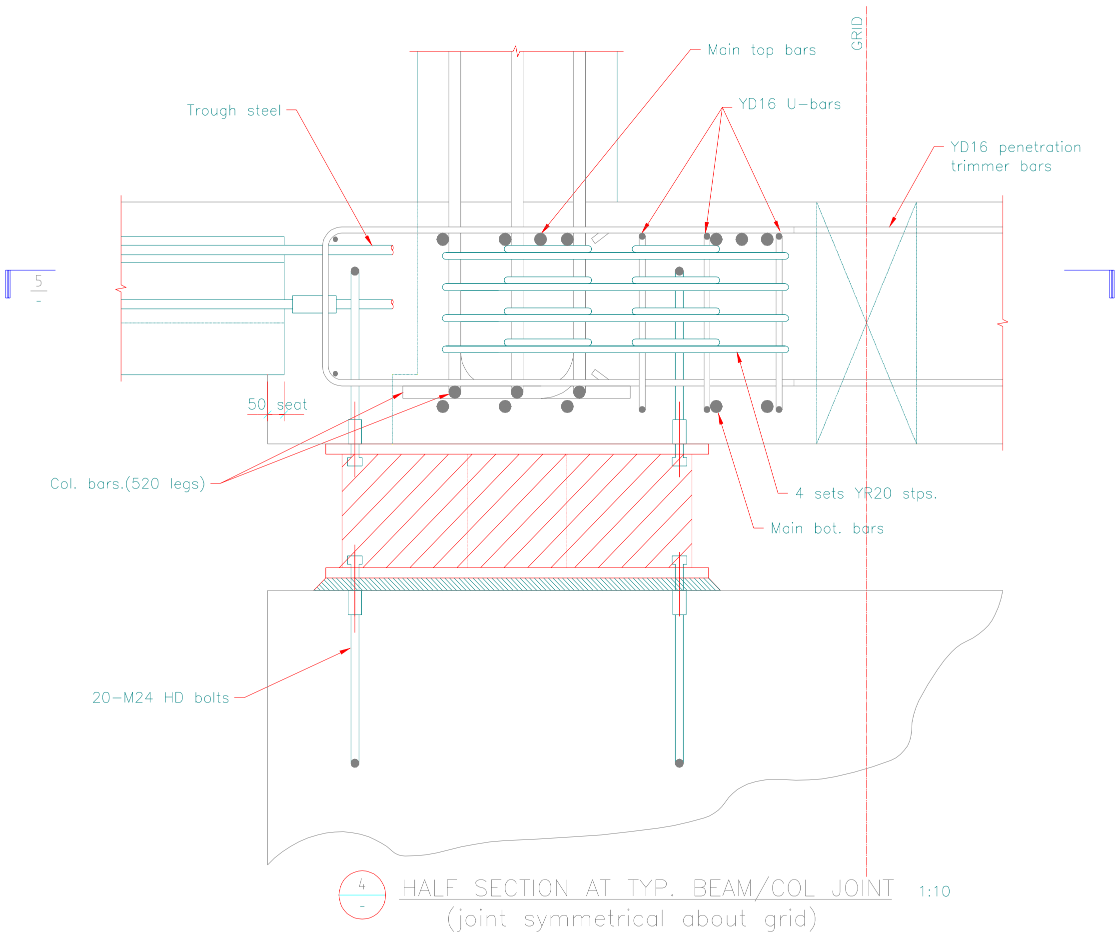


FIGURE 11-8 EXAMPLE INSTALLATION : EXISTING MASONRY WALL

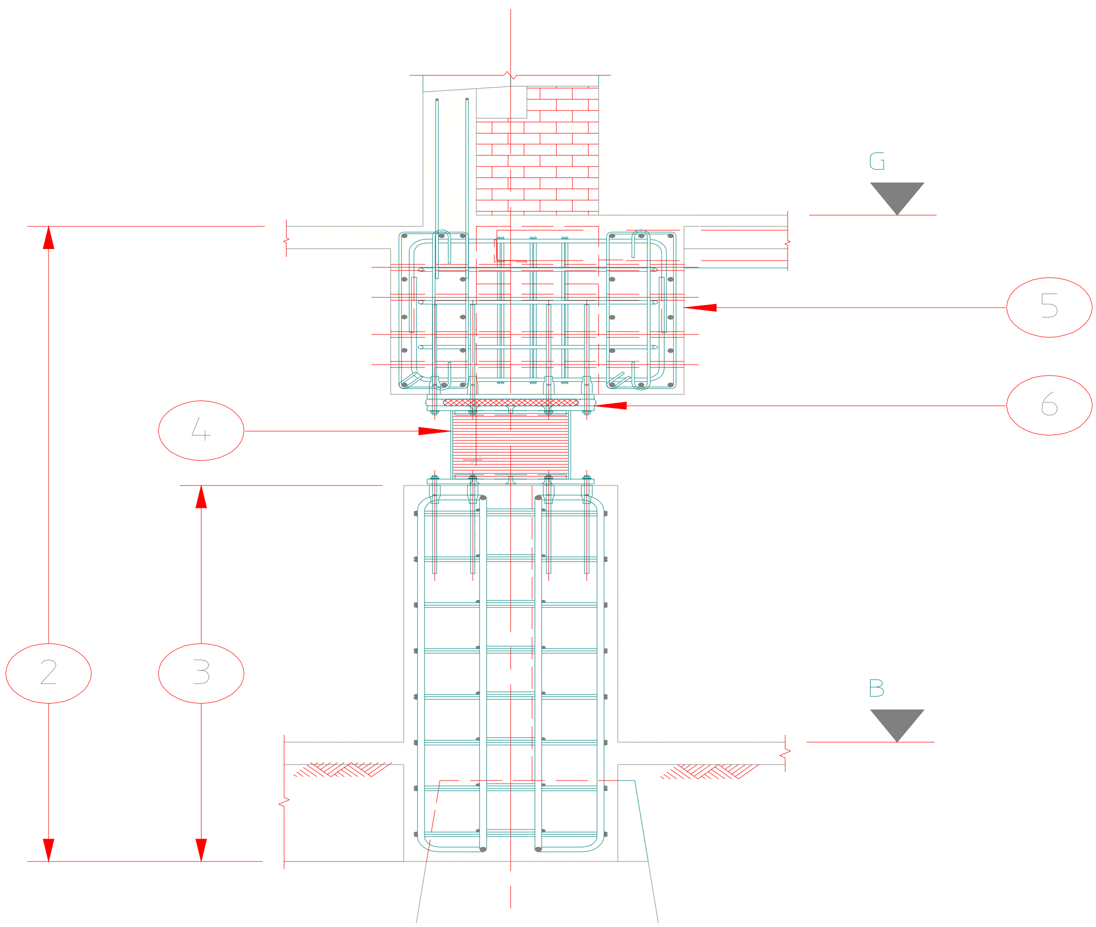


FIGURE 11-9 EXAMPLE INSTALLATION : EXISTING COLUMN

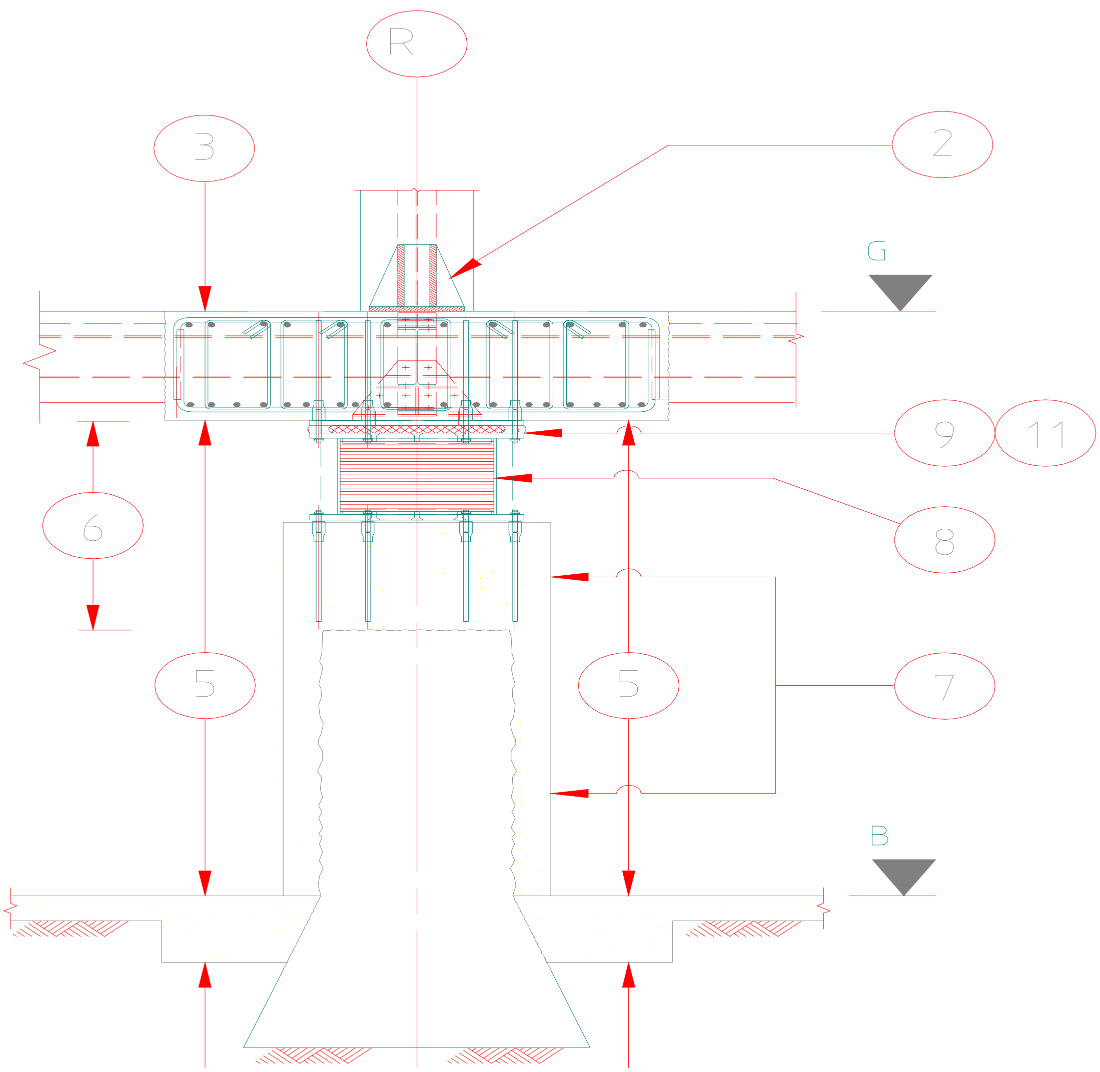


FIGURE 11-10 EXAMPLE INSTALLATION : EXISTING MASONRY WALL

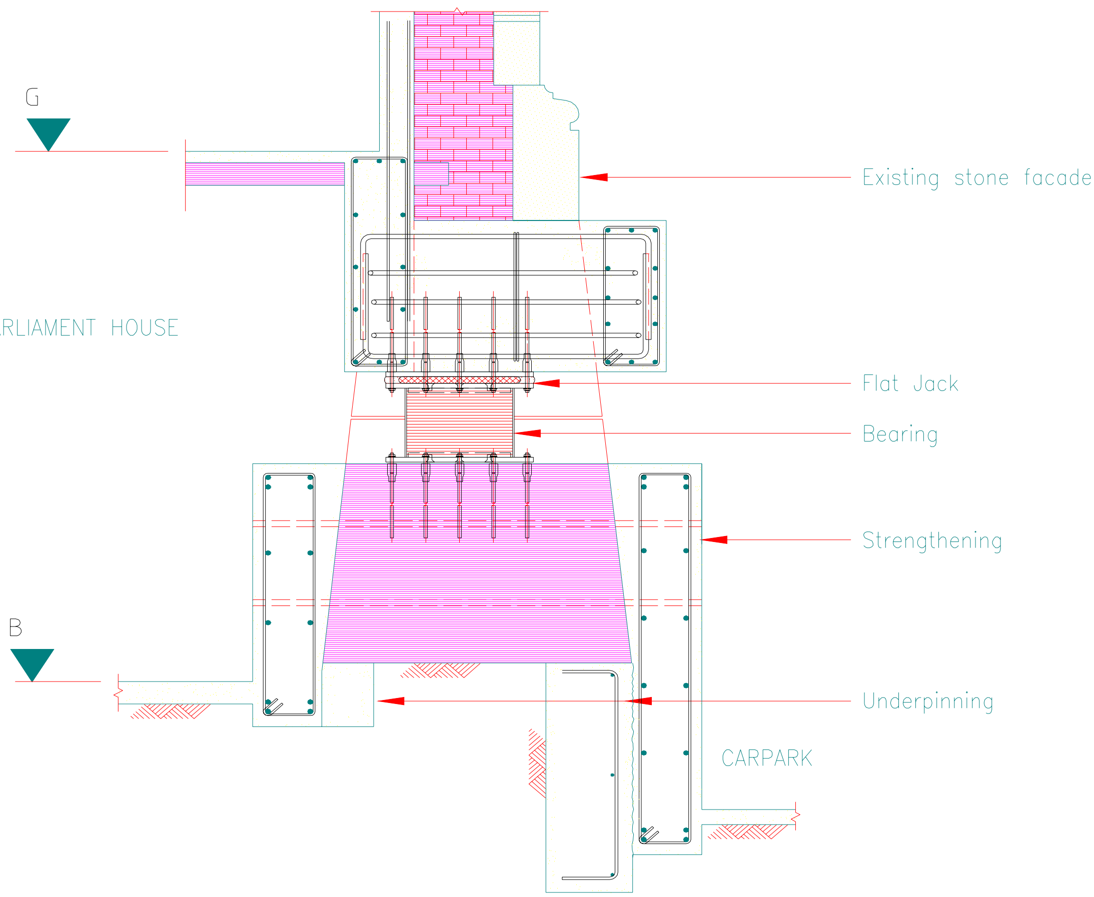


fig. 4 – Typical bearing installation

FIGURE 11-11 EXAMPLE INSTALLATION : STEEL COLUMN

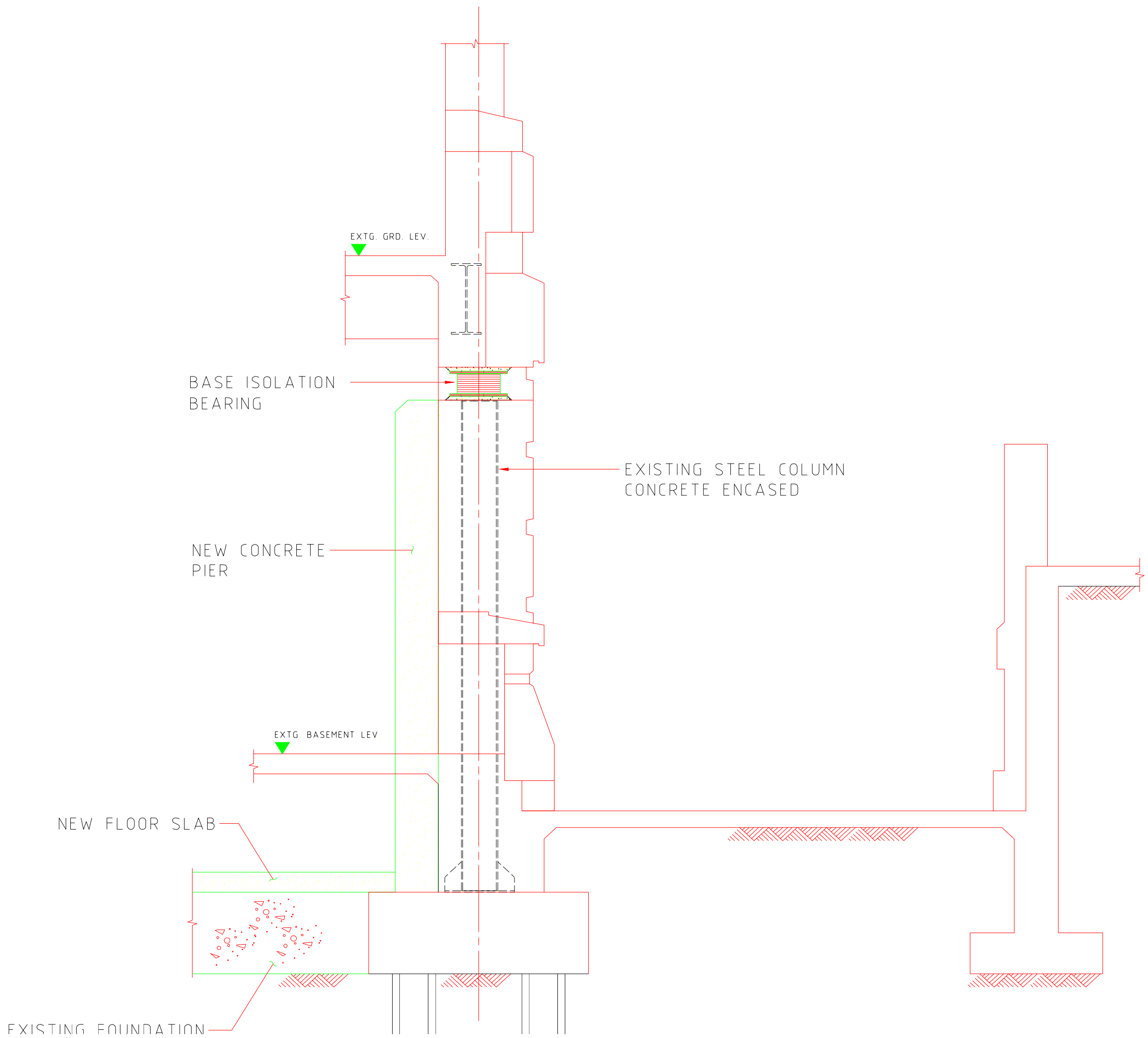
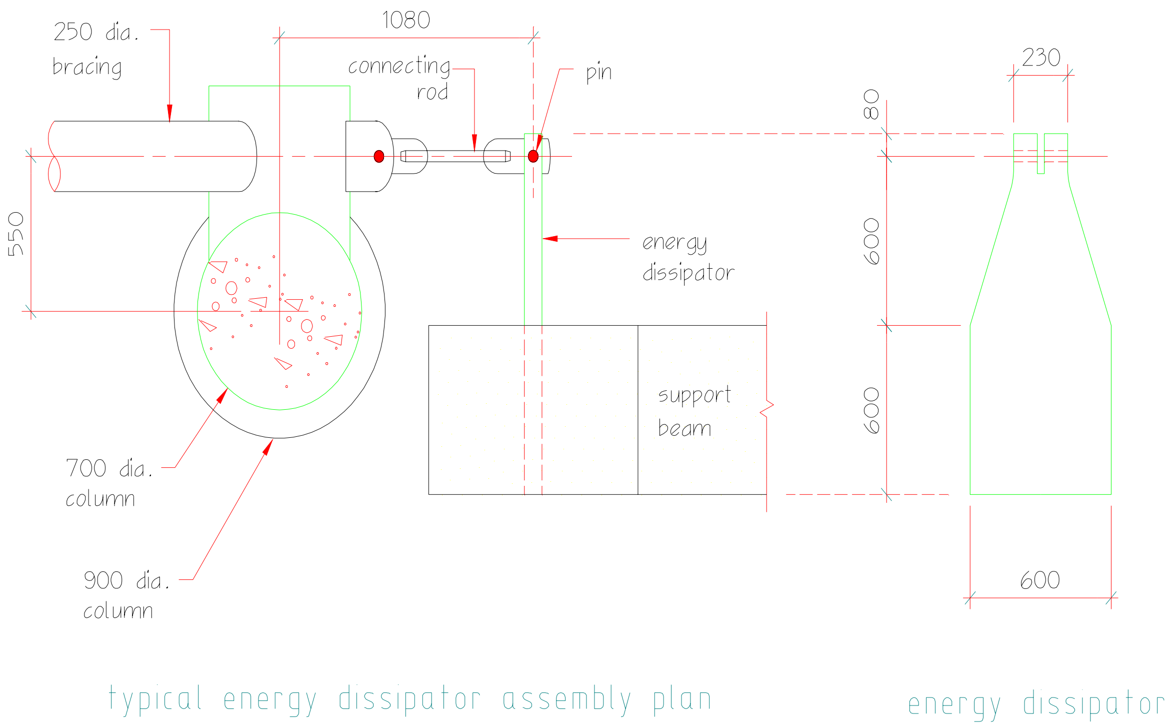


FIGURE 11-12 EXAMPLE INSTALLATION : STEEL ENERGY DISSIPATOR



12 STRUCTURAL DESIGN

12.1 Design Concepts

The isolation system design and evaluation procedures produce the maximum base shears, displacements and structural forces for each level of earthquake, usually the DBE and MCE. These represent the maximum elastic earthquake forces that will be transmitted through the isolation system to the structure above. Even though isolated buildings have lower seismic loads than non-isolated buildings, it is still not generally cost effective to design for elastic performance at the MCE level and sometimes yielding may be permitted at the DBE level.

Building projects to date in New Zealand have generally been designed elastically to the DBE level of loading with some ductility demand at the MCE level. This is because of the nature of buildings isolated so far, which have been either older buildings with limited ductility or buildings providing essential services where a low probability of damage is required.

For new buildings in the ordinary category, design forces are usually based on the DBE level of load reduced to account for ductility in the structural system. This is the approach taken by the UBC for new buildings. An isolated building, if designed elastically to the DBE, will likely have higher design forces than a ductile, non-isolated building which would be designed for forces reduced by ductility factors of 6 or more. These higher forces, plus the cost of the isolation system, will impose a significant first cost penalty on the isolated building.

Total life cycle costs, incorporating costs of earthquake damage over the life of the building, will usually favor the isolated building in high seismic regions. However, life cycle cost analysis is rare for non-essential buildings and few owners are prepared to pay the added first cost.

The UBC addresses this issue by permitting the structural system of an isolated building to be designed as ductile, although the ductility factor is less than one-half that specified for a non-isolated building. This provides some added measure of protection while generally reducing design forces compared to an equivalent non-isolated building. This approach would seem to be permitted by NZS3101 for New Zealand buildings and so it is recommended that the UBC approach be modified for local conditions for projects outside the U.S.

12.2 UBC Requirements

The UBC requirements for the design of base isolated buildings differ from those for non-isolated buildings in three main respects:

1. The importance factor, I, for a seismic isolated buildings is taken as 1.0 regardless of occupancy. For non-isolated buildings I = 1.25 for essential and hazardous facilities. As discussed later, a limitation on structural design forces to the fixed base values does indirectly include I in the derivation of design forces.

2. The numerical coefficient, R, which represents global ductility is different for isolated and non-isolated buildings.

3. For isolated buildings there are different design force levels for elements above and elements below the isolation interface.

12.2.1 Elements Below the Isolation System

The isolation system, the foundation and all structural elements below the isolation system are to be designed for a force equal to:

$$V_B = k_{D\max} D_D$$

Where k_{Dmax} is the maximum effective stiffness of the isolation system at the design displacement at the center of mass, D_D. All provisions for non-isolated structures are used to design for this force.

In simple terms, this requires all elements below the isolators to be designed elastically for the maximum force that is transmitted through the isolation system at the design level earthquake.

One of the more critical elements governed by elastic design is the total moment generated by the shear force in the isolation system plus the P-Δ moment. As discussed in the connection design chapter, the moment at the top and bottom of an elastomeric type isolation bearing is:

$$M = \frac{1}{2}(V_B H + P D_D)$$

where H is the total height of the bearing and P the vertical load concurrent with V_B. The structure below and above the bearing must be designed for this moment. For some types of isolators, for example sliders, the moment at the location of the slider plate will be PD_D and the moment at the fixed end will be VH.

12.2.2 ELEMENTS ABOVE THE ISOLATION SYSTEM

The structure above the isolators is designed for a minimum shear force, V_S, using all the provisions for non-isolated structures where:

$$V_S = \frac{k_{D\max} D_D}{R_I}$$

This is the elastic force in the isolation system, as used for elements below the isolators, reduced by a factor R_I that accounts for ductility in the structure.

Table 12-1 lists values of R_I for some of the structures included in UBC. For comparison the equivalent ductility factor used for a non-isolated building, R, is also listed in Table 12-1. UBC includes other structural types not included in this table so consult the code if your structural system does not fit those listed in Table 12-1. All systems included in Table 12-1 are permitted in all seismic zones.

The values of R_I are always less than R, sometimes by a large margin. The reason for this is to avoid high ductility in the structure above the isolation system as the period of the yielded structure may degrade and interact with that of the isolation system.

TABLE 12-1 STRUCTURAL SYSTEMS ABOVE THE ISOLATION INTERFACE

Structural System	Lateral Force Resisting System	Fixed Base R	Isolated R_I
Bearing Wall System	Concrete Shear Walls	4.5	2.0
	Masonry Shear Walls	4.5	2.0
Building Frame System	Steel Eccentrically Braced Frame (EBF)	7.0	2.0
	Concrete Shear Walls	5.5	2.0
	Masonry Shear Walls	5.5	2.0
	Ordinary Steel Braced Frame	5.6	1.6
	Special Steel Concentric Braced Frame	6.4	2.0
Moment Resisting Frame System	Special Moment Resisting Frame (SMRF)		
	Steel	8.5	2.0
	Concrete	8.5	2.0
	Intermediate Moment Resisting Frame (IMRF)		
	Concrete	5.5	2.0
	Ordinary Moment Resisting Frame (OMRF)		
	Steel	4.5	2.0
Dual Systems	Shear Walls		
	Concrete with SMRF	8.5	2.0
	Concrete with steel OMRF	4.2	2.0
	Masonry with SMRF	5.5	2.0
	Masonry with Steel OMRF	4.2	2.0

Structural System	Lateral Force Resisting System	Fixed Base R	Isolated R_I
	Steel EBF		
	With Steel SMRF	8.4	2.0
	With Steel OMRF	4.2	2.0
	Ordinary braced frames		
	Steel with steel SMRF	6.5	2.0
	Steel with steel OMRF	4.2	2.0
	Special Concentric Braced Frame		
	Steel with steel SMRF	7.5	2.0
	Steel with steel OMRF	4.2	2.0
Cantilever column Building systems	Cantilevered column elements	2.2	1.4

There are design economies to be gained by selecting the appropriate structural system. For example, for a non-isolated building the design forces for a special moment resisting steel frame are only about 53% of the design forces for an ordinary steel moment resisting frame. However, for an isolated moment frame the design force is the same regardless of type. In the latter case, there is no benefit for incurring the extra costs for a special frame and so an ordinary frame could be used. Be careful with this because, as discussed later, there may be some penalties in structural design forces if the ratio of R/R_I is low.

Table 12-1 also shows that some types of building are more suited to isolation, in terms of reduction in design forces, than others. For bearing wall systems the isolation system only needs to reduce response by a factor of 4.5 / 2 = 2.25 or more to provide a net benefit in design forces. On the other hand, for an eccentrically braced frame the isolation system needs to provide a reduction by a factor of 7.0 / 2.0 = 3.5 before any benefits are obtained, a 55% higher reduction.

The value of V_S calculated as above is not to be taken as lower than any of:

1. The lateral seismic force for a fixed base structure of the same weight, W, and a period equal to the isolated period, T_D.

2. The base shear corresponding to the design wind load.

3. The lateral force required to fully activate the isolation system factored by 1.5 (e.g. 1½ times the yield level of a softening system or static friction level of a sliding system).

In many systems one of these lower limits on V_S may apply and this will influence the design of the isolation system.

<u>Fixed Base Structure Shear</u>

In general terms, the base shear coefficient for a fixed base structure is:

$$C = \frac{C_V I}{RT}$$

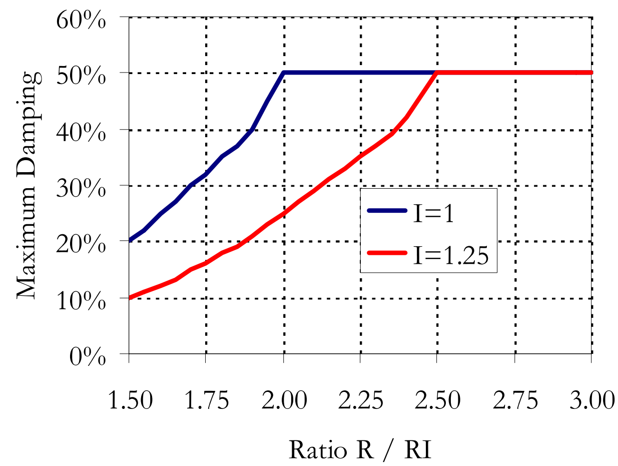
and for an isolated structure

$$C_I = \frac{C_{VD}}{R_I BT}$$

There is a change in nomenclature in the two sections and in fact $C_V = C_{VD}$ and so to meet the requirements of Criterion 1 above, $C_I \geq C$, the two equations can be combined to provide:

$$B \leq \frac{R}{R_I I}$$

FIGURE 12-1 LIMITATION ON B



Therefore, the limit on forces to be not lower than the fixed base shear for a building of similar period effectively limits the amount of damping in the isolation system, measured by B, which can be used to reduce structure design forces.

The range of R/R_I (Table 12-1) is from 1.57 to 4.25. Figure 12-1 shows the limitation on the damping that can be used to derive structure design forces for this range of factors.

Most systems target 15% to 35% equivalent damping at the design basis earthquake level. As seen from Figure 12-1, this damping may not be fully used to reduce design forces where the ratio of R/R_I is less than about 1.8, or less than 2.2 for a structure with an importance factor I = 1.25.

This limitation on design base shears will generally only apply when you have a structure with a relatively non-ductile lateral load system (low R) and/or an importance factor greater than 1.0.

Design Wind Load

Most isolation systems are installed in relatively heavy buildings because isolation is most effective for high mass structures. The yield level selected for optimum damping, usually 5% to 15% of the weight of a structure, is generally much higher than the wind load, which is often less than 2% of the weight. As the isolation system yield level is always set higher than the wind load then the third criterion, discussed below, will govern rather than wind.

Factored Yield Level

The requirement that the design lateral force be at least 1½ times the yield force will govern in many isolation designs and will be a factor in selection of the isolator properties. High yield forces are used to increase the amount of damping in a system to control displacements. Generally, the higher the seismic load and the softer the soil type the higher the optimum yield level.

The design spreadsheet calculates the value of V_S and the lower limits imposed by the three criteria above. Design is often a process of adjusting the yield level (for example, lead core size in LRBs) until the value of V_S calculated from the isolation system performance is approximately equal to $1.5F_y$.

The drift limitations for isolated structures may also limit the design of the structural system:

Response Spectrum Analysis	$\delta \leq 0.015 / R_I$
Time History Analysis	$\delta \leq 0.020 / R_I$

These are more restrictive than for non-isolated buildings where the limits are:

Period $<$ 0.7 seconds	$\delta \leq 0.025 / 0.7R$
Period $\geq$ 0.7 seconds	$\delta \leq 0.020 / 0.7R$

Although the UBC is not specific in this respect, it can probably be assumed that the value of R_I can be lower but not greater than the values specified in Table 12-1 above. If one of the three UBC lower limits apply to the design lateral force for the structure then you should calculate the actual value of R_I that corresponds to this force. This will effect the calculation of the drift limit.

12.3 MCE Level of Earthquake

The UBC defines a total design displacement, D_{TD}, under the design basis earthquake (DBE) and a total maximum displacement, $D_{TM,}$ under the maximum capable earthquake (MCE). The vertical load-carrying elements of the isolation system are required to be stable for the MCE displacements. The MCE displacements also define the minimum separations between the building and surrounding retaining walls or other fixed obstructions.

There are no requirements related to the MCE level of load for design of the structural elements above or below the isolation interface. Presumably, it is assumed that the elastic design of elements below the isolation system produces sufficient overstrength for MCE loads and that the limitations on R_I provide sufficient ductility above the isolation system for MCE loads.

12.4 Nonstructural Components

The UBC requires components to be designed to resist seismic forces equal to the maximum dynamic response of the element or component under consideration but also allows design to be based on the requirements for non-isolated structures.

For components, there are three aspects of the dynamic response which define the maximum force:

1. The maximum acceleration at the location of the component. UBC defines this for non-isolated components as a function of the ground acceleration, C_a, and the height of the component, h_x, relative to the height of the structure, h_r:

$$C = C_a (1 + 3\frac{h_x}{h_r})$$

2. Component amplification factor, which defines the extent of amplification when flexible components are excited by structural motion. In UBC this is defined as a_p.

3. The ductility of the part can be used to reduce the design forces in a similar manner to R is used for the structural system. UBC defines this as R_p.

UBC requires forces to be factored by the importance factor for the part, I_p, and a simplification also allows the component to be designed for the maximum acceleration ($4C_a$) and ignore both a_p and R_p.

For isolated structures, it is usual to replace the value of C calculated above with the peak floor accelerations obtained from the time history analysis. Values of a_p and R_p as for non-isolated structures are then used with this value.

As time history analyses are generally used to evaluate isolated structures it is possible to generate floor response spectra and use these to obtain values of a_p, defined as the spectral acceleration at the period of the component. As this requires enveloping a large number of spectra, this procedure is usually only used for large projects.

12.5 Bridges

Although there are differences in detail, the same general principles for the structural design of bridges apply as for buildings:

1. The elastic forces transmitted through the isolation system are reduced to take account of ductility in the sub-structure elements. The 1991 AASHTO permitted use of R values equal to that for non-isolated bridges but in the 1999 AASHTO this has been reduced to one-half the value for non-isolated bridges. This provides a range of R_I from 1.5 to 2.5, which implies relatively low levels of ductility. R_I need not be taken less than 1.5.

2. A lower limit on design forces is provided by non-seismic loads, the yield level of a softening system or the friction level of a sliding system. AASHTO does not require the 1.5 factor on the yield level or friction level that is specified in the UBC. For buildings non-seismic lateral loads are usually restricted to wind but bridges have a number of other cases which may influence design (wind, longitudinal force, centrifugal force, thermal movements etc.).

3. Connection design forces for the isolators are based on full elastic forces, that is, R = 1.0.

Although bridges do not generally use the DBE and MCE terminology typical of buildings, design of the structure is based on a 475 year return period earthquake and the isolators must be tested to displacement levels equivalent to that for a 2,400 year return period earthquake. The 2,400 year displacement is obtained by applying a factor of 2.0 to the design displacements for low to moderate seismic zones (accelerations $\leq 0.19g$) and 1.5 for high seismic zones (accelerations $> 0.19g$).

13 SPECIFICATIONS

13.1 GENERAL

Examples of specifications for seismic isolation can be obtained from the job files of projects we have completed and sample specifications reflecting current US practice are provided in Naeim and Kelly [1999]. Figure 13-1 shows the major headings that are generally included in base isolation specifications.

FIGURE 13-1 SPECIFICATION CONTENTS

1. PRELIMINARY
2. SCOPE
3. ALTERNATIVE BEARING DESIGNS
4. SUBMITTALS
5. REFERENCES
6. BEARING DESIGN PROPERTIES
7. BEARING CONSTRUCTION
   7.1. Dimensions
   7.2. Fabrication
   7.3. Fabrication Tolerances
   7.4. Identification
   7.5. Materials
8. DELIVERY, STORAGE, HANDLING AND INSTALLATION
9. TESTING OF BEARINGS
   9.1. General
   9.2. Production Testing
      9.2.1. Sustained Compression Tests
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   9.3. Rubber Tests
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      9.4.1. General
      9.4.2. Definitions
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      9.4.6. Design Properties Of The Isolation System
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10. WARRANTIES

Most of our projects have provided a particular bearing design which manufacturers can bid directly and also permit alternate systems to be submitted. Procedures need to be specified for the manner in which alternate systems are designed and validated. Generally, this will require that you specify the seismic design parameters, the form of the analysis model and the performance requirements for the system. Performance requirements almost always include maximum displacements and maximum base shear coefficients and may also include limits on structural drifts, floor accelerations, member forces and other factors which may be important on a specific project.

One aspect of the specification to be careful of is not to mix prescriptive and performance requirements. If performance requirements of the isolation system are specified, including the evaluation method to define this performance and the testing required to validate the properties, then you should not also specify design aspects such as the shear modulus of the rubber (for LRBs or HDR bearings) or the radius of curvature (for curved sliders).

Even for the complying design, you should place the onus on the manufacturer to verify the design because aspects of isolation system performance are manufacturer-specific, such as damping or lead core effective yield level. We often include a clause such as the following:

> *The manufacturer shall check the bearing sizes and specifications before tendering. If the manufacturer considers that some alteration should be made to the bearing sizes and/or properties to meet the stated design performance requirements the engineer shall be advised of the alterations which the manufacturer intends making with the tender.*

13.2 Testing

Code requirements for base isolation require testing of prototype bearings, to ensure that design parameters are achieved, and additional production testing is performed as part of quality control. The UBC and AASHTO codes specify procedures for prototype tests and most projects generally follow the requirements of one of these codes. These require that two bearings of each type be subjected to a comprehensive sequence of tests up to MCE displacements and with maximum vertical loads.

Because of the severity of the tests, prototype bearings are not used in the structure, with some exceptions in low seismic zones. The extra isolators can add significantly to the costs of projects that have a small number of bearings, or a number of different types.

Production tests usually include compressive stiffness testing of every bearing plus combined shear/compression testing of from 20% to 100% of isolators. Compression testing has been found difficult to use as a control parameter to ensure consistency. This is because typical vertical deflections are in the order of 2 mm to 5 mm whereas for large bearings the out of parallel between top and bottom surfaces will be of the same order. This distorts apparent compressive stiffness. On recent projects, these difficulties have lead to the measurement of shear stiffness on all production bearings to ensure consistency.

Testing requirements in the UBC have become more stringent in later editions of the code. In particular, the 1997 edition requires cyclic testing to the MCE displacements whereas earlier editions required a single loading to this displacement level. This requires high capacity test equipment because MCE displacements may be 750 mm or more. Test equipment that can cycle to this magnitude of displacement is uncommon. Where we are not required to fully comply with UBC it may be cost-effective to use the earlier UBC requirement of a simple stability test to the MCE displacement. This may allow manufacturers to bid who would not otherwise bid because of insufficient test capacity. This needs to be assessed on a project by project basis.

14 BUILDING EXAMPLE

14.1 Scope Of This Example

This example design is based on the submittal for a small health care facility located in New Zealand. The contract documents for supply of isolation bearings specified that design calculations were to be provided for the isolators and the results of dynamic analysis of the structure modeled using the assumed isolator properties.

The Structural Engineer had set performance criteria for the isolation system based on considerations of seismic loads, building weights and performance requirements and the isolation system had to meet the following requirements:

1. Total design displacement not to exceed 350 mm for DBE.

2. Total maximum displacement not to exceed 400 mm for MCE

3. Elastic base shear for DBE not to exceed 0.65.

4. Interstory drift ratio of the structure above the isolation system not to exceed 0.0100.

The building was a low-rise structure located in a high seismic zone with relatively light column loads. As will be seen in the design of the isolation system, this limits the extent of the period shift and so the degree of isolation which can be achieved.

HCG provided the isolation design and analysis using a lead-rubber isolation system as submitted by the successful isolation system bidder, Skellerup Industries.

14.2 Seismic Input

A site-specific seismic assessment of the site developed a suite of time histories to define the DBE and MCE levels of load. Five time histories defined the lower level earthquake and four the upper level. For each level, one modified (frequency scaled) and four or three as-recorded time histories were used. The records are listed in Table 14-1.

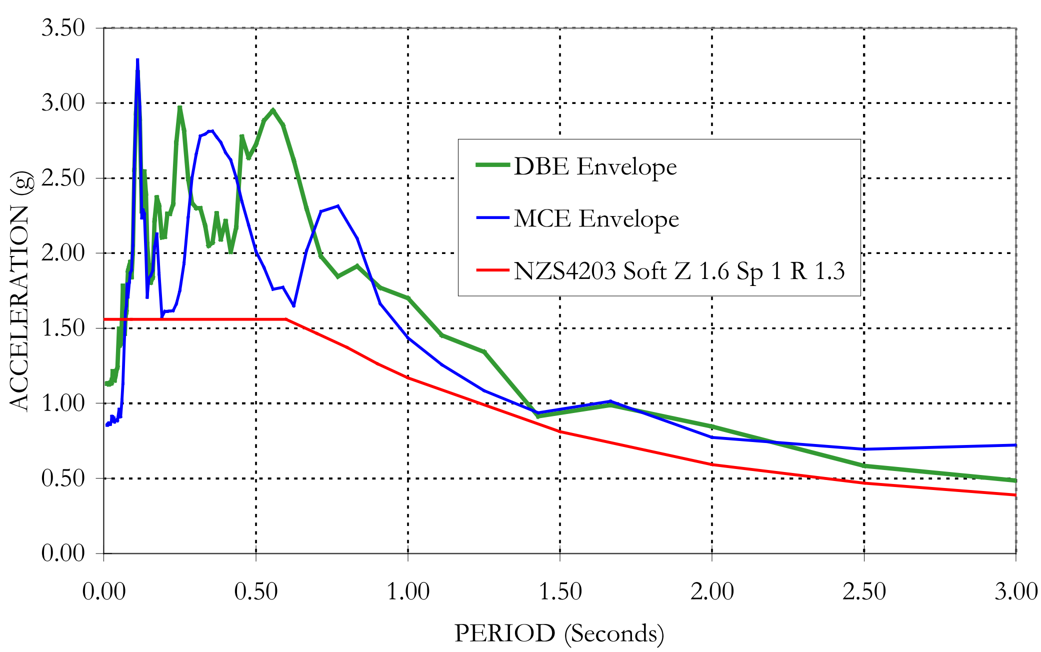
Figure 14-1 plots the envelopes of the 5% damped response spectra of the records used to define the DBE and MCE levels of load respectively. Also plotted on Figure 14-1 is the code response spectrum for a hospital building on this site.

The seismic definition has an unusual characteristic in that the MCE level of load is approximately the same as the DBE level of load whereas usually it would be expected to be from 25% to 50% higher. This is because the earthquake probability at this location is dominated by the Wellington Fault which has a relatively short return period and is expected to generate an earthquake of high magnitude.

TABLE 14-1 INPUT TIME HISTORIES

	Level	Filename	Record	Scale Factor
EQ1	DBE	EL40N00E	El Centro 1940 NS	3.25
EQ2		EL40N90W	El Centro 1940 EW	3.90
EQ3		HOLSE000	Hollister and Pine0 deg, 1989 Loma Prieta	1.69
EQ4		GZ76N00E	Gazli, 1976 NS	1.17
EQ5		HMEL40NE	Modified El Centro 1940 NS	1.30
EQ6	MCE	GZ76N00E	Gazli 1976 NS	1.20
EQ7		SYL360	Sylmar Hospital 360 deg, 1994 Northridge	1.00
EQ8		EL79723	El Centro Array No. 7 230 deg, 1979	1.70
EQ9		MEL79723	Modified El Centro Array No. 7, 1979	1.00

FIGURE 14-1 5% DAMPED ENVELOPE SPECTRA



14.3 Design Of Isolation System

The isolation system was designed using the procedures incorporated in the HCG design spreadsheets. A series of design studies was performed using the spreadsheet to optimize the isolator dimensions, shear modulus and lead core diameter for the level of seismic load as determined from envelope spectra of the time histories. These studies resulted in an isolator size of 675 mm diameter x 380 mm high (total rubber thickness 210 mm) a 130 mm diameter lead core. Design was based on a moderately soft rubber (G = 0.60 MPa).

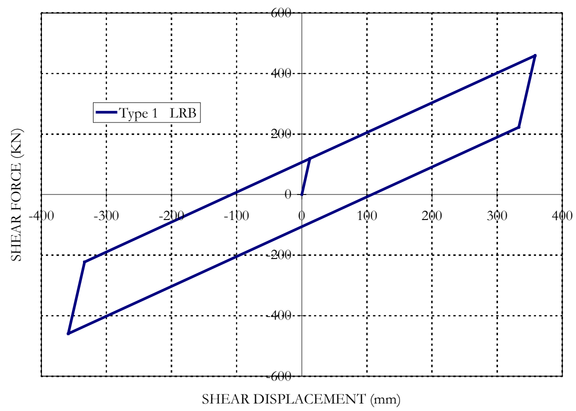
Figure 14-2 summarizes the performance of the isolation system as designed. This isolation system configuration provided an effective period of approximately 1.5 seconds. The lead cores provided displacement-dependent hysteretic damping. The equivalent viscous damping ranged from 32% at 50 mm displacement to 13% at 400 mm displacement.

Figure 14-2 Summary of Isolation Design

PERFORMANCE SUMMARY

	Type 1	Pad B	DBE	MCE
Gravity Strain F.S.	16.34			
Buckling F.S	12.25			
DBE Strain F.S	2.14			
Buckling F.S	3.71			
MCE Strain F.S	1.76			
Buckling F.S	2.81			
Reduced Area / Gross Area	25.6%			
Maximum Shear Strain	196%			
Effective Period T_D T_M			1.455	1.479
Displacement D_D D_M			313	358
Total Displacements D_{TD} D_{TM}			360	412
Force Coefficient V_b / W			0.596	0.659
Force Coefficient V_s / W			0.298	
1.5 x Yield Force / W			0.229	
Wind Force / W			0.000	
Fixed Base V at T_D			0.348	
Governing Design Coefficient			0.348	
Base Shear Force			8730	
Damping β_{eff}			15.66%	14.22%
Damping Coefficients B_D B_M			1.35	1.32

FIGURE 14-3 HYSTERESIS TO MAXIMUM DISPLACEMENT



14.4 ANALYSIS MODELS

The performance was quantified using time history analysis of a matrix of 18 ETABS models (Figure 14-4). The models included the 9 earthquake records, one mass eccentricity location (+5% X and Y) and earthquakes applied along the X and Y axes of the structure respectively. As the structure is doubly symmetric only the positive eccentricity case was evaluated. Spring properties for the isolators were as listed in Figure 14-5.

FIGURE 14-4 : ETABS MODEL

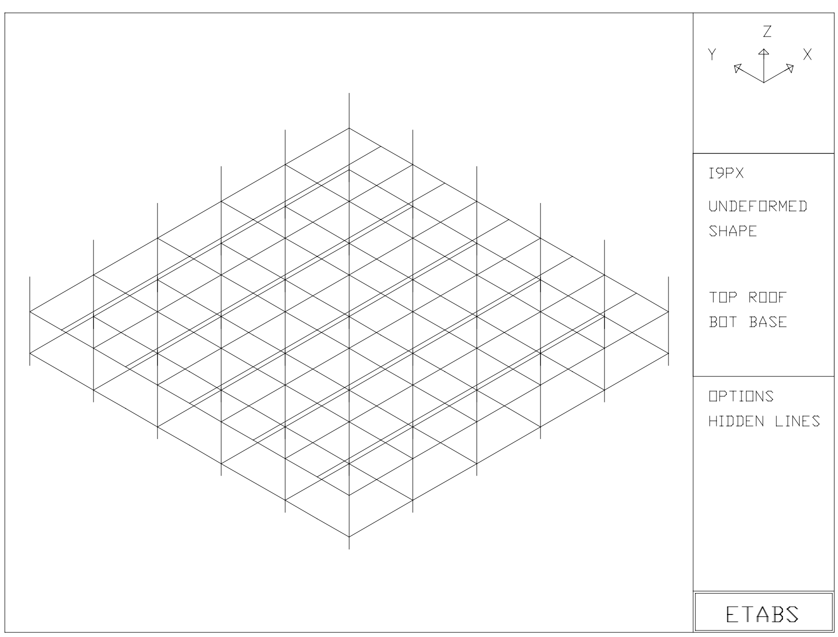


FIGURE 14-5 ETABS PROPERTIES

	Type 1		
	LRB		
First Data Line:			
ID	1		Identification Number
ITYPE	ISOLATOR1		Biaxial Hysteretic/Linear/Friction
KE2	1.28		Spring Effective Stiffness along Axis 2
KE3	1.28		Spring Effective Stiffness along Axis 3
DE2	0.092		Spring Effective Damping Ratio along Axis 2
DE3	0.092		Spring Effective Damping Ratio along Axis 3
Second Data Line:			
K1	895.7		Spring Stiffness along Axis 1 (Axial)
K2	9.55		Initial Spring Stiffness along Axis 2
K3	9.55		Initial Spring Stiffness along Axis 3
FY2/K11/CFF2	118.39		Yield Force Along Axis 2
FY3/K22/CFF3	118.39		Yield Force Along Axis 3
RK2/K33/CFS2	0.103		Post-Yield stiffness ratio along Axis 2
RK3/CFS3	0.103		Post-Yield stiffness ratio along Axis 3
A2			Coefficient controlling friction Axis 2
A3			Coefficient controlling friction Axis 3
R2			Radius of Contact 2 direction
R3			Radius of Contact 3 direction

14.5 ANALYSIS RESULTS

Figures 14-6 to 14-8 show histograms of maximum response quantities for each of the scaled earthquakes used to evaluate performance (refer to Table 14-1 for earthquake names and scale factors).

- The maximum response to DBE motions is dominated by EQ3, which is the Hollister Sth and Pine Drive 0 degree record from the 1989 Loma Prieta earthquake, scaled by 1.69. This record produced maximum displacements about 20% higher than the next highest records, the 1940 El Centro NW record scaled by 3.90 and the 1976 Gazli record scaled by 1.17.

- A similar dominant record appears for MCE response, the El Centro Array No. 7 230 degree component from the 1979 Imperial Valley earthquake, which produced results higher than the other two records for all response quantities.

Figure 14-9 shows the input acceleration record for the dominant DBE earthquake, EQ3. This record has peak ground accelerations of approximately 0.6g. The trace is distinguished by a large amplitude cycle at approximately 8 seconds, a characteristic of records measured close to the fault.

The maximum bearing displacement trace, Figure 14-10, shows a one and one-half cycle high amplitude displacement pulse between 8 and 10 seconds, with amplitudes exceeding 300 mm. The remainder of the record produces displacements not exceeding 150 mm. This is typical of the response of isolation systems to near fault records.

Figure 14-11 plots the time history of story shear forces. The bi-linear model used to represent the isolators has calculated periods of 0.54 seconds (elastic) and 1.69 seconds (yielded). The fixed base building has a period of 0.42 seconds. The periodicity of response would be expected to reflect these dynamic characteristics and Figure 14-11 does show shorter period response imposed on the longer period of the isolation system.

FIGURE 14-6 : TOTAL DESIGN DISPLACEMENT

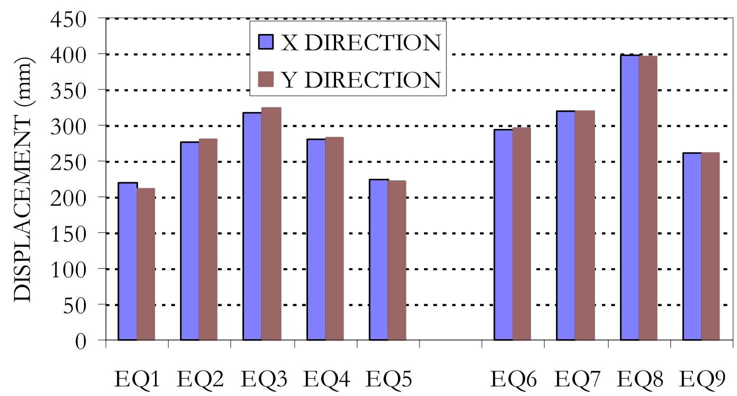


FIGURE 14-7 : BASE SHEAR COEFFICIENT

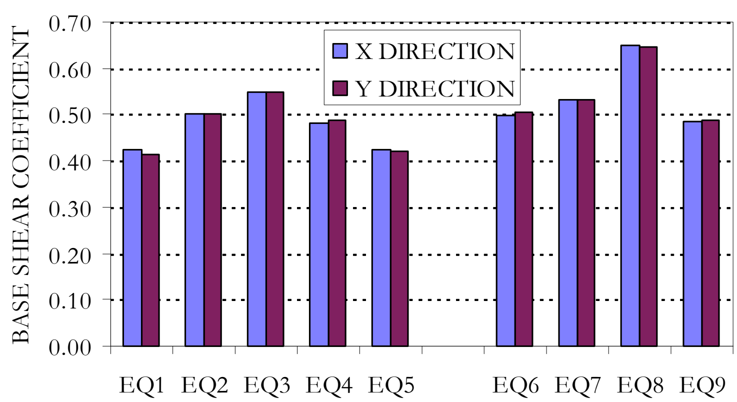


FIGURE 14-8 : MAXIMUM DRIFT RATIOS

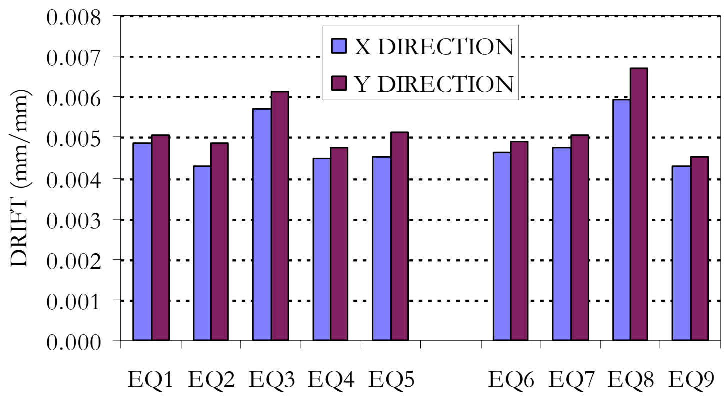


FIGURE 14-9: DBE EARTHQUAKE 3 INPUT

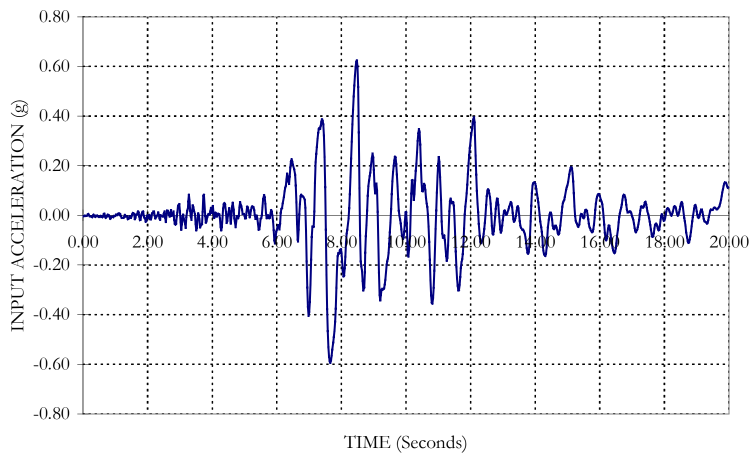


FIGURE 14-10 : DBE EARTHQUAKE 3 : BEARING DISPLACEMENT

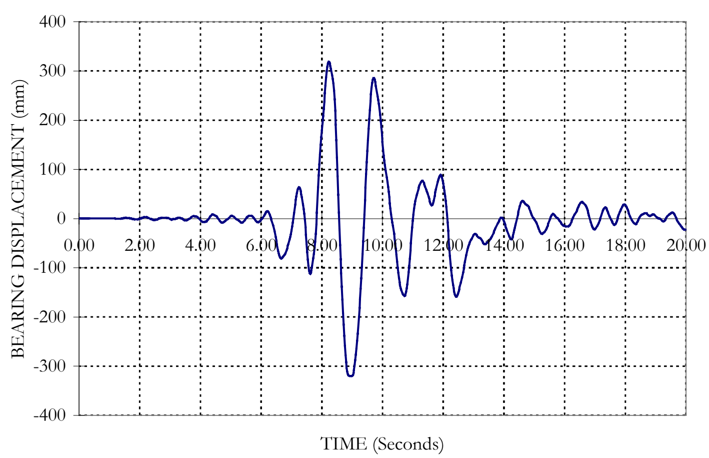
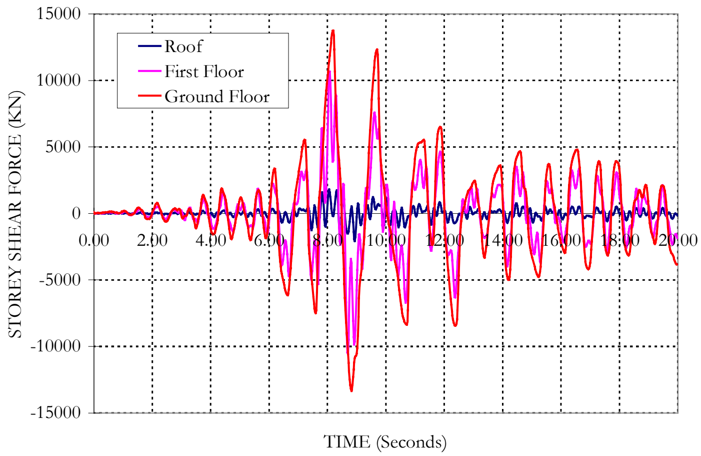


FIGURE 14-11 : DBE EARTHQUAKE 3 : STORY SHEAR FORCES



14.5.1 SUMMARY OF RESULTS

Table 14-2 lists the maximum values of the critical response parameters identified in the specifications:

- Maximum bearing displacements of 323 mm (DBE) and 397 mm (MCE) were within the specification limits of 350 mm and 400 mm respectively.
- A maximum bearing force of 423 KN at DBE levels, less than the maximum specified value of 450 KN.
- A maximum drift ratio of 0.0067 (MCE), within the specification limits of 0.0100.

TABLE 14-2 : SUMMARY OF RESULTS

	DBE	LIMIT	MCE	LIMIT
Base Shear Coefficient	0.549		0.650	
Center of Mass Displacement	290		364	
Maximum Bearing Displacement	323	350	397	400
Maximum Bearing Force	423	450	497	
Maximum Drift	0.0061		0.0067	0.0100

14.6 Test Conditions

The results of the ETABS analysis were used to derive the prototype and production test conditions, as listed in Table 14-3. The displacements and vertical loads define the test conditions. The shear force and the hysteresis loop area define the performance required of the prototype and production tests.

Table 14-3 Prototype Test Conditions

Test Parameter	**Prototype Test**	**Production Test**
Total Design Displacement (mm)	323	260
Total Maximum Displacement (mm)	397	
Average DL (KN)	950	
Average 1.2D + 0.5LL + E (KN)	795	
Average 0.8D - E (KN)	369	
Maximum 1.2D + LL + E (KN)	1224	1100
Shear Force (KN)	424	375.4
Hysteresis Loop Area (KN-mm)	131,925	117,520

The acceptance criteria for prototype and production tests are set out in codes such as the UBC and AASHTO. Some obvious criteria relate to the requirements that the bearings remain stable and that there be no signs of damage in the test. Other UBC requirements relate to the change in properties over multiple cycles. However, the UBC does not provide guidance as to the comparison between test properties and design properties. AASHTO provides some guidance in this respect, requiring that the effective stiffness be within 10% of the design value and that the hysteresis loop area be at least 70% of the design value for the lowest cycle.

For building projects, specifications generally require that the average effective stiffness (or shear force, which is proportional) be within 10% of the design value and that the average loop area be at least 80% of the design value (no upper limit). These were the limits for this project.

14.7 Production Test Results

A total of 38 isolation bearings were tested under combined compression and shear to the center of mass DBE displacement of 260 mm. The isolators were tested in pairs and so 19 individual results were obtained.

Table 14-4 provides a summary of the production tests results. The measured shear forces ranged from –2.5% to +3.2% of the design value, with a mean value –0.8% lower than the design value. The measured hysteresis loop areas exceeded the design value by at least 17.8%, with a mean value 22% higher than the design value.

TABLE 14-4 SUMMARY OF 3 CYCLE PRODUCTION TEST RESULTS

	Shear Force (KN)			Loop Area (KN-mm)		
	Measured	Deviation From Specification	Deviation From Mean	Measured	Deviation From Specification	Deviation From Mean
Maximum	387.4	3.2%	4.0%	148,757	26.5%	3.7%
Mean	372.5	-0.8%	0.0%	143,425	22.0%	0.0%
Minimum	365.9	-2.5%	-1.8%	138,456	17.8%	-3.5%

Figure 14-12 provides an example of one production test of a pair of isolation bearings. As is typical of LRBs, the first loading cycles produces a much higher shear force than subsequent unloading and loading cycles. Although the reasons for this are not fully understood, the increase is believed to be a function of both lead core characteristics and the effect of the initial loading pulse from zero displacement and velocity.

For most LRB tests it is common to test for one cycle additional to the test requirements and exclude the first cycle from the evaluation of results. This is shown in Figure 14-12. The production tests specify 3 cycles at a 260 mm displacement and the test is performed for 4 cycles with the results processed from Cycles 2 to 4.

14.8 SUMMARY

In terms of suitability for isolation this building would be termed marginal in terms of site suitability (soft soils) and building suitability (a light building with relatively low total seismic mass). However, in terms of need for isolation it scored highly as the site-specific earthquake records indicated a maximum elastic coefficient of over 2.5g at the fixed base period.

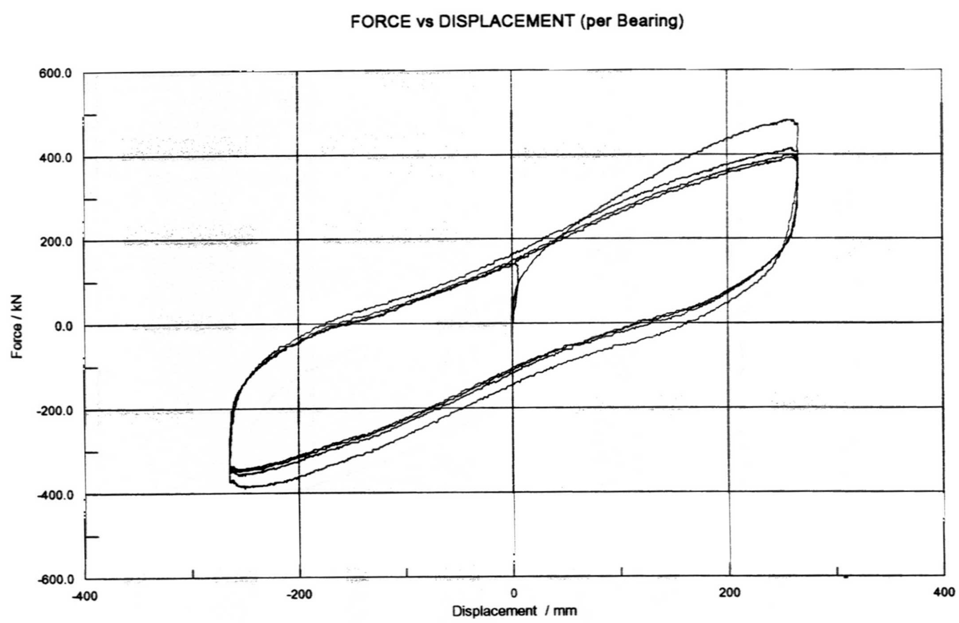
The low mass necessitated a short isolated period (approximately 1.5 seconds) and a high yield level to provide damping to control displacements due to the soft soil (yield level equal to 15% of the weight). This resulted in a high base shear coefficient at the MCE level of 0.65g. However, in spite of these restrictions on isolation system performance the force levels in the structure were still reduced by a factor of almost 4 (from 2.5g to 0.65g) and floor motions would be reduced proportionately.

Standard design office software (ETABS) was used to evaluate the performance of the structure and isolation system using the non-linear time history method of analysis. The results from this analysis provided the displacements, vertical loads and required performance characteristics to define the prototype and production test requirements.

The prototype and production test results demonstrated that the design performance could be achieved. In this example, the production test results produced a mean shear force within 1% of the design values and hysteresis loop areas exceeding the design values by at least 15%.

FIGURE 14-12 EXAMPLE PRODUCTION TEST

Loop	Min Load KN	Max Load KN	Min Disp mm	Max Disp mm	Loop Area KN.mm	Strain %	Keff KN	Qd KN	Damping %
1	-390.6	480.5	-264.7	266.2	173081	126%	1.641	154.3	23.8%
2	-359.4	416.0	-264.7	265.7	149232	126%	1.462	132.8	23.1%
3	-349.6	400.4	-264.7	265.7	140849	126%	1.414	125.0	22.5%
4	-345.7	392.6	-264.7	265.7	136797	126%	1.392	121.1	22.2%
Avg	-351.6	403.0	-264.7	265.7	142293	126%	1.423	126.3	22.6%
	377.3		265.2						



15 BRIDGE EXAMPLE

15.1 Example Bridge

This example is the design of lead-rubber isolators for a 4 span continuous bridge. The bridge configuration was a superstructure of 4 steel plate girders with a concrete deck. The substructure was hollow concrete pier walls (see Figures 15-1 and 15-2).

Two isolation options, both using lead-rubber bearings, were designed:

1. Full base isolation (period approximately 2 seconds)

2. Energy dissipation (period approximately 1 second).

For the base isolation design the aim was to minimize forces and distribute them approximately uniformly over all abutments and piers. The objective of the energy dissipation design was to minimize displacements and attempt to resist more of the earthquake forces at the abutments than at the piers.

The response predicted by the design procedure was checked using nonlinear analysis. The designs were based on an AASHTO design spectrum with an acceleration level, A, of 0.32 and a soil factor, S, of 1.5.

15.2 Design of Isolators

15.2.1 Base Isolation Design

The base isolation design used 500 mm square lead rubber bearings with 100 mm diameter lead cores at each abutment. At the piers, the size was increased to 600 mm and the lead core to 110 mm diameter. All bearings had 19 rubber layers each 10 mm thick, providing a total bearing height of 324 mm, slightly above one-half the plan dimension. These isolators met all AASHTO requirements for lead-rubber bearings.

For seismic loads, this isolation system provided an effective period of 1.94 seconds (excluding the effects of bent flexibility) and equivalent viscous damping of 16%. The design spectrum gives a displacement of 166 mm and a force coefficient of 0.18 for this period and damping.

FIGURE 15-1 : LONGITUDINAL SECTION OF BRIDGE

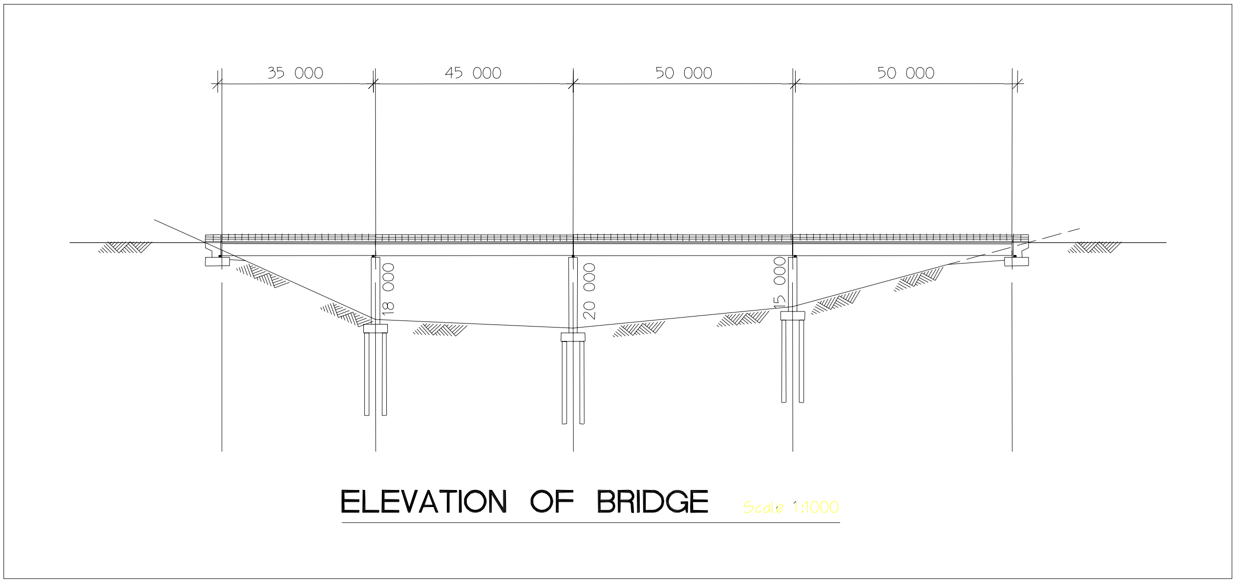
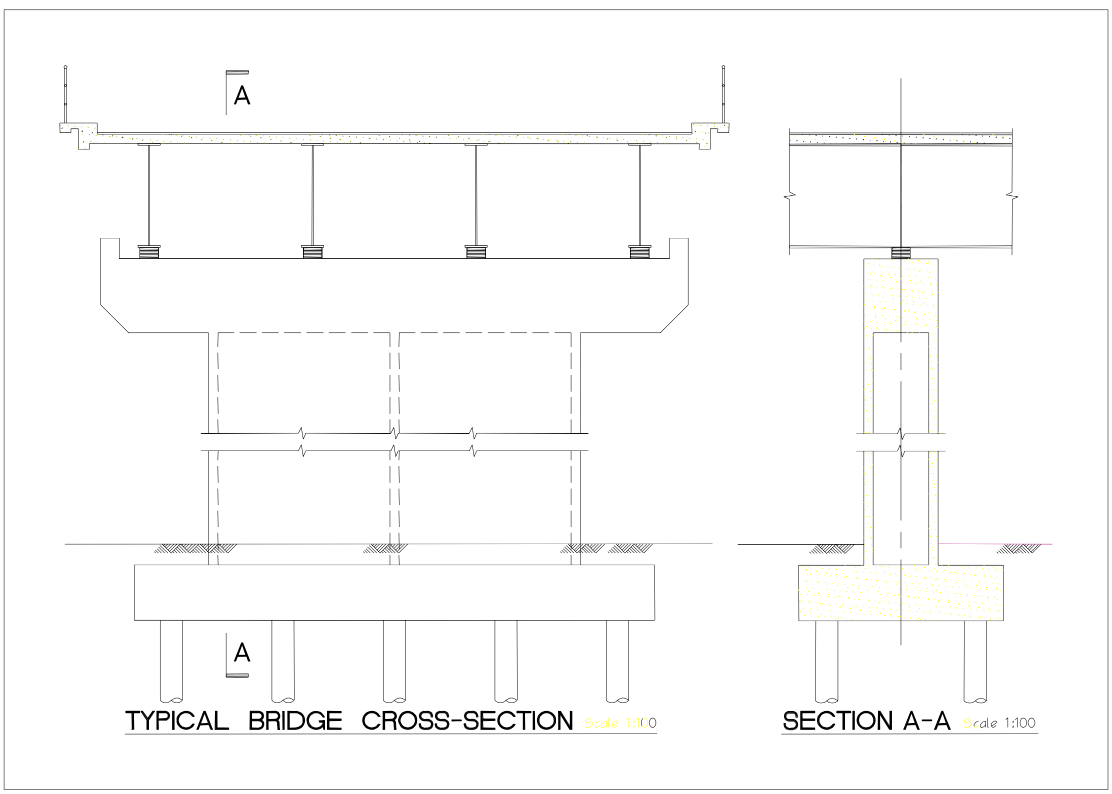


FIGURE 15-2: TRANSVERSE SECTION OF BRIDGE



15.2.2 Energy Dissipation Design

For the energy dissipation design the abutment bearings were increased in size to 750 mm square lead rubber bearings with 250 mm diameter lead cores. At the piers, the size was 600 mm square and the lead cores were 100 mm diameter. The abutment bearings had 8 rubber layers 10 mm thick, providing a total bearing height of 181 mm, about one-quarter the plan dimension. The pier bearings had 12 layers and a total height of 233 mm.

As for the base isolation design, the bearings met all AASHTO requirements.

For seismic loads, the energy dissipation system provided an effective period of 0.86 seconds (excluding effects of bent flexibility) and equivalent viscous damping of 22%. The design spectrum gives a displacement of 67 mm and a force coefficient of 0.36 for this period and damping.

The energy dissipation design produces displacements about one-half that of the base isolation design but the force coefficient is approximately twice as high. This is a result of lesser benefits from the period shift effect.

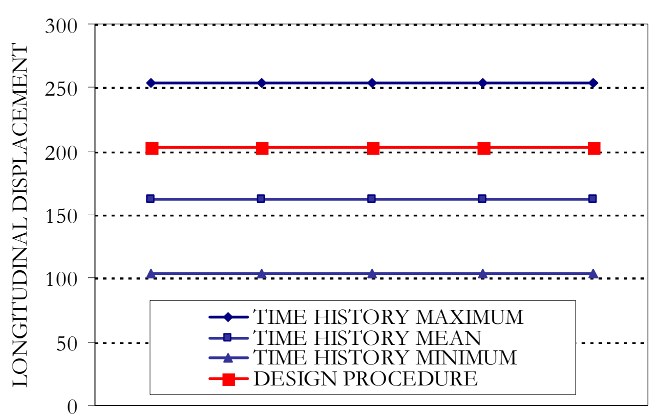
15.3 Evaluation of Performance

15.3.1 Analysis Procedure

AASHTO permits the use of either the single mode method of analysis or a time history analysis. The single mode method can be incorporated through spreadsheet calculations and this is incorporated in the BRIDGE workbook. This method includes the effect of bent flexibility but does not account for flexure of the bridge deck or the self-weight of the bents.

The time history method can be based on either the maximum response from three time histories or the mean results of 7 time histories. The procedure included in BRIDGE is the latter, the mean of 7 time histories.

Figure 15-3 Longitudinal Displacements

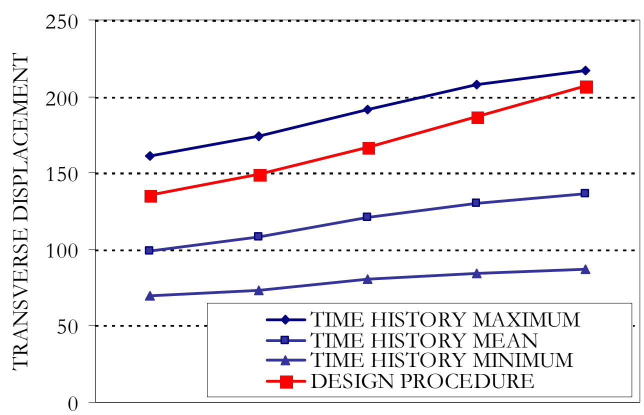


Figures 15-3 and 15-4 compare the results from the single mode method (termed Design Procedure) with the minimum, maximum and mean results from the 7 time histories.

1. The bent displacements are equal at each location as the deck does not deform axially. Figure 15-3 shows that the design procedure estimate of displacements lies approximately midway between the mean and the maximum time history values.

2. The transverse response of the bridge includes a rotational component because of the unequal height of the piers and non-symmetrical span lengths, as shown in Figure 15-4. The time history results also show a slight effect of flexure of the deck between abutments. As for longitudinal displacements, the transverse displacements estimated from the design procedure are between the mean and maximum values from the time history analysis. The design procedure also shows slightly more rotation than the time history results.

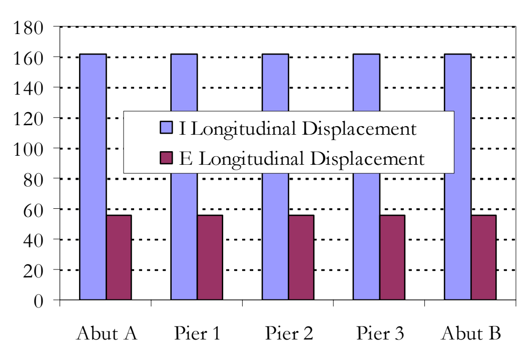
FIGURE 15-4 TRANSVERSE DISPLACEMENTS



These results show that the simplified design procedure, including bent flexibility, provides a good approximation to the more accurate response calculated from the time history. The design procedure results are higher than the mean from the 7 time history results but lower than the maximum results.

These results imply that, depending on whether the *mean of 7* or *maximum of 3* time history method were chosen, the design procedure would be either slightly conservative or slightly non-conservative. Given the uncertainties implicit in the selection of the time histories, this suggests that an isolation system based on the simplified design procedure would provide a system with satisfactory performance.

FIGURE 15-5 LONGITUDINAL DISPLACEMENTS



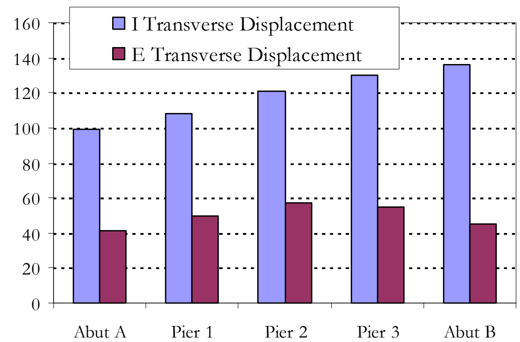
15.3.2 EFFECT OF ISOLATION ON DISPLACEMENTS

Figures 15-5 and 15-6 compare respectively the longitudinal and transverse deck displacements for the isolation system (I) and the energy dissipation system (E).

In the longitudinal direction the displacements at deck level are enforced to be equal by the axial stiffness of the deck. The base isolation displacements of 160 mm are approximately 2½ times as high as the maximum energy dissipation displacements of 60 mm.

In the transverse direction the large lead cores at the abutments restrain rotation and there is some deformation due to flexure of the deck. The maximum isolated displacement is 136 mm at Abutment B whereas with the energy dissipation bearings the maximum displacement is 57 mm. The isolated displacements are higher by about 2½ times, the same factor as for the longitudinal displacements

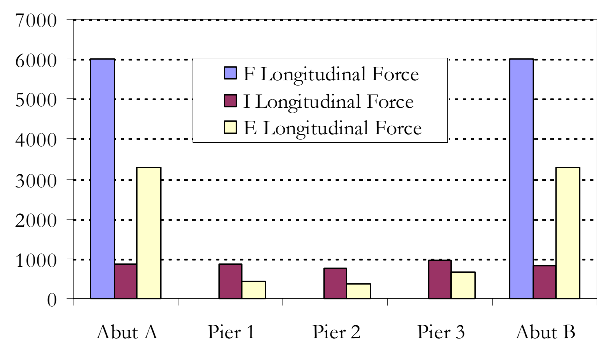
FIGURE 15-6 TRANSVERSE DISPLACEMENTS



15.3.3 EFFECT OF ISOLATION ON FORCES

Figures 15-7 and 15-8 compare the longitudinal and transverse bearing forces respectively for the fixed bearings (F), the isolation bearings (I) and the energy dissipation bearings (E). The isolation and energy dissipation bearings have a marked effect on both the magnitude and the distribution of the forces.

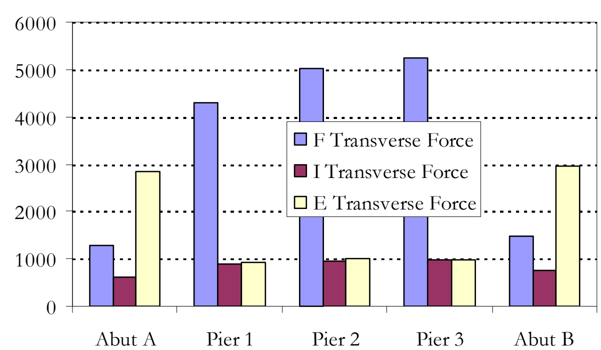
FIGURE 15-7 LONGITUDINAL FORCES



<u>Longitudinal</u>

- In the longitudinal direction, when the bearings are fixed the total seismic force is resisted equally at the two abutments with a maximum force of 6,000 KN at each location.

FIGURE 15-8 TRANSVERSE FORCES



- When isolation bearings are used the loads are distributed approximately equally over the abutments and piers with maximum forces ranging from 769 KN to 975 KN, a reduction by a factor of 6 compared with the fixed bearing configuration.

- The energy dissipation bearings produce a force distribution where most force is resisted at the abutments (maximum 3284 KN) and a much lower force at the piers (maximum 652 KN). The forces are higher than for the isolated bridge but still reduce the fixed bearing abutment forces by a factor of 2.

Transverse

- In the transverse direction, the fixed bearings distribute seismic forces to the piers in approximate proportion to their stiffness as shown in Figure 15-8. The maximum abutment force is 1479 KN and the maximum pier force is 5,243 KN at Pier 3, the shortest pier.

- With isolation bearings, the forces are distributed relatively uniformly between all abutments and piers. The maximum abutment force is 741 KN, the maximum pier force is 987 KN. As for the longitudinal direction, peak forces are reduced by a factor of over 5.

- The energy dissipation bearings produce a force distribution which differs from both the fixed bearings and isolation bearings. Most force is resisted at the abutments, which have a maximum force of 2952 KN and a small proportion at the piers, where the maximum force of 1012 KN is similar to the maximum for the isolation bearings.

15.4 Summary

This bridge example has illustrated how bearings can be used in bridge structures both to reduce seismic forces on the bridge and to alter the distribution of these forces to the different substructure elements.

In this example, a typical seismic isolation design is based on a 2 second isolated period and similar isolators at all abutments and piers. This reduces total seismic forces by a factor of 2.8 longitudinally and 4 transversely. Because the forces are approximately equally distributed, the isolation bearings reduce local forces by a greater factor. The longitudinal abutment forces are reduced by a factor of 7, the transverse pier forces are reduced by a factor of 5. The force reductions were associated with maximum deck displacements of 160 mm.

The energy dissipation design used stiffer bearings with large lead cores at the abutments to reduce the isolated period to 1 second and concentrate forces in the abutments. This reduced the displacements compared to the isolated design, with deck displacements of 60 mm, 35% of the isolated displacements. As the period shift was less, the seismic force reductions compared to the fixed bearing design were smaller, 1.5 in the longitudinal direction and 2.0 in the transverse direction. This design reduced longitudinal abutment forces by a factor of 2 and the transverse pier forces by a factor of 5.

This example provides two examples of achieving different objectives using lead rubber bearings and there are numerous other possible permutations. The design spreadsheet is set up to enable rapid evaluation of alternative isolator configurations.

16 INDUSTRIAL EQUIPMENT EXAMPLE

16.1 SCOPE OF THIS EXAMPLE

Heavy, squat industrial equipment can be isolated using the procedures described earlier for buildings and bridges. However, much industrial equipment is often defined by characteristics of low mass and a high center of gravity. These properties make it difficult to achieve a long period of isolation and may cause problems with uplift at the isolator locations.

This example is for a boiler and economizer, two equipment items at a refinery complex. Each are characterized by low mass and high center of gravity and they illustrate the difficulties in achieving effective isolation for these conditions.

Design was in accordance with the 1997 Uniform Building Code (UBC) for Soil Category S_D, Seismic Zone 3. The support layout for the boiler provided 27 bearings and a total seismic weight of 2372 KN. The economizer had 12 supports and a total seismic weight of 520 KN.

16.2 ISOLATOR DESIGN

All isolators were designed as natural rubber elastomeric bearings using a natural rubber with a nominal shear modulus of 0.4 MPa, the softest practical natural rubber formulation. Circular isolators were used as these are considered the most suitable shape for omni-directional earthquake attack.

Plain elastomeric isolators were selected as there were no requirements to resist non-seismic lateral loads, this type tends to be the most efficient in minimizing accelerations up the height of the structure and the analysis of the isolated structure is simplest for a linear elastic system.

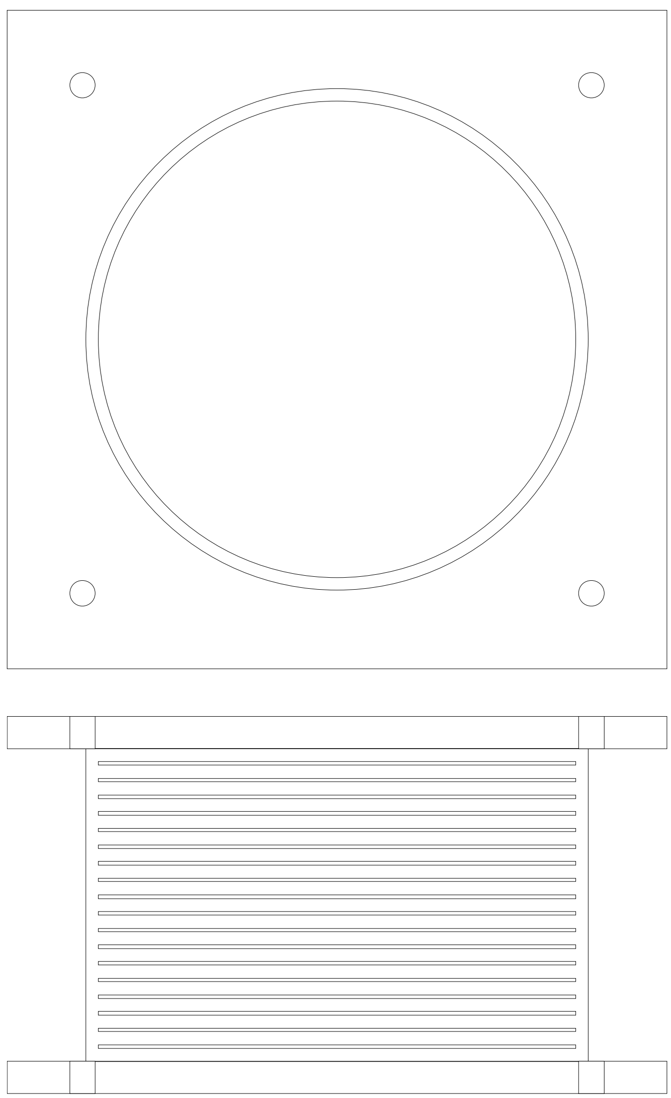
A single isolator was used for all pads in both the Boiler and the Economizer. The plan size and maximum height were governed by the buckling load capacity of Pad Type B under the Boiler. Dimensions are listed in Table 16-1.

TABLE 16-1 : ISOLATOR DIMENSIONS (MM)

Plan Dimension	400
Layer Thickness	10
Number of Layers	18
Shape	CIRCULAR
Side Cover	10
Internal Shim Thickness	2.7
Load Plate Thickness	25.4
Load Plate Dimension	525
Load Plate Shape (S or C)	SQUARE
Connection Bolts	4M20
Total Height	276
Weight (kg)	177

Although the buckling load was critical for the Type B pad, it would probably not be economical to reduce bearing sizes at other locations. The Type A and C pads for the Boiler need to be at least 380 mm diameter and the Economizer pads at least 370 mm. The savings for this small reduction in plan size would not justify the complications and cost of additional prototype bearings and tests.

FIGURE 16-1: ISOLATOR CONSTRUCTION



16.3 Seismic Performance

A summary of the seismic performance is as provided in Table 16-2. The degree of isolation is the maximum that could be obtained within the material and dimensional constraints.

TABLE 16-2 : SEISMIC PERFORMANCE (UNITS MM)

	Boiler	**Economizer**
Effective Period T_D	1.170	1.131
Displacement D_D	157	152
Total Displacements D_{TD}	186	179
Force Coefficient V_b / W	0.462	0.477
Force Coefficient V_s / W	0.231	0.239
Base Shear Force (KN)	547	236
Damping β_{eff}	5.00%	5.00%
Damping Coefficients B_D	1.00	1.00

The design assumed equivalent viscous damping of 5% in the bearings. Natural rubber bearings tested according to the UBC requirements for prototypes typically produce a damping ratio of 2% to 5%, depending on vertical load and strain.

The prototype tests are designed to measure only the hysteretic damping component of the total damping. Most test machines cannot operate at the required frequency to measure the viscous damping component. However, most design codes assume 5% viscous damping for all structural components and so on this basis an assumption of 5% equivalent damping is justified.

16.4 Alternate Isolation Systems

The system as designed assumed natural rubber bearings with 5% equivalent viscous damping. Most isolation systems use supplemental damping. Two alternatives were considered, one using high damping rubber (HDR) and the other replacing the Type A pads with lead rubber bearings (LRB) with 50 mm lead cores.

Tables 16-3 and 16-4 list the performance with these alternate systems. The increased damping reduces both the maximum displacements (by up to 40%) and the maximum forces (by up to 18%).

The use of higher damped systems generally requires more complex analysis than a linear elastic system. For the lead rubber bearing alternate, the use of two different bearing types would require additional bearings for prototype testing.

Both the HDR and LRB are likely to be more expensive than the elastomeric bearing option. The engineering costs are also likely to be higher. Therefore, these systems would be used only if there were cost savings due to the lower displacements and base shear forces.

TABLE 16-3 : ALTERNATE SYSTEMS FOR BOILER

	Elastomeric	High Damping Rubber	Lead Rubber
Effective Period T_D	1.170	1.020	0.989
Displacement D_D	157	118	92
Total Displacements D_{TD}	186	140	109
Force Coefficient V_b / W	0.462	0.457	0.379
Force Coefficient V_s / W	0.231	0.228	0.190
Base Shear Force (KN)	547	541	450
Damping β_{eff}	5.00%	9.38%	18.39%
Damping Coefficients B_D	1.00	1.16	1.44

TABLE 16-4 : ALTERNATE SYSTEMS FOR ECONOMIZER

	Elastomeric	High Damping Rubber	Lead Rubber
Effective Period T_D	1.131	0.976	0.951
Displacement D_D	152	113	89
Total Displacements D_{TD}	179	134	105
Force Coefficient V_b / W	0.477	0.477	0.394
Force Coefficient V_s / W	0.239	0.238	0.197
Base Shear Force (KN)	236	236	195
Damping β_{eff}	5.00%	9.50%	18.79%
Damping Coefficients B_D	1.00	1.16	1.44

16.5 SUMMARY

This example of the isolation of low mass equipment has illustrated that the benefits of isolation are restricted by the low mass. The elastic design coefficient for these items of equipment with a fixed base is 0.90. An elastic isolation system can reduce this by a factor of approximately 2 (to 0.48) and a yielding LRB system can reduce it by a factor of 2.3 (to 0.39). Although these force reductions are not large, they may have an impact of design for fragile items of equipment.

For enclosed equipment with no non-seismic lateral loading is may be most cost effective to use a linear elastic system with no supplemental damping. These bearings are usually cheaper than LRB and HDR bearings and do not require a time history analysis. They will have larger displacements and shear coefficients than the other types and so the cost / benefits of the different types of system should be assessed on a project specific basis.

17 PERFORMANCE IN REAL EARTHQUAKES

The best way to finish up these guidelines is to examine the actual performance of isolation systems in real earthquakes and make sure that we learn from these experiences.

One of the most frequent questions asked by potential users of isolation systems is *Has it been proved to work in actual earthquakes*? The short answer is a qualified no; no isolated building has yet been through "The Big One" and so the concept has not been tested to the limit but some have been subjected to earthquakes large enough to activate the system.

Our buildings in New Zealand have not been subjected to any earthquakes yet, although four are located in Wellington so it is only a matter of time until this happens according to most seismologists. There were two bridges on lead-rubber isolation systems subjected to strong motions during the 1987 Edgecumbe earthquake. One, Te Teko bridge, had an abutment bearing roll out because the keeper plates were misplaced. This caused minor damage. This type of connection is no longer used. The other bridge was on a privately owned forestry road and, other than a report that there was no damage, there is no information on this.

Table 17-1 summarizes the reported performance of structures world-wide during earthquakes. Many of the structures are not instrumented and so much evidence of performance is either indirect or anecdotal.

There are features of observed response that can teach us lessons as we implement base isolation:

- Some structures performed well and demonstrated the reductions in response that base isolation is intended to achieve. The most successful is probably the USC Hospital in Los Angeles. Occupants reported gentle shaking during the main shock and after shocks of the 1994 Northridge earthquake. The pharmacist reported minimal to no damage to contents of shelves and cabinets. Other successful installations were the Tohuku Electric Power building in Japan, the Stanford Linear Accelerator and Eel River Bridge in California and several bridges in Iceland.

- At the LA County Fire Command Center, the contractor had poured a reinforced concrete slab under the floor tiles at the main entrance to the buildings, preventing free movement in the E-W direction. Apparently the reinforcing was added after the contractor had replaced the tiles several times after minor earthquakes and did not realize that this separation was designed to occur. This emphasizes the importance of ensuring that building operational procedures are in place for isolated buildings.

- The Seal Beach and Foothills buildings demonstrate that accelerations will be amplified as for a fixed base building for accelerations which do not reach the trigger point for the system.

- The West Los Angeles residence used an owner-installed system of steel coil springs and dashpots without a complete and adequate plane of isolation. The springs allowed vertical movement and the building apparently responded in a pitching mode. The owner was satisfied with the performance.

- The Matsumura Gumi Laboratory building in Japan did not amplify accelerations as for a fixed base building but also did not attenuate the motions as expected. The period of response was shorter than expected and a possible reason for this was the temperature of the isolators, estimated at 0°C in the unheated crawl space. Potential stiffening of rubber as temperatures are reduced needs to be accounted for in design if isolators are in locations where low temperatures may occur.

- Bridges in Taiwan and Kobe were partially isolated, a design strategy often used for bridges where the system provides energy dissipation but not significant period shift. The response of these bridges shows benefits in the isolated direction compared to the non-isolated direction but the reductions are not as great as for fully isolated structures.

- The dissipators at the Bolu Viaduct in Turkey were severely damaged due to near fault effects when the displacement caused impact at the perimeter of the dissipator. This appeared to be mainly due to large displacement pulses near the fault but may have been accentuated by use of an elastic-perfectly plastic system rather than the more common strain hardening system.

In all structural engineering, we need to learn from the lessons which earthquakes teach us. There is discussion throughout these guidelines on aspects of isolation that can degrade performance if not properly accounted for. These earthquakes have shown the importance of attending to all these details.

TABLE 17-1 EARTHQUAKE PERFORMANCE OF ISOLATED BUILDINGS

			Acceleration		
Structure	**System Type**	**EQ**	**Free Field(g)**	**Structure (g)**	**Comments**
USC Hospital	LRB	1994 Northridge	0.49	0.21	Movement estimated at up to 45 mm. No damage Continued Operation
LA County Fire Command Center	HDR	1994 Northridge	0.22 E-W 0.18 N-S	0.35 E-W 0.09 N-S	Minor ceiling damage. Continued Operation
Seal Beach Office	LRB	1994 Northridge	0.08	0.15	No damage. System not activated.
Foothills Law & Justice Center	HDR	1994 Northridge	0.05	0.10	No damage. System not activated.

			Acceleration		
Structure	**System Type**	**EQ**	**Free Field(g)**	**Structure (g)**	**Comments**
West Los Angeles Residence	Springs	1994 Northridge	0.44	0.63	No structural damage. Some damage at movement joint. Unusual isolation system.
Tohuku Electric Power	LRB Steel Dampers	1995 Kobe	0.31	0.11	No damage. Movement estimated at 120 mm.
Matsumura Gumi Laboratory	HDR	1995 Kobe	0.28	0.27	Isolators at 0°C, may Have stiffened
Stanford Linear Accelerator	LRB	1989 Loma Prieta	0.29	0.14	Not instrumented, estimated response. Movement estimated at 100 mm
Bai-Ho Bridge	LRB PTFE	1999 Taiwan	0.17 L 0.18 T	0.18 L 0.26 T	Longitudinal isolation only.
Matsunohama Bridge	LRB	1995 Kobe	0.15 L 0.14 T	0.20 L 0.36 T	Longitudinal isolation only.
Bolu Viaduct	PTFE Crescent moon Energy dissipator	1999 Turkey	1.0 +		Displacements exceeded device limit of 500 mm, causing damage
Eel River Bridge	LRB	1992 Cape Mendocino	0.55 L 0.39 T	Not measured	Estimated acceleration. Movement estimated at 200 mm L and 100 mm T. Minor joint spalling.
Four bridges in Iceland	LRB	M6.6 and M6.5 in June 2000.	0.84	Not measured	No damage.
Te Teko Bridge (NZ)	LRB	1987 Edgecumbe	0.33		Estimated acceleration. Estimated displacement 100 mm. Abutment bearing dislodged, caused minor damage.

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