Application of Seismic Isolation as a Performance-Based Earthquake-Resistant Design Method

by

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Abstract

Ground motion due to an earthquake excitation often induces disastrous disturbances that severely affect structures and their contents. Conventional earthquake-resistant design focuses on the strengthening of structures to safely resist these disturbances and avoid structural collapse, while little attention is given to the prevention of damage. Using this design approach, it is almost impossible to construct completely ‘earthquake-proof’ structures of reasonable cost and aesthetically acceptable.

Seismic isolation, an innovative performance-based design approach may alternatively be used to minimize earthquake induced loads and resulting damage in low- to medium-rise buildings. The thesis explores this design approach. The necessity of the method, given the limitations of the conventional earthquake-resistant design is first discussed, followed by a detailed description of its characteristics, advantages and limitations. The capability of a seismic isolation system to decouple a structure from the damaging effects of ground acceleration, and thus reduce seismic forces on the structure is demonstrated. Currently used isolation systems and their components are described in detail. The advantage of using a hybrid system is emphasized. The performance of seismically isolated structures is evaluated, and examples of such structures and their response under an earthquake are presented. Finally, a preliminary design methodology based on seismic isolation is proposed. It involves several design stages, including the design of the isolation system and that of the superstructure, and the estimation of critical parameters. The latter will be used in the final analysis to determine the most appropriate design. The potential of the method and its establishment as a powerful earthquake-resistant design approach is also assessed.
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Αφιερώμενο, με όλη μου την αγάπη,
στοὺς γονεῖς μου.
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Chapter 1

Introduction

1.1 Motivation

Earthquakes are natural threats that, depending on their magnitude, can be disastrous. For structures in high seismicity regions, earthquake loading is considered the most significant and possibly the most destructive external load, particularly for low- to medium-rise buildings. In contrast, wind loads usually dominate the design of high-rise buildings, even in highly seismic areas. Today, technological advancement allows us to control the consequences and hazards of severe earthquakes. Unfortunately, advancement in the design and construction technology is partially based on experience gained after the occurrence of severe earthquakes with extensive damage and loss of human lives. In the dominant conventional earthquake-resistant design approach, safety measures are emphasized while measures to reduce the resulting damage are given less attention. Although the methods of analysis, design and construction of earthquake-resistant structures have been substantially improved in the last decade, severe earthquakes still result in undesirable damage, even in the case of structures designed with the most conservative and rigorous seismic codes. High damage levels are, however, no longer acceptable considering the higher performance demand implied by technological advancement, as well as due to the high cost equipment in structures which must be protected. Thus, alternative design and construction methods must be employed. Motivated by the necessity for an innovative design approach to address resistance of structures under earthquake excitations and minimization of damage, the thesis explores seismic isolation.

1.2 Thesis Outline

Seismic isolation is an alternative to conventional design approach to account for earthquake loadings in low- to medium-rise structures. To place the discussion in the appropriate context and eventually highlight the potential of seismic isolation, the current conventional earthquake-resistant design philosophy, its disadvantages and limitations are first discussed in Chapter 2. The necessity for an alternative design approach to address the issue of damage is also emphasized.
Seismic isolation is discussed in detail in Chapter 3. First, the fundamental features and goal of seismic isolation, namely to decouple the structure from the damaging effects of ground accelerations, are presented. The discussion includes the basic components and advantages of seismic isolation as well as a simple example in which a fixed supported structure and a seismically isolated one are compared, to demonstrate the effectiveness of the method. Constraints and limitations in the practical implementation of seismic isolation are also discussed, followed by an account of the factors which enable its development and use. Some economic issues related to seismic isolation are mentioned, for both new and retrofitted structures. Finally, reasons for the lack of extensive use of seismic isolation are discussed.

Chapter 4 describes the most commonly used seismic isolation devices, elastomeric bearing and sliding systems in particular. The discussion focuses on the former since they are the most commonly used seismic isolation devices. The characteristics of lead rubber bearings and high damping rubber bearings are given as well as those of a hybrid isolation system which incorporates the advantages of both basic schemes, using high damping rubber bearings combined with lead plugs. The potential of the hybrid system for extensive use in seismic isolation applications is emphasized.

The practical implementation of seismic isolation, related practical problems and maintenance and management issues are discussed in Chapter 5. First, a historical overview and selected practical applications of seismic isolation are presented, followed by information on the observed performance of seismically isolated structures. Then, the practical issue of the selection of the appropriate location of the isolation level is discussed, as well as some practical problems which may arise when the system is incorporated in a structure. Finally, pertinent seismic isolation codes and provisions in the United States and Japan are identified and discussed.

Chapter 6 describes the structural characteristics of commonly used isolation systems. First, different mathematical models used in the preliminary design stage and the final analysis are discussed. Then, the stiffness and damping characteristics of both linear and bilinear systems are presented and a comparative study of both systems is performed, to identify the advantages and disadvantages of each system. The alternative use of a hybrid system, such as one composed of high damping rubber lead bearings, is proposed, based
on the results of the above-mentioned study. The linearization of the bilinear behavior of the isolation system and a study on its accuracy are also included. A study to determine the effect of superstructure stiffness on the behavior of the isolated structure is finally performed.

A preliminary design procedure for lead-rubber bearings is proposed in Chapter 7, using results obtained in the previous chapter. The procedure is based on the maximum acceptable displacement which must be accommodated for the useful life-span of the structure. It provides initial values for the dimensions and the configuration of the isolation system which are needed in the final and more detailed analysis. Finally, issues of concern at a more detailed analysis stage are discussed and their importance is emphasized. A summary of the results of the thesis is provided and important observations are discussed in chapter 8, followed by an assessment on the future potential of seismic isolation.
Chapter 2

Conventional Earthquake-Resistant Design and its Limitations

2.1 Objectives and Design Philosophy

In high seismicity areas, it is almost impossible to built a conventionally fixed supported low- to medium-rise building of reasonable cost, which is also “earthquake proof” even for a severe earthquake. Therefore, emphasis is usually given to the limitation of casualties and the avoidance of structural collapse. Damage, which results from inelastic deformations, is the safety valve to dissipate incident energy due to the earthquake and to reduce, when the structure is properly designed, the probability of structural collapse. Although this design approach has saved many lives in technologically advanced countries where it is properly applied, it has still resulted in extensive damage with severe economic consequences.

The purpose of conventional earthquake resistant design, as defined in most current design codes, is to resist minor earthquakes and prevent all damage, resist moderate earthquakes with limited non-structural damage, and prevent structural collapse under a severe earthquake but with the acceptance of both structural and non-structural damage. Structural damage refers to damage of structural members, while non-structural damage refers to damage of non-structural parts of the structures, such as architectural facades, partition walls and ceilings. Properly designed conventional earthquake-resistant structures have the strength and ductility to withstand seismic loads and avoid structural collapse, but allow structural damage.

Seismic loads are due to accelerations of the masses of the structure which, for low- to medium-rise buildings, are amplifications of the ground accelerations. Conventional earthquake-resistant design is based on reduced seismic loads, smaller than those expected if the structure could withstand them without inelastic deformations; otherwise it would be practically impossible to provide the strength required to avoid inelastic deformations within reasonable economic and architectural costs. This reduction is achieved by providing adequate levels of ductility to structural members so that they can experience inelastic deformations without failure under a major earthquake. The ductility capacity is provided
by proper design and construction of the structural members, to yield at particular locations and be capable to withstand several cycles of reversed load without brittle failure. The energy inserted in the structure due to the earthquake excitation is absorbed through plastic deformations resulting in significant and undesirable damage. This energy absorption mechanism is harmful not only to individual structural members but also to adjacent non-structural elements. The absorption of energy is implicitly taken into consideration by performing an elastic analysis and using reduction factors to account for ductility. The seismic capacity of a relatively stiff structure increases when the structure experiences inelastic deformations. This increase is due to the resulting reduction of stiffness which shifts the fundamental frequency of the structure towards the range where seismic effects are less significant.

Most seismic codes are based on design spectra constructed using several earthquake excitations and their corresponding response spectra. A response spectrum is a plot of maximum displacement, velocity or acceleration for many possible single degree of freedom (DOF) systems due to a particular excitation. A design spectrum is usually constructed to envelope the mean value plus a standard deviation of several response spectra. Considering the uncertainties associated with both structural characteristics and the expected earthquake excitation, a design spectrum may be preferable to a response spectrum. Figure 2.1 shows a typical design spectrum. The top curve (3) depicts the force level, according to Structural Engineers Association of California (SEAOC), of a fixed supported structure if the structure was designed to withstand the seismic loads elastically. The lowest curve (1) corresponds to the forces which would be used in the design of a fixed structure according to the Uniform Building Code [UBC, 1991]. Taking into account the factors of safety used in the design, the estimated strength of a fixed-supported structure is given by curve (2). The difference between the maximum elastic force and the probable strength must be absorbed by ductility.
The difference must be absorbed by ductility.

Figure 2.1: Typical design spectra.

2.2 Disadvantages
At the beginning of the discussion it was mentioned that conventional earthquake-resistant design has some disadvantages. The most important ones are briefly discussed here.

2.2.1 Resonance Problem For Low- to Medium-Rise Structures
A serious problem that arises in low- to medium-rise buildings is that their fundamental frequency is in the range of the dominant frequencies of earthquakes. This results in amplification of the ground acceleration causing severe damage to structures and their contents. This disadvantage of conventional earthquake-resistant design may be demonstrated through the construction of a response spectrum.

Through a mode superposition analysis, it can be shown that the response of most structures is mainly characterized by motion in the first few natural eigenmodes. The contribution of the fundamental eigenmode is of great importance since it is, at least in most cases, the dominant portion of the structural dynamic response. Most modern seismic codes provide equivalent static procedures based on the contribution of the fundamental mode.

The response spectra of six earthquake excitations, shown in Figure 2.2, have been constructed using the Newmark \( \alpha-\delta \) method, described in Appendix A. The influence of the fundamental period of a structure on its response is evident from the response spectra. The response spectra of maximum relative displacements and spectral pseudo-accelerations for these excitations, and for damping ratios ranging from 0\% to 20\% are plotted in Figures 2.3 and 2.4, respectively.
Figure 2.2: Earthquake excitations.

Figure 2.3: Maximum relative displacements $S_d$. 
Spectral displacement and accelerations are reduced with increasing damping ratio $\xi$. The relative displacement $S_d$ increases while the spectral acceleration $S_a$ decreases as the natural period increases. Assuming that a typical low-rise building has a fundamental period typically in the range of 0.1 to 0.7 sec, we observe that at its fundamental frequency it is in resonance. To avoid such a situation, the period must be either lengthened, by increasing the flexibility of the structure, or shortened, by increasing the stiffness. An increase in stiffness reduces relative displacements, while an increase in flexibility results in substantial reduction of accelerations.

The effect of the natural period of a structure may be illustrated by considering a mass connected to the ground with an infinitely flexible spring, as shown in Figure 2.5a. In the case of an earthquake excitation, i.e. support motion, the mass will remain essentially in its initial position unable to follow the ground motion. This response will lead to approximately zero acceleration and zero force in the spring. However the relative displacement will equal the ground displacement $u_g(t)$. Consider now a mass connected to the ground by an infinitely rigid spring, as shown in Figure 2.5.b. In this case, the mass will follow the ground motion. This response implies zero relative displacements. However the acceleration will equal the ground acceleration $a_g(t)$, which may be beyond the acceptable limit.
It follows from the single mass example that a very flexible structure will have very large interstory displacements but small accelerations and consequently small shear forces. In contrast, a very stiff structure will have very small interstory displacements but accelerations equal to the ground accelerations and relatively high seismic forces. Therefore, using the conventional earthquake-resistant design approach we cannot reduce both interstory deflections and floor accelerations in order to avoid damage.

2.2.2 Consequences of Conventional Earthquake-Resistant Design Philosophy

In most cases, it is virtually impossible to construct an earthquake-proof structure. Thus, earthquake-resistant design is geared towards human safety and thus the avoidance of structural collapse. The consequences of an earthquake excitation in a conventionally designed structure include structural and non-structural damage, mainly due to interstory displacements and in particular interstory drifts (or story shear strains). These are the relative displacements between two subsequent floors, divided by the story height. Acceptable story shear strains are typically less than 1/500, to avoid non-structural damage. Special design and construction details, such as isolation of the facade walls from the surrounding structural members, may allow higher values for story shear strain. Another consequence is damage of building contents due to the floor accelerations which usually increase with height. In the case of buildings with very sensitive and usually expensive equipment the consequences may be catastrophic. A final but most crucial consequence is the disruption of operation of the damaged structures for repairs. This is disastrous, especially in the case
of hospitals and fire stations which must remain fully operational, particularly after a severe earthquake.

2.3 Need for a Different Design Approach

To minimize or eliminate the above consequences and still use the conventional design approach, construction costs must be significantly increased, the architectural design must be aesthetically affected and several construction methods unsuitable for earthquake resistance must be completely avoided. Even if the structure were to behave elastically under a severe seismic excitation, the resulting stresses would be several times higher than the yield stress of the material, if members of reasonable size are used. This is actually the reason why structural engineers compromise in conventional earthquake-resistant design approach and assume lower seismic loads than those expected. This compromise relies on the ability of the structure, when properly designed and constructed, to provide high levels of ductility and implicitly accepts the above damage.

In the past few years it has been realized that in the case of a severe earthquake the safety of residents is not the only issue of concern. First of all, the conventional earthquake-resistant design approach is expensive due to the high cost of repairs. Second, the loss of functionality of structures which have essential facilities, that must remain operational, and the protection of very sensitive and expensive equipment, are also very important. Finally, human discomfort during a strong earthquake and the resulting psychological effects which are related to the magnitude of floor accelerations, must also be addressed. Thus, a different design approach is needed, to account for the above-mentioned effects of seismic excitations.
Chapter 3

Seismic Isolation: An Alternative Earthquake-Resistant Design Approach

3.1 The Concept

Seismic isolation is an old design idea, proposed to decouple a structure, part of it or even equipment placed in the structure from the damaging effects of ground accelerations. One of the goals of seismic isolation is to shift the fundamental frequency of the structure away from the dominant frequencies of seismic excitations and the fundamental frequency of the fixed superstructure. This innovative design approach aims mainly at the isolation of the structure from the supporting ground usually only in the horizontal direction, in order to reduce the transmission of the earthquake motion to the structure. In most applications it is placed at the base of the structure and is thus referred to as base isolation. Although three-dimensional isolation systems have also been developed to account for seismic excitation in both the horizontal and vertical directions, they are very complicated and thus hard to be practically implemented. In addition, for most structures the vertical earthquake components are not as dangerous as the horizontal ones. The main reason is that the majority of structures are designed and built to carry vertical loads and are therefore less vulnerable to additional such loads. Also, the vertical components of ground motion are usually weaker than the horizontal ones.

3.2 Basic Components

Seismic isolation introduces flexibility at the base of the structure in the horizontal direction, shifting its fundamental frequency away from the dominant frequency range of the earthquake. In addition, it also provides an energy dissipation mechanism at the level of isolation, reducing the relatively large relative displacements between the superstructure and the supporting ground. Finally, the seismic isolation system provides either rigidity under minor lateral loads, e.g. wind loads, or an energy dissipation mechanism to suppress the response due to service loads.
3.3 Advantages and Applications of Seismic Isolation

The increased flexibility of the seismically isolated structure is its most important feature and results in the avoidance of resonance and the decrease of the accelerations of the structure at different levels. In Chapter 2 it has been discussed that, while a very flexible structure has very small accelerations and very large relative displacements, a very rigid structure has very small relative displacements but accelerations equal to the ground acceleration. Seismic isolation actually takes advantage of the benefits of both the above-mentioned extreme cases, reducing both interstory deflections and floor accelerations. Interstory deflections are reduced because of the rigid body motion of the superstructure. The superstructure is relatively very stiff compared to the inserted flexibility at the isolation level. The reduction of interstory deflections of the structure minimizes or even eliminates structural or non-structural damage. In addition, the lengthening of the fundamental period due to the inserted flexibility reduces the accelerations significantly. This is an important achievement, in particular for buildings with very sensitive equipment which must remain operational even after a severe earthquake. Finally, appropriate design of the isolation system may reduce torsional effects due to the eccentricities of the superstructure. This may easily be achieved by coinciding the center of stiffness of the isolation system with the center of mass of the superstructure.

Another advantage of seismic isolation is related to the energy dissipation mechanism. In conventional earthquake design the energy dissipation mechanism is based on plastic deformations at scattered locations in the structure. In contrast, in the case of seismic isolation, the energy dissipation mechanism is concentrated at the isolation level and may therefore be more easily designed, particularly to withstand several inelastic cycles under reversed loading, and may also be monitored to ensure appropriate performance. Note that the design must allow for the easy replacement of the energy dissipation mechanism in the case of malfunction, as it will be described in Chapter 5. Since the dissipation of energy of a seismically isolated structure does not depend primarily on the ductility of the superstructure but on the dissipation mechanism of the isolation system, the ductility requirements of the superstructure could ideally be partially released. However, considering the limited experience in seismically isolated structures, and the uncertainties associated with the method itself and the nature and frequency of occurrence of the ground excitations as
well as other unexpected excitations, such as earthquakes with strong vertical and long period components, the existing ductility requirements should not be released.

Seismic isolation is based on the local insertion of isolators and the provision of a gap to accommodate the relative displacement of the superstructure at the isolation level. It may be applied to both new and existing structures, especially in bridges which are usually founded on common thermal expansion bearings. In such cases, the installation of the isolation system is simply achieved by replacing the common bearings with the isolation ones. Lead rubber bearings are suitable in these cases, as they provide all the features of seismic isolation in one compact unit. In addition, proper sizing of the lead plug cylinder and the bearing may be used in the redistribution of the lateral forces from individual weaker piers of lower seismic capacity to stronger piers and abutments. The recent strong earthquakes and the resulting traffic disruption indicate the necessity of seismic upgrading of existing bridges to remain operational after such strong events. Many bridges have been built with insufficient seismic performance requirements. These bridges must be seismically upgraded and seismic isolation is a relatively cheap and effective method, easily implemented without significant disruption of the operation of the bridge.

Seismic isolation may also be used to retrofit existing structures not appropriately designed for seismic loads and in particular to rehabilitate historical structures which must maintain their original architectural characteristics. In these cases, instead of increasing the ability of the structure to withstand the typical seismic loads for a fixed supported structure, the imposed seismic loads are substantially decreased using seismic isolation. The required ability to withstand seismic forces and the ductility demand capacity of the structure are decreased to appropriate levels. Seismic isolation is the least disruptive technique since most construction work is usually confined at the level of isolation, typically at the basement of the building. The loss of functionality of the superstructure is limited, considering that only limited stiffening or strengthening may be required. This is very important for buildings that house essential facilities which must remain operational during seismic retrofitting. In contrast, conventional strengthening of buildings involves extensive modifications at all levels, and the loss of their functionality is unavoidable. For historic structures, where the preservation of the architectural characteristics of the superstructure dominates the design, seismic isolation has the least aesthetic impact. This is also
due to the fact that seismic isolation reduces the level of seismic forces and there is, there-fore, no reason to significantly modify the superstructure to sustain the large seismic forces of a fixed supported one. In contrast, conventional strengthening techniques modify the structure to make it stronger which in turns attracts higher seismic forces. Unfortunately, practical constraints in several existing buildings, historical ones in particular, do not allow the use of seismic isolation. On the other hand, seismic isolation may be used for new structures in combination with methods of construction which are usually not suitable in earthquake-resistant design, since the induced seismic forces are substantially decreased without the need of high ductility.

3.4 Comparison of a Fixed Supported and a Seismically Isolated Structure

A 3-story structure shown in Figure 3.1, both fixed supported and seismically isolated, is studied to demonstrate the effectiveness of seismic isolation. The structural characteristics of both models and their responses are summarized in Table 3.1. The latter include maximum interstory deflections, mass accelerations and story shear forces at each level of both the fixed-supported and the seismically isolated structures. The analysis is performed assuming the El-Centro excitation, scaled to have a peak ground acceleration ($pga$) of 0.50g. It is observed that maximum floor accelerations, interstory deflections and story shear forces are reduced substantially, even if the energy dissipation mechanism of the isolation system is neglected. For both structures a 5% damping ratio $\xi$ is used. If the relative damping, either hysteretic or viscous, of the isolation system is taken into account, further reductions in the response of the isolated structure result.

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<tr>
<td>Isolation</td>
<td>335</td>
<td>30</td>
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<td>136.1</td>
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<td>1</td>
<td>305</td>
<td>1250</td>
<td>5.7</td>
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<tr>
<td>2</td>
<td>285</td>
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<td>3</td>
<td>265</td>
<td>1125</td>
<td>2.9</td>
<td>0.8</td>
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Table 3.1: Structural characteristics and maximum response of both systems.
Figure 3.1 shows the relative displacements of the fixed supported system, and Figure 3.2 shows the relative displacements of the same system, when flexibility is inserted at its base. Figures 3.3 and 3.4 shows the story shear forces for both models.

Figure 3.1: Interstory deflections of a fixed supported system.

Figure 3.2: Interstory deflections of a system mounted on a flexible spring.
From the structural characteristics and the response of both models it is evident that when a conventional earthquake-resistant system is used, the seismic forces, relative displacements and accelerations are larger than those occurring when a seismic isolation system is used.
3.5 Constraints in the Implementation

The main constraint in the practical implementation of seismic isolation is the large relative displacements between the superstructure and the supporting ground at the isolation level, which must be ensured for the whole life of the structure. Large relative displacement at the isolation level limits the extent of isolation. It is indicated by the shift of the fundamental period of the isolated structure with respect to that of the fixed supported superstructure. However, this displacement may be reduced by providing an energy dissipation mechanism. Among several such mechanisms the ones most commonly used are based on the plastic deformation of a metal, such as lead or steel, or to the inherent damping properties of some types of rubber. Another constraint is the possibility of uplift of the isolators. During severe earthquakes, the lateral seismic forces and resulting moments may cause axial loads larger than the gravity loads. In such cases, tensile stresses may be applied to the elastomeric bearings which are not designed to carry these stresses, and which may thus impoverish the bearings’ performance. Connection details and proper configuration of the layout of the structure are necessary to avoid excessive amounts of uplift forces. The possibility of uplift of the isolators increases with height and particularly with the aspect ratio (height-to-width) of the structure due to the increased overturning moments. This makes the use of seismic isolation in high-rise buildings, inappropriate. Also, isolation of a very flexible structure may be inefficient, since the fundamental frequency of such a structure is usually already outside the dangerous for resonance range. Thus, the main feature of seismic isolation, the avoidance of resonance, has no significant effect on flexible structures. In addition, high-rise buildings are more vulnerable to wind loads than to earthquake loads and seismic isolation is not an appropriate way to handle these loads. Seismic loads do not usually dominate the design of high-rise buildings. Some devices have been developed to provide energy dissipation without the incorporation of isolation and are more suitable for high-rise buildings. Active control, either in the form of tuned mass dampers or closed-loop actuators, may be used to mitigate wind excitations.

3.6 Limitations

Although seismic isolation is a very promising design method to account for earthquakes loads, it may not be used for all structures and at all sites. Some examples have been given
in the previous section. Therefore, a feasibility study must be carried out, early in the
design phase, to determine the plausibility of this design approach. This study must be
based on a number of factors:

**Superstructure characteristics:** As previously mentioned, in general seismic iso-
lation is suitable for low- to medium-rise buildings (usually up to 10-12 stories) which
have their fundamental frequency in the range of the usual dominant frequencies of earth-
quakes. Superstructure characteristics, such as height, width, aspect ratio and stiffness are
related to the applicability and effectiveness of seismic isolation.

**Seismic hazard and site characteristics:** Seismicity of a particular region must be
considered, to determine the necessity of seismically isolating a structure in that region.
Foundation soil characteristics, such as the site period $T_s$ and the expected dominant fre-
quencies of earthquakes in a particular area, are factors which must be considered in a fea-
sibility study. In general, the stiffer the foundation soil, the more effective seismic
isolation is. In cases where site characteristics (e.g. soft soils) and information from previ-
ous earthquake records indicate low dominant site frequencies, i.e. high $T_s$, seismic isola-
tion must be avoided. The site period $T_s$, considering geotechnical characteristics,
corresponds to the fundamental vibrational mode of the geotechnical soil profile. Deep
deposits of soft soils usually amplify the low frequency motions, eliminate the high fre-
quency motion, and impose low frequency ground motion on the structure. In these cases,
the foundation soil may be treated as a physical partial seismic isolation and the structure
must be as rigid as possible. In addition, the site-expected earthquake predominant period
tends to be lengthened with distance from the epicenter, due to high frequency component
attenuation. A flexible structure is more vulnerable to low frequency motion due to reso-
nance. Finally, another issue which must be considered is the proximity to active faults
due to possible long period components of the expected seismic excitation. These are
near-fault effects caused when a rupture propagates towards the region with fault rupture
propagation velocity equal to the seismic wave velocity, resulting in long unidirectional
pulses. The latter may lead to higher displacements than those expected.

**Surrounding structures:** All adjacent structures or facilities which may impose
restrictions on the seismic isolation system must be taken into account, especially in order
to estimate the maximum allowable displacement. A clearance around the building, of the
order of 10 to 20 cm, must be ensured for its whole life, to accommodate large relative dis-
placements between the superstructure and the ground foundation. Special care must be
taken in heavily constructed areas to avoid collision of the structure with surrounding
ones. Note that there are also cases where the location of adjacent structures do not allow
the use of seismic isolation. Finally, the surrounding ground may also impose restrictions
and special design details may be needed to enable the use of seismic isolation. For exam-
ple, a seismically isolated structure located on a slope of a hill must be protected by spe-
cial design and construction details from possible application of soil thrust pressure on it,
which would be detrimental to seismic isolation.

**Minor lateral loads:** Wind loads and other lateral loads, must be limited, i.e. usually
less than 10%; otherwise seismic isolation is inappropriate. It must be ensured that the
seismically isolated structure performs well under these minor loads.

### 3.7 Factors Which Enable the Use of Seismic Isolation

Several factors favor the development and practical application of seismic isolation. First
of all, there are increased requirements for the performance of structures under severe
earthquakes which are not met by the current conventional design philosophy. These are
crucial to structures with sensitive and expensive equipment, vulnerable even to
microtremors. Second, the advancement of computer technology and modern structural
analysis methods enable the development of reliable software to simulate the response of
structures. The development of seismic engineering and earthquake engineering, to the
level that reliable predictions can be made for expected earthquakes, is another important
factor. Today a dynamic time history analysis is easily implemented on a personal com-
puter. In addition, the construction of shaking tables which can simulate actual earthquake
excitations, make possible the experimental validation of the behavior of seismic isolation
systems. The study of other minor loads such as wind loads, and the reliable quantification
of their expected intensities and frequency of occurrence, also enable the use of seismic
isolation. Finally, development, manufacture, and extensive research in the area of struc-
tural materials enable the reliable use of modern materials for seismic isolation devices.
The development of devices which dissipate energy and provide a restoring force to avoid
permanent displacements also allow the practical implementation of the seismic isolation


3.8 Economics of Seismic Isolation

In terms of cost, although seismic isolation usually requires an additional initial cost, it ensures long term savings due to better seismic performance and damage reduction. In particular, an additional initial cost of the order of 1% to 5% of the structural cost, may be required to incorporate seismic isolation in a new building [Mayes et al., 1990]. However, the isolated structure is designed to respond to a strong earthquake without the damage of a conventionally designed fixed-supported structure. When the method is used for seismic upgrading of an existing building, savings may result according to the level of the structure’s seismic performance capacity, the desired seismic upgrade, and the already present constraints. In bridge construction or seismic upgrading, seismic isolation usually results in cost savings, especially when high performance requirements are considered in the design. Most existing bridges may be seismically isolated by simple substitution of the existing thermal expansion bearings, which in many cases have to be anyway replaced due to deterioration, by isolation bearings and provision of the required gap for the relative displacements of the superstructure.

It is difficult to compare seismic isolation with conventional design in economic terms, due to their different performance, the uncertainties associated with their response and the exact level of damage each method allows. For a specific earthquake, and with the acceptance of the same damage, seismic isolation typically reduces the initial construction cost. An example which supports this argument is nuclear plants, for which the earthquake performance requirements are specifically defined and thus their design implies equal performance. A feasibility study for a nuclear power plant [Burns and Roe, 1983], showed that incorporation of seismic isolation results in a cost savings of $34 million for a project of $1760 millions, i.e. the savings are of the order of 2% of the initial structural cost. A detailed comparison between the cost of seismic isolation and that of conventional earthquake resistant design was made for the Fire Command and Control Facility of the Los Angeles County. An essential design requirement was to ensure that the building would remain functional under the maximum credible earthquake. It was found that seismic isolation would result in approximately 6% savings more than the conventional fixed sup-

concept.
ported design. The significant savings difference resulted from the reduced mechanical and electrical equipment requirements, because of the smaller imposed seismic loads. In contrast, more stringent equipment and non-structural requirements had to be adopted for the conventionally designed building to reduce non-structural damage, considering larger seismic loads [Anderson, 1989]. In most cases, the seismic isolation option is compared to other conventional options but at different performance levels. The aim of seismic isolation is to account for earthquake excitations with better performance, avoid the damage resulting when the structure is fixed supported, and not to reduce the initial construction cost for the same level of performance. The goal of seismic isolation to avoid damage cannot be achieved with a conventional design of reasonable cost.

There are, however, several factors that may cause an increase in the cost if seismic isolation is used. First, it is the cost of the isolation system itself; a typical lead rubber bearing costs between $5,000 to $25,000. Also, additional members or structural changes may be required to insert the isolators; this also implies an additional cost. For example, an additional slab at the isolation level is necessary to have uniform deformation and stress distribution in the isolators. Cost of modifications, such as shear walls around the structure to accommodate the large relative displacement of the isolation system must also be taken into account, as well as the loss of potentially usable area which is used for the required seismic gap. This loss may be significant in densely populated areas where land is expensive. Finally, seismically isolated structures require rigorous maintenance and frequent inspections to ensure their adequate performance during the life-span of the structure, which result in additional cost.

There are also several factors that may lead to savings when seismic isolation is used. The superstructure construction cost may be lower due to lower seismic forces and reduced seismic capacity demand, which is more evident in seismic retrofitting of existing structures. The reduced seismic loads allow more deliberate design of the superstructure. Thus, construction methods which are not suitable in high seismicity areas may then be used for seismically isolated structures. Such methods include prestressed and precast concrete construction which do not provide sufficient ductility capacity. In addition, considering the substantially smaller interstory drifts, cheaper non-structural parts, such as cladding, may be used. Long-term savings also result from the reduction of repair costs for
structural and non-structural damage due to the substantially decreased interstory displacements, and from the increased building contents protection due to reduced floor accelerations. The avoidance of the disruption of the structure’s operation due to the better performance of seismic isolation, also results in savings. In general, repairs are very time consuming and the loss of functionality of some structures may be more disastrous than the actual damage. The use of an effective earthquake-resistant design such as seismic isolation always results in benefits from the increased safety and the reduced injuries, deaths and related lawsuits. However, savings from insurance costs cannot be currently estimated since insurance companies do not provide special premiums for seismically isolated structures. Such premiums can be more readily provided, though, for structures with higher reliability than for conventionally designed ones. Insurance companies define such high deductibles and premiums and it may be more beneficial to incorporate seismic isolation to reduce damage below the deductibles, and thus counterbalance the additional initial cost from the amount paid for annual insurance premiums.

3.9 Reasons for the Lack of Extensive Use of Seismic Isolation

The extent of use of seismic isolation is affected by several factors. Clients are not always aware of the objectives, limitations and undesirable consequences of conventional earthquake-resistant design. They are often mislead to believe that a conventional fixed supported building, designed with the current earthquake-resistant codes is absolutely earthquake proof, for earthquakes of any magnitude. In general, people learn of the actual purpose of the current earthquake design code only after a severe earthquake of disastrous consequences. Thus, they rely on the current conventional approach and are not motivated to use a more controversial and yet promising approach, especially when its incorporation requires an additional cost. Indeed, seismic isolation, in general, requires an additional initial cost and provides savings only in the long run, when different performance requirements are used. It is very difficult to convince people, who are usually interested on immediate savings, to invest on a design which will provide only long-term savings resulting from the reduced damage of a possible future earthquake. As long as the decision on the design and construction of a structure is dominated by its initial cost, it is difficult for seismic isolation to be widely used. When more strict requirements for the seismic
response and the allowable level of damage are adopted, seismic isolation will be compared to the currently used design method under approximately the same response requirements, and may be shown to ensure savings even in the initial cost.

Information on seismic isolation available to the public is limited. In addition, the method is not introduced as a topic of study in most universities. Therefore, structural engineers cannot consider it as an alternative earthquake design method. Only in few universities this method is taught or even mentioned to students. Therefore, its practical implementation, which requires exposure of structural engineers to seismic isolation, is limited. Also, design firms involved in seismic isolation projects release only limited pertinent information to the civil engineering community, due to market competitiveness. I believe, however, that the design aids, procedures and, any other pertinent information must be published and distributed not only to practicing structural engineers but also to students. The release of related to seismic isolation information and data from manufacturing, design and construction companies is essential in the wide-spreading of the method.

Seismic isolation is an innovative and unconventional design idea and engineers are usually conservative in implementing such ideas. Thus, in the same way that it took a few decades for the use of prestressed concrete in construction to be established, it may take time for seismic isolation to be understood and established. Considering that seismic isolation has been used in many projects in the last 30 years, the time for its establishment has possibly come. For the practical implementation of seismic isolation several people with different interests and backgrounds must work together or independently. In this way, large companies which specialize on seismic isolation should have a staff of engineers and scientists, or even different divisions, which will take care of the different issues involved in the implementation of the method.

Unfortunately, most isolation systems are currently patented and may therefore be used only by the few companies which have the rights of these systems. Architectural and construction technology has been developed and advanced for thousands of years and therefore belongs to society. Constraints on the free use of seismic isolation systems can only cause more casualties and extensive damage, a fact that the companies which have the systems' exclusively rights do not consider, in view of the large profits they make.
Chapter 4

Seismic Isolation Devices

4.1 Types of Seismic Isolation Systems

Various seismic isolation systems have been developed and several have also been applied in practice. Seismic isolation systems used in the United States include ones that consist of elastomeric bearings and sliding systems. Figure 4.1 shows the current percentage use of these systems.

8% Sliding Systems - Friction Pendulum
23% High Damping Rubber Bearings (HDRB)
69% Lead Rubber Bearings (LRB)

Figure 4.1: Seismic isolation systems used in United States as of January 1994.

The above systems combine simplicity, reliability and economy. In addition, there is substantial experience, at least with elastomeric bearings since they have already been used extensively in bridge construction to accommodate deformations due to thermal changes. Several other systems have been developed, but not as successful.

4.2 Sliding Isolation Systems

Sliding systems, which are based on friction, allow transmission of shear forces up to a particular level, beyond which sliding occurs and transmission is prevented. Such systems may be designed to allow a very low base shear force. This is independent of the severity of the earthquake since the transmitted force is based on the friction coefficient. Thus, sliding systems are very efficient to mitigate the effects of severe earthquake excitations. In addition, they are relatively cheap and of compact size and thus appropriate to be used in retrofits of existing structures. However, many sliding isolation systems may not return to their initial configuration after the excitation, due to absence of a restoring force. Permanent offset of the isolated structure from its original location may result, decreasing the displacement to be accommodated during a future earthquake. In addition, the significant difference between the initial, i.e. prior to sliding, and the post-yield stiffness causes high frequency effects. These may in turn increase the floor accelerations and consequently the
possibility of damage of the structure’s contents. Therefore, sliding systems are not suitable for structures with content protection requirements. Another disadvantage is that the coefficient of friction is not constant; it may vary with temperature and time due to material deterioration. This may cause an increase in the actually inserted seismic forces, resulting in severe damage if lower seismic forces have been considered in the design. The combination of sliding systems with elastomeric bearings results in hybrid systems which may have the advantages of both individual systems.

4.3 Elastomeric Isolation Systems

The most popular seismic isolation systems use elastomeric bearings (Figure 4.2) which consist of thin rubber sheets bonded on thin steel plates and combined with an energy dissipation mechanism. The latter is based either on the plastic deformation of a metal or on the inherent damping properties of the rubber. In the first case, either lead plugs are inserted in the elastomeric bearings or auxiliary dampers based on plastic deformations of lead or steel are used.

![Elastomeric Bearing](image)

**Figure 4.2:** Elastomeric Bearing.

Lead rubber bearings (LRB) and high damping rubber bearings (HDRB) are most useful in seismic isolation since in one unit, they provide the following:

- Vertical support due to the high vertical stiffness, which is usually several hundred times the horizontal stiffness. Sufficient vertical stiffness is necessary to avoid rocking of the structure.

- Horizontal flexibility which shifts the fundamental frequency of the structure out of the dangerous, for resonance, frequency range.

- An energy dissipation mechanism, either by the plastic deformation of the lead plug or by the inherent damping properties of high damping rubber.

In addition, a lead rubber bearing provides initial rigidity under service lateral loads, such as wind loads, due to the presence of the lead plug and its high stiffness prior to
yielding. In that case though, the energy dissipation mechanism is activated only once the lead plug has yielded. High damping rubber bearings provide an energy dissipation mechanism on a continuous base, but not the above-mentioned initial rigidity. Finally, there are also some systems that use natural rubber bearings (NRB) with an additional steel or lead damper; in this case energy dissipation results from the plastic deformation of the damper.

4.3.1 Lead Rubber Bearings (LRB)

This type of bearing consists of thin layers of natural rubber sandwiched between steel plates, and a lead cylinder plug firmly fitted in a hole at its center to deform in pure shear (Figure 4.3). Lead is a crystalline material which changes its structure temporarily, under deformations beyond its yield point, and regains its original structure and elastic properties as soon as the deformation is removed by the restoring force in the rubber. As the lead is forced to deform plastically in shear, once it has exceeded its yield stress, it dissipates energy hysteretically. Lead has an elastic plastic behavior with a yield stress \( \sigma_y \) of approximately 10.5MPa. The strength of lead is much less than the theoretically computed one from the interatomic distance-force curve, due to the presence of defects termed dislocations. These allow the bonds to break, one at a time, causing the material to yield and deform plastically for relatively low stress. Note that lead has good fatigue properties for subsequent cycles of loading beyond its yield point.

![Figure 4.3: Lead Rubber Bearing.](image)

Hysteretic energy dissipation is equal to the area offset of the loading and unloading curves under cycling loads. At each cycle this area represents the energy dissipated in the form of heat. It is actually due to work done during loading which did not completely recover during unloading, but was converted from kinetic to thermal energy and dissipated.
The behavior of the LRB is characterized by a high initial, i.e. prior to yielding, stiffness \( K_{el} \), due to the presence of the lead plug \( K_l \), and by a low post-yield stiffness \( K_{pl} \) which is equal to the shear stiffness \( K_r \) of the rubber, (Figure 4.4). The force required for a particular displacement, when the yield force of lead has been exceeded for the first time is equal to the sum of the two individual forces, since they act as springs in parallel. The force prior to yielding is equal to \( F = \sigma_y A + K_r u = (K_{el} + K_{pl}) u \) and that after yielding is \( F = K_r u = K_{pl} u \). For reversed cyclic loads the exact history of the loading is required to obtain the force \( F \).

![Figure 4.4: Mechanical behavior of LRBs.](image)

Usually, the behavior of the LRB is modeled by an equivalent linear viscously damped system with an effective stiffness \( K_{eff} \) and an equivalent viscous damping ratio \( \xi_{eff} \), as it will be described in Chapter 6. However, this is not always a good approximation, since the equivalent viscous damping varies considerably with both the specific characteristics and the intensity of earthquake, as well as with the characteristics of the isolation system and the superstructure. Lead rubber bearings provide initial rigidity, due to the high elastic stiffness of lead, which is essential for minor lateral loads. The presence of the lead plugs, the variable horizontal effective stiffness and the variable effective fundamental period of the structure, enable the structure to vibrate outside the dangerous for resonance frequency range as the response increases. The initial stiffness of the LRB is very high but as the intensity of the excitation increases, the stiffness is reduced and the isolation system becomes more effective. The initial elastic stiffness \( K_{el} \) is approximately ten times the post-yield stiffness \( K_{pl} \). The horizontal stiffness of the LRB decreases as shear strain increases; beyond 250-300% strain it increases again due to hardening effects [Robinson, 1982]. The breaking point occurs at shear strain levels up to 500% and is found to be inde-
pendent of compressive or tensile stresses and the presence of the lead plug. The shear strain may also be considered independent of the vertical load. However, the vertical stiffness depends on shear deformations. Failure of the bearing occurs mainly due to the formation and growth of flaws in the elastomer. The extent of rupture depends on the compounding and stiffness of the material.

It has previously been mentioned that lead rubber bearings do not provide an energy dissipation mechanism under service loads, i.e. prior to the yielding of the lead plugs. This makes them ineffective to dissipate energy against microtremors or minor lateral loads which may affect structures with sensitive equipment. Auxiliary dampers may therefore be incorporated to dissipate energy prior to yielding. Also, the actually bilinear behavior of the LRB results in a non-constant equivalent damping ratio which varies with both the excitation characteristics (intensity, dominant frequencies, etc.) and the isolated structure’s characteristics. Finally, the continuous sudden changes of stiffness of LRB may excite higher modal response effects.

4.3.2 High Damping Rubber Bearings (HDRB)

This type of bearing consists of thin layers of high damping rubber sandwiched between steel plates. High damping rubber is actually a filled rubber compound with inherent damping properties due to the addition of special fillers, carbon in particular. The addition of fillers increases the inherent damping properties of rubber without affecting its mechanical properties. When shear stresses are applied to high damping rubber, sliding of molecules generates frictional heat which is a mechanism of energy dissipation. In unfilled natural rubber, used for LRBs, frictional heat is negligible because the molecular attraction in physical cross links is very weak. The energy dissipation mechanism of a HDRB is available for both small and large strains, is constant and characterized by smooth elliptical hysteresis loops. However, a HDRB does not provide the necessary initial rigidity under service loads and minor lateral loads. A structure isolated with HDRBs has a constant, large fundamental period due to the flexibility of the isolation system, which makes the structure vulnerable to wind action with dominant frequencies close to the fundamental frequency. In addition, the damping and mechanical properties of the HDRB appear to be temperature dependent while the hysteretic energy dissipation mechanism of the LRB is not. HDRBs are not as widely used in seismic isolation as the LRBs.
4.3.3 Hybrid Type: Lead High Damping Rubber Bearing (LHDRB)

A hybrid type of lead high damping rubber bearing (LHDRB) may consist of layers of high damping rubber sandwiched between steel plates and a smaller diameter lead cylinder plug firmly fitted in a hole at its center. This hybrid bearing may have the advantages of both isolation systems discussed above. The LHDRB has both an initial rigidity, due to the presence of the lead plug, and a continuous energy dissipation mechanism, due to the damping properties of the high damping rubber. Therefore, the isolation system is expected to perform well under both weak and extreme earthquakes as well as under minor lateral loads. In addition, the properties of this bearing will be less dependent on temperature changes and shear strains than those of HDRB. Also, the use of LHDRBs allows the reduction of the initial stiffness of the isolation system which is responsible for higher frequency effects. A final advantage is that the effectiveness of such a device and its compact size makes it suitable for cases where installation space is limited.

When practically possible, it may be more effective to place the high damping rubber bearings with lead plugs at the perimeter of the building, preferably under the columns situated as far away as possible from the center of stiffness and mass of the isolated structure. High damping rubber bearings with no lead plugs may be used under the internal columns. This configuration will allow lower prior to yielding stiffness of the lead plugs and, consequently, a smoother change of the stiffness during yielding and reverse loading. The high initial stiffness and its sudden changes are responsible for higher mode effects and acceleration increases, which may be avoided by reducing the initial stiffness. In addition, lower initial stiffness will provide a higher degree of isolation at the prior to yielding stage. Placing the bearings with the lead plugs under the external columns away from the center of stiffness will provide higher resistance against torsion, due to the larger distance of the points of application of forces from the center of stiffness of the isolation system. Note that this configuration may be used only when there is a rigid diaphragm at the isolation level to redistribute the inertia forces to the LHDRBs.

An effective isolation system must have both viscous and hysteretic damping and must ensure a continuous energy dissipation mechanism. The viscous damping, which is velocity dependent, will ensure a continuous energy dissipation mechanism for both severe
earthquakes and microtremors. Viscous damping may be provided by actual viscous dampers or by rubber with inherent damping properties. The latter does not actually provide such damping, which may, however, be assumed due to the rubber’s smooth elliptical hysteresis loops during cyclic loading. The optimum viscous damping ratio lies within 20 to 30%; higher values lead to increases in floor accelerations. Viscously damped systems are not so effective for severe excitations and do not, in general, provide high initial stiffness. As already mentioned, hysteretic damping is usually provided by the plastic deformation of a metal, typically lead or steel, and is very effective for strong motions reducing substantially the response, especially the relative displacement at the isolation level. It provides high initial stiffness, required for service lateral loads, but not a continuous dissipation mechanism. The sudden stiffness changes may increase the floor accelerations. Therefore, a compromise between these two damping extremes would be optimum in the design for a seismically isolated structure.

4.4 Mechanical and Physical Characteristics of Elastomeric Bearings

4.4.1 Characteristics

Elastomeric bearings must be strong and stiff for vertical loadings and flexible under shear stresses. The properties of the bearings can be controlled by the proper selection of the elastomer compound and the bearing geometry. The bearings must be designed to be sufficiently large to be able to support, with an appropriate factor of safety, the maximum expected vertical loads or the service vertical loads considering the corresponding maximum horizontal displacement. In the latter case, vertical forces are transmitted through an active area of the bearing due to deformations. Steel laminates are bonded to the rubber to restrict bulging under compressive stresses. The thickness of the steel laminates \( t_s \) and the thickness of the rubber sheets \( t_r \) determine the vertical stiffness of the bearing. The vertical load capacity of a bearing increases as the thickness of the rubber sheets is reduced. The total height \( h_r \) of the rubber \( (h_r = n_r \cdot t_r) \), is determined by the required horizontal flexibility to be inserted, taking into account the mechanical properties of the elastomer. The diameter of the lead plugs of the LRBs is determined by the required initial rigidity for minor lateral loads. The area of the lead plugs must be such that the corresponding shear forces may be transmitted without yielding. The yield force level is usually defined as a
percentage (typically of the order of 5%) of the weight of the structure. The diameter of HDRBs may be chosen in a similar way to transmit minor lateral loads or a fraction of them without yielding. Finally, the location and dimensions of the bearings must be such that torsional vibrations of the structure are avoided. This is easily achieved by forcing the center of stiffness of the isolation system and the center of mass of the superstructure to coincide. When LRBs or LHDRBs are used, special care must be taken to ensure that the above mentioned centers coincide, both prior to and after yielding of the lead plugs.

It must be ensured that the mechanical and damping properties of the bearings remain constant over the whole life of the structure. In addition, design details, as it will be described in Chapter 5, must be used to allow replacement of malfunctioning units. The elastomer which is used in the manufacture of the bearings is characterized by its ability to return to its original configuration and dimensions when unloaded. It usually consists of natural rubber (NR-natural polyisoprene), neoprene (CR-chloroprene), E.P.D.M. (ethylene propylene diene monomer), or nitrile butadiene rubber (NBR). Natural rubber and neoprene are very well known materials and are extensively used in common bridge bearings. The chemical composition and the manufacturing process determine the mechanical and physical properties of the elastomer. This has long randomly oriented molecule chains and cross-links between the chains, introduced during the vulcanization process. The molecule chains are held together by sparse strong covalent cross-links and weak Van der Waals bonds, when below the glass-transition temperature $T_g$. The latter bonds are due to the non-uniform distribution of electrons; the former are due to sharing of electrons between atoms. When the elastomer is deformed, the molecules tend to align, thus changing the entropy of the material. The high flexibility of the elastomers is due to this change of the entropy. The restoring force and the shear and Young’s moduli are mainly due to the reordering of the chains and their tendency to return to a random configuration. When a load is applied in all three directions, reordering cannot take place and the stiffness is very high; thus the bulk modulus of rubber is much higher than its Young’s and shear moduli. Although nonlinear, the behavior of the elastomer is elastic since it returns to its initial configuration, and its properties return to their original values when the applied load is removed. At very small strains, the elastomer is essentially linear while at higher strains the molecule chains change orientation and align in the direction of the loading.
The mechanical properties of the elastomer are time and temperature dependent. In general, its stiffness and damping properties decrease as temperature increases. The bearing may stiffen in low temperatures, with the danger of possible crystallization of the elastomer when exposed to very low temperatures for a long time. For some regions where temperatures are very low, thermal stiffening of the elastomer may be a serious problem since it reduces the flexibility of the isolation system and consequently the degree of isolation. In such cases, this problem must definitely be considered and an appropriate elastomer compound must be selected. In particular, neoprene becomes extremely stiff at approximately \(-40\) °C, approaching a glass transition condition, while natural rubber approaches a glass transition condition at \(-55\) °C [Roeder et al, 1987]. At such low temperatures the elastomer bearings may transfer forces which are several times larger than those expected and the assumed isolated structure may actually be fixed supported. The temperature dependence of the material properties must be taken into account, to avoid the use of incorrect values measured under different than the actual on site conditions. In what concerns time dependence, according to experiments [Stanton and Roeder, 1982] and [Robinson, 1982], deformation capacity and ultimate strength have no significant time dependencies. Measurements of the Young’s and shear moduli depend on the method used. Dynamic tests give much higher values up to twice the statically measured ones. It is therefore more reasonable to use dynamically measured values. The Young’s and shear moduli may be determined using a hardness test [Stanton and Roeder, 1982].

Elastomers sufficiently resist the action of oil and chemical substances, assuming that only their surface may be exposed to such effects. Bridge rubber bearings have been used extensively for very long periods with no significant deterioration problems. Natural rubber has been extensively used for automobile engine mounts under long continuous exposure to oils, without significant damage. Ozone concentration for long periods may result in ozone cracking, which mainly occurs when the rubber is in tension, a situation which is not of concern in the case of bearings. Ozone cracking may be reduced or even eliminated via the addition of an antiozonant during the compounding process. A protective end and side rubber cover may be used to protect the bearings from weather, moisture, ozone and other dangerous effects. The rubber cover also increases the bearing’s fire resistance which is though satisfactory, according to related experiments. Finally, bonding of the rub-
ber sheets on the steel plates does not create any problems since the adhesion between rubber and steel plates is greater than that of the rubber itself.

4.4.2 Design Considerations

Two shape factors, $S_1$ and $S_2$, are usually used to describe the geometric characteristics of elastomeric bearings which are related to their mechanical properties. The primary shape factor $S_1$ is defined as the ratio of the area of the rubber constrained by the steel plates to the free surface area. Appropriate values of the primary shape factor are between 10 and 20 [Kelly, 1991]. This factor influences the vertical displacement; as $S_1$ increases the vertical stiffness increases. For a circular bearing, $S_1$ is given by

$$S_1 = \frac{\text{Loaded Area}}{\text{Force Free Area}} = \frac{\pi D^2}{4 \pi D t_r} = \frac{D}{4 t_r}$$

The secondary shape factor $S_2$ is defined as the ratio of the diameter of the rubber to the total thickness of the rubber layers

$$S_2 = \frac{D}{n_r t_r}$$

where $D$ is the diameter of bearing, $t_r$ the thickness of rubber sheet, and $n_r$ the number of rubber sheets.

Although the behavior of the elastomer is nonlinear, a linear elastic analysis is useful for the design of elastomeric bearings. The following assumptions are made in such an analysis:

- At any point $\sigma_{xx} = \sigma_{yy} = \sigma_{zz} = p(x, y)$ (Figure 4.5.a).
- The distribution of pressure $p(x,y)$ is parabolic and the pressure is zero on the bearing faces and circumference (Figure 4.5.b).
- Points lying on a vertical line prior to loading lie on a parabola after deformation (Figure 4.5.c).
- Horizontal planes remain horizontal after deformation.
Using the above assumptions, the displacements are given by

\[ u(x, y, z) = u_o(x, y) \left( 1 - \left( \frac{2z}{t_r} \right)^2 \right) \]  
(4.3)

\[ v(x, y, z) = v_o(x, y) \left( 1 - \left( \frac{2z}{t_r} \right)^2 \right) \]

\[ w(x, y, z) = w(z) \]

The change of volume of an infinitesimal element of dimensions \( dx, dy \) and \( t_r \) in the \( X, Y \) and \( Z \) directions, respectively, is

\[ \Delta V = \varepsilon_c t_r \ dx \ dy - \frac{\partial}{\partial x} \left( \frac{4}{6} t_r \ u_o(x, y) \right) \ dx \ dy - \frac{\partial}{\partial y} \left( \frac{4}{6} t_r \ v_o(x, y) \right) \ dx \ dy \]  
(4.4)

where \( \varepsilon_c \) is the compressive strain. This volumetric change is related to the bulk modulus \( K \) of the elastomer through the expression

\[ \frac{\varepsilon_c}{K} = \frac{p \ \Delta V}{K} = \frac{p \ \varepsilon_c}{K} \]

Thus,

\[ \frac{p}{K} = \varepsilon_c - \frac{2}{3} \left( \frac{\partial}{\partial x} u_o(x, y) + \frac{\partial}{\partial y} v_o(x, y) \right) \]  
(4.5)

(Bulk and shear moduli are \( K = \frac{E}{3 (1 - 2v)} \) and \( G = \frac{E}{2 (1 + v)} \), respectively.)

Shear strains are related to shear stresses and deformations by
\[ \tau_{xz} = G \gamma_{xz} = G \left( \frac{\partial}{\partial x} w(z) + \frac{\partial}{\partial z} u(x, y, z) \right) = G \frac{\partial}{\partial z} u(x, y, z) \] (4.6)

\[ \tau_{yz} = G \gamma_{yz} = G \left( \frac{\partial}{\partial y} w(z) + v \right) = G \frac{\partial}{\partial z} v(x, y, z) \]

Substituting the expressions for displacements, after proper differentiation, we obtain

\[ \tau_{xz} = G \frac{\partial}{\partial z} u(x, y, z) = \frac{8}{2} \frac{G z u_o(x, y)}{t_r} \] (4.7)

\[ \tau_{yz} = G \frac{\partial}{\partial z} v(x, y, z) = -\frac{8}{2} \frac{G z v_o(x, y)}{t_r} \]

Imposing equilibrium we have

\[ \frac{\partial \tau_{xx}}{\partial x} + \frac{\partial \tau_{zx}}{\partial z} = 0 \] (4.8)

\[ \frac{\partial \tau_{yy}}{\partial y} + \frac{\partial \tau_{zy}}{\partial z} = 0 \]

\[ \frac{\partial \tau_{xx}}{\partial x} = -\frac{\partial \tau_{zx}}{\partial z} = \frac{8}{2} \frac{G u_o(x, y)}{t_r} \] (4.9)

\[ \frac{\partial \tau_{yy}}{\partial y} = -\frac{\partial \tau_{zy}}{\partial z} = \frac{8}{2} \frac{G v_o(x, y)}{t_r} \]

Thus,

\[ u_o(x, y) = \frac{t_r}{8} G \frac{\partial \tau_{xx}}{\partial x} \Rightarrow \frac{\partial}{\partial x} u_o(x, y) = \frac{t_r}{8} \frac{\partial}{\partial x} \frac{\partial^2 \tau_{xx}}{\partial x^2} \] (4.10)

\[ v_o(x, y) = \frac{t_r}{8} G \frac{\partial \tau_{yy}}{\partial y} \Rightarrow \frac{\partial}{\partial y} v_o(x, y) = \frac{t_r}{8} \frac{\partial}{\partial y} \frac{\partial^2 \tau_{yy}}{\partial y^2} \]

Substitute the above expressions in Equation 4.5 to obtain:

\[ \frac{\partial^2 \tau_{xx}}{\partial x^2} + \frac{\partial^2 \tau_{yy}}{\partial y^2} = \frac{12}{2} \frac{G}{t_r} \left( \varepsilon_c - \frac{p}{K} \right) \Rightarrow \frac{\partial^2}{\partial x^2} p(x, y) + \frac{\partial^2}{\partial y^2} p(x, y) = \frac{12}{2} \frac{G}{t_r} \left( \varepsilon_c - \frac{p}{K} \right) \] (4.11)
The resulting partial differential equation relates the vertical strain $\varepsilon_c$ to the applied load $p(x,y)$. For circular bearings it is more convenient to use a cylindrical coordinate system $(r,\theta,z)$, instead of the cartesian system $(x,y,z)$. In such a case we have:

$$\frac{\partial^2 p}{\partial r^2}(r,\theta) + \frac{1}{r} \frac{\partial}{\partial r} p(r,\theta) + \frac{1}{r^2} \frac{\partial^2}{\partial \theta^2} p(r,\theta) = \frac{12}{t_r} \frac{G}{2} \left( \varepsilon_c - \frac{p}{K} \right) \quad (4.12)$$

$$\Rightarrow \frac{\partial^2}{\partial r^2} p(r,\theta) + \frac{1}{r} \frac{\partial}{\partial r} p(r,\theta) = \frac{12}{t_r} \frac{G}{2} \left( \varepsilon_c - \frac{p}{K} \right)$$

The bulk modulus of rubber is relatively very large, and it is therefore reasonable to assume that rubber is incompressible. Consequently, the volumetric change is assumed to be zero. The governing differential equation is reduced to a Poisson equation with boundary condition $p \left( r = R = \frac{D}{2} \right) = 0$, i.e. zero pressure at the circumference of the bearing.

$$\frac{\partial^2}{\partial r^2} p(r,\theta) + \frac{1}{r} \frac{\partial}{\partial r} p(r,\theta) = \frac{12}{t_r} \frac{G}{2} \varepsilon_c \quad (4.13)$$

The solution to the above equation is

$$p(r,\theta) = p(r) = \frac{3}{t_r} \frac{G}{2} \left( R^2 - r^2 \right) \varepsilon_c \quad (4.14)$$

Integrate the pressure over the bearing area to obtain the load $P$ as

$$P = \int_{r=0}^{R} 2\pi r p(r) \, dr = \frac{3}{2} \frac{G}{t_r} \frac{\pi R^4}{2} \varepsilon_c \quad (4.15)$$

Considering that for a circular bearing $S_1 = \frac{R^2}{t_r}$ and that $A = \pi R^2$ we can write

$$P = 6 \frac{G}{2} S_1^2 A \varepsilon_c \quad (4.16)$$

Define now the compression modulus $E_c$ as

$$E_c = E_c(\gamma) = \frac{P}{A \varepsilon_c} = 6 \frac{G}{2} S_1^2 \quad (4.17)$$

We may obtain an expression for the vertical stiffness $K_v(\gamma)$, neglecting axial deformations, and considering the shear strains of rubber as a function of the primary shape factor.
Axial deformations may be taken into account assuming that the stiffness from shear and axial strains of rubber act as springs in parallel. Then, the vertical stiffness may be expressed as

$$K_v(\gamma) = \frac{E_c(\gamma)A}{h_r} = \frac{6G S_I^2 A}{h_r}$$

(4.18)

Considering that the Poisson ratio $\nu$ for rubber is approximately 0.5, the Young’s modulus is $E = 3 \cdot G$. Therefore, the vertical stiffness $K_v$ may be expressed as a function of the effective Young’s modulus $E_c$. This takes into account the restraints against lateral displacement imposed by the steel sheets.

$$K_v = K_v(\gamma) + K_v(a) = \frac{6G S_I^2 A + E \cdot A}{h_r}$$

(4.19)

$$E_c = 6G S_I^2 + 3 \cdot G = E \cdot \left(2 \cdot S_I^2 + 1 \right)$$

(4.20)

The volumetric change can also be considered, assuming that the above stiffness and that from the volume change of rubber act as springs in parallel. The following expression for the effective modulus of elasticity $E_{cv}$ in compression considers both shear and axial deformations, as well as the compressibility of rubber.

$$\frac{1}{E_{cv}} = \frac{1}{E_c} + \frac{1}{K} = \frac{1}{6G \cdot S_I^2 + 3 \cdot G} + \frac{1}{K} \Rightarrow E_{cv} = \frac{K \cdot \left(6G \cdot S_I^2 + 3 \cdot G\right)}{6G \cdot S_I^2 + 3 \cdot G + K}$$

(4.21)

The above expressions may be used in the design of elastomeric bearings, according to the significance of each component of deformation.

Another issue which must be addressed is the prevention of buckling, which may occur due to the reduction of the critical buckling load caused by the shear flexibility of the rubber. The buckling load of a slender column with length $l$ and bending rigidity $EI$, either fixed supported or hinged at both ends, is given by the following two expressions:
\[ P_{cr} = P_E = \frac{4 \cdot \pi^2 \cdot E \cdot I}{l^2}, \quad P_{cr} = P_E = \frac{\pi^2 \cdot E \cdot I}{l^2} \]

The reduction of the buckling load due to shear flexibility, for \( P_{cr} = P_E \) is given by the following expression:

\[ P_{cr} = \frac{\sqrt{1 + 4 \cdot \frac{P_E}{P_G} - 1}}{2/P_G} \quad (4.23) \]

[Stanton and Roeder, 1982], where \( P_G = G \cdot A_s \) and \( A_s \) is the effective shear area.

Elastomeric bearings are very flexible in shear while very stiff in compression and thus the ratio \( P_E/P_G \) is large. As this ratio approaches infinity, the critical for buckling loading tends to

\[ P_{cr} = \sqrt{P_E \cdot P_G} \quad (4.24) \]

A correction coefficient may be applied to consider the non-absolutely rigid, in flexure, steel laminates. The proposed [Stanton and Roeder, 1991] correction coefficient \( f_r \) is equal to \( f_r = B_r \cdot S_1^2 \) where \( B_r \) is equal to 0.742 for a square bearing and 0.50 for a circular one. Therefore, the critical load, for a circular bearing, with design details which avoid transmission of bending moments, is determined by

\[ P_{cr} = \sqrt{\frac{2 \cdot E \cdot \pi \cdot D^4}{h_r^2 \cdot \left( \frac{4 \cdot t_r}{D} \right)^2 \cdot G \cdot \pi \cdot D^2}} \quad (4.25) \]

When Equation 4.17 is substituted above, we obtain the following expression for the buckling load which may be used for a stability check.

\[ P_{cr} = \sqrt{\frac{2 \cdot 6 \cdot G \cdot D^2 \cdot \pi \cdot D^4}{h_r^2 \cdot \left( \frac{4 \cdot t_r}{D} \right)^2 \cdot 64} \cdot 0.5 \cdot \left( \frac{D}{4 \cdot t_r} \right)^2 \cdot G \cdot \pi \cdot D^2 \cdot 64 = 0.134 \cdot G \cdot D^5}{h_r^2 \cdot t_r^2} \quad (4.26) \]

Finally, another issue of concern is the vertical load capacity of a bearing which depends on the plan area of the bearing and the internal rubber sheet thickness. An effective column may conservatively be considered to transfer axial loads, neglecting the remaining area (Figure 4.6). The effective area \( A_{eff} \) may be computed by simple geometric considerations, as a function of the plan area \( A \) when the maximum displacement \( U_{max} \) is defined.
\[
\theta = 2 \cos \left( \frac{U_{\text{max}}}{D} \right), \quad A_{\text{eff}} = \left( \frac{\theta}{180} - \frac{\sin \theta}{\pi} \right) A
\] (4.27)

As shown below, the \( U_{\text{max}}/D \) ratio must be larger than 0.40 to have an effective overlapping area of more than half of the total plan area \( A \).

**Figure 4.6:** Effective column area reduction with displacement.

In addition, even when design details do not allow the transfer of bending moments to the bearings, the distribution of compressive stresses is not uniform, due to the shear force and the resulting overturning moment which must be counterbalanced. Thus, either the maximum compressive stresses, i.e. those at the edges, must be checked or appropriate safety factors must be used for average compressive stresses.
Chapter 5

Practical Applications of Seismic Isolation

5.1 Introduction

The concept of decoupling a structure from the ground, via a mechanism on which the structure is supported, dates back to the early 1900s [Buckle and Mayes, 1990]. Initially, layers of sand or a similar material at the foundation level had been proposed, to allow slippage of the structure during a severe earthquake. The concept of seismic isolation resulted from observations of structures which were accidentally built on soft soil layers and which experienced limited damage under severe earthquakes. These structures slipped as rigid bodies on the soft soil layers. In contrast, adjacent fixed supported structures experienced severe damage or even collapsed. Since then, several seismically isolated structures have been built, either intentionally or accidentally. The fact that in most cases these structures survived severe earthquakes, while adjacent structures collapsed, established seismic isolation as a way to resist earthquakes. The first official reference to seismic isolation was made in 1906 in United States by a patent application for an earthquake-proof building mounted on rollers. In the same period, the idea of resisting the actions of ground disturbances on buildings by founding them on layers of talc for isolation purposes was proposed in England [Kelly, 1986]. In 1908 in Italy, a proposal was also made for the reconstruction of an earthquake damaged area, using rollers or layers of sand to separate each building from its foundation [Augenti and Serino, 1991]. Today, more than four hundred isolated structures have been built and the rate is still increasing.

A potential area for the application of seismic isolation is the seismic upgrading of existing structures, bridges in particular. The rehabilitation and seismic performance upgrade of historical structures of architectural importance, using seismic isolation, may be accomplished with much less modifications than for any other strengthening method. The effectiveness of seismic isolation in the retrofit of existing structures is based on the reduction of seismic forces on the superstructure instead of strengthening the structure to withstand seismic forces. Some stiffening of the superstructure may also increase the effectiveness of seismic isolation. The method is also attractive for the seismic upgrade of
existing bridges since it can be easily installed by simply replacing standard bearings, used for thermal expansion, with seismic isolation ones. The bearings are usually inserted between the top of the piers and the superstructure of the bridge. In addition, it is necessary to provide a gap to accommodate the larger relative displacements. The most effective way to isolate an existing bridge is to use lead rubber bearings, since they combine the required initial rigidity for minor loads, e.g. the horizontal forces from car breaking, flexibility to isolate the superstructure, restoring force to bring the structure to its original configuration and a dissipation mechanism, in one compact unit, while supporting the weight of the structure.

5.2 Practical Applications

5.2.1 Seismic Isolation in United States

The Foothill Communities Law and Justice Center in San Bernardino, a four story concentrically braced steel frame building, was the first seismically isolated structure built in the United States. The building, located in a high seismicity area, was mounted on 98 high damping rubber isolators, instead of lead rubber bearings which are currently the most commonly used. More than twenty buildings, either new constructions or retrofits of existing ones, and several bridges have been seismically isolated in United States. In particular, in 1989 only 5 or 6 such buildings and bridges existed while by 1993 20 buildings and 50 bridges (Figure 5.1) had been seismically isolated [Mayes, 1993].

![Figure 5.1: Increase of Seismically Isolated Buildings and Bridges in the United States.](image)

**The Salt Lake City and County Building**

The Salt Lake City and County Building is the first historic structure to have been seismically isolated. The five story, 80 by 40 m in plan building, which weighs approximately 34,000 tons, was built at the end of the nineteenth century out of unreinforced brick,
masonry and sandstone, (Figure 5.2). A twelve story, 69 m high and a 4 sq.m. clock tower, located at the center of the building, has also been built out of unreinforced masonry. Seismic isolation was provided, using 447 elastomeric bearings installed between the structure and its massive foundation system [Bailey and Allen, 1988]. Lead rubber bearings were used under the exterior walls to provide high initial rigidity in order to account for wind loads. A reinforced retaining wall around the structure provided a 30 cm gap to accommodate the relative displacement at the level of the isolation system. Seismic isolation was selected as the least disruptive of different methods, in terms of architectural aesthetics, and the most effective to minimize damage from a strong seismic excitation among three alternative retrofit schemes. The disruption of activity in the masonry superstructure was minimized due to the substantial reduction of the induced seismic forces and, consequently, the limited required stiffening. Seismic isolation was incorporated at a comparable cost to the conventional UBC strengthening scheme. The structure is in a high seismicity area, situated within two miles from the Wasatch fault. Seismic isolation reduces the induced seismic forces by a factor of six, thus eliminating the need for conventional strengthening of the building [D.I.S., 1994]. Computer simulations indicate that the structure will remain elastic under the design seismic excitation of 0.20 g peak ground acceleration and will not therefore be damaged. In contrast, the analysis indicates that it would be impossible for the superstructure to remain elastic if conventional strengthening methods were instead used.

![Figure 5.2: Salt Lake City and County Building.](image-url)
University Hospital of the University of Southern California

The U.S.C. University Hospital, shown in Figure 5.3, is the first seismically isolated building in the United States to have experienced a strong earthquake excitation, namely the Northridge earthquake in 1994. This, 8-story, 350,000 sq. ft building is also the world’s first seismically isolated hospital. The steel perimeter braced frame structure was completed in 1991 and was mounted on 149 lead rubber and elastomeric bearings. At the exterior columns of this asymmetric in plan building lead rubber bearings were used, while in the interior columns elastomeric bearings without lead plugs were used. Structural cost for both conventional and isolated options were considered and compared economically. The comparison indicated that the savings from the structural frame, when seismic isolation is incorporated, counterbalanced the additional cost of the isolation system and the additional slab at the isolation level, placed to ensure uniform distribution of the lateral forces at the isolators [Mayes et al., 1990]. The provided seismic gap around the building is 26 cm. The January 17, 1994 Northridge earthquake of 6.7 magnitude on the Richter scale resulted in 61 deaths and over 20 billion dollars in damage. The earthquake significantly damaged numerous buildings in the Los Angeles area, including 31 hospitals [D.I.S., 1994]. The U.S.C. University Hospital, located 36 km form the earthquake epicenter, remained operational with absolutely no damage, while many conventionally designed earthquake resistant hospitals suffered extensive damage. This is due to the fact that the earthquake ground accelerations, and consequently the resulting seismic forces, were reduced by 65%. Preliminary accelograms have been released by the California Strong Motion Instrumentation Program (SMIP) which demonstrate the effectiveness of seismic isolation and encourage its use. The peak free-field and peak foundation acceleration in the north-south direction, in which the strongest motions were recorded, were 0.49 and 0.37 g, respectively. The maximum floor acceleration were also recorded at the base and the roof of the building to be 0.13 and 0.21g, respectively [Moehle, 1994]. The variations of ground and floor accelerations are shown in Figure 5.3. The corresponding amplification factors of the ground motion were 0.35 and 0.57. In contrast to this building’s performance, the Los Angeles County General Hospital complex suffered 389 million dollars in damage; two wings, significantly damaged, had to be evacuated.
Seismic Retrofit of Oakland City Hall Using Seismic Isolation

The 18 story, 99m high Oakland City Hall was the first high-rise building in the United States, and is now the tallest seismically isolated building in the world [Elsesser et al., 1994]. Interest in seismic retrofit and upgrading of the steel frame building with infill walls was expressed, after the extensive damage it experienced during the Loma Prieta earthquake. In the vertical direction, the building is composed of a 3-story base, a 10-story office tower, a 2-story clock tower base and a 3-story clock tower, as shown in Figure 5.4. Due to the architectural value of the building, the preservation of its characteristics was the dominant design requirement. Thus, seismic isolation was superior to conventional strengthening techniques. In particular, a fixed-base approach would result in higher seismic forces due to the addition of the required stiffness and strength capacity, while seismic isolation reduced both floor accelerations and interstory deflections by a factor higher than 3. Shear drifts were reduced to protect the brittle facades and structural elements of the building. Stiffening of the superstructure was necessary to reduce its fixed base fundamental period and separate it from the fundamental period of the isolated building. This made the seismic isolation more effective, with the superstructure behaving as a rigid body under earthquake excitations and thus reducing the interstory deflections. Where necessary, new steel braced frames were added to improve the structural behavior. Thick concrete shear walls were also added at the lower portion of the building for stiffening purposes. In the basement, deep steel transfer trusses were used to distribute the overturn-
ing moment reactions over the whole base. Truss grids transferred the lateral loadings to
the 111 elastomeric bearings, of which 36 had lead plug cores. The dimensions of the
bearings varied from 740 to 940 mm in diameter and 400 mm in height. The isolation sys-
tem was supported on a thick concrete mat foundation. A continuous 500 mm gap had also
been provided to ensure the accommodation of large relative displacements at the isola-
tion level.

Figure 5.4: Oakland City Hall.

5.2.2 Seismic Isolation in Japan
In Japan, the first seismically isolated structure was a two story reinforced concrete dwell-
ing. It was mounted on six elastomeric bearings inserted at the foundation level under its
six columns. The first large seismically isolated building, a four story reinforced concrete
structure, was completed in 1986. Today there are more than 70 seismically isolated build-
ings and 20 bridges. The seismic isolation concept had been initially proposed in 1924
with a ball bearing system [Kitagawa, 1989]. In the late 1970’s, partial isolation was used
in more than a hundred continuous bridges, in combination with viscous dampers. The
most commonly used system in fully seismically isolated structures is one consisting of
lead rubber bearings or natural rubber bearings with auxiliary mechanical dampers. Today,
there is an increasing trend in the use of high damping rubber bearings.
Many isolated structures are fully instrumented to record earthquake excitations. Thus, due to the frequent occurrence of seismic events, important information has been collected for the performance of seismically isolated structures. Whenever the response of an isolated structure was recorded and compared with that of an adjacent fixed base structure, the comparison confirmed the superiority of seismic isolation, especially for ground motions with high acceleration levels [Kelly, 1993].

**Computer Center for Tohoku Electric Power Co.**

The six story computer center was constructed to house expensive and critical equipment of the electric power utility (Figure 5.5.a). When completed in 1990, it was the largest seismically isolated building in Japan with 10,000 m² floor space. The building was constructed on 40 high damping bearings with diameters from 0.90 to 1.20 m, each carrying vertical loads between 400 and 800 tons. The seismic isolation system cost an additional 5% of the total 20 million dollars structural cost. Its installation was very simple and the construction of the building took only one year.

![Figure 5.5: (a)Tohoku Electric Power Computer Center (b) C1-Building.](image)

**The C-1 Building**

This building, (Figure 5.5.b), houses a computer center and was completed in 1992 in Fuchi City Tokyo. It is now the largest seismically isolated building in the world. However, the new Post Office Building in Tokyo, to be seismically isolated, will have twice the area of C-1 when constructed [Skinner et al., 1993]. It is a 7-story, composite reinforced concrete-steel structure of 41.4 m maximum height above ground level, 62,800 tons total
weight and 37,850 m² total isolated superstructure floor area. Expensive and sensitive equipment is stored in the building. The building contents’ protection was the dominant design requirement. The superstructure is mounted on 68 circular lead rubber bearings of 1.1 to 1.5 m diameter, with lead core plugs of 180 to 200 mm diameter. The 1.5 m diameter bearings are the biggest lead rubber bearings used in the world [Nakagawa and Kawamura, 1991]. The isolators carry large loads since they are installed on a 15 m by 15 m grid. The fundamental period of the isolated structure varies from 1.4 to 3.0 seconds depending on the intensity of the excitation. A dynamic analysis was performed for three levels of maximum ground velocity: the standard 25 cm/sec, 50 cm/sec and 75 cm/sec for fail-safe characteristics check. The fundamental period is 3 sec and corresponds to the second level of input. The base shear due to design wind loads is 43% of the yield force of the isolation system ensuring its safety against wind loads.

5.2.3 Seismic Isolation in New Zealand

Together with Japan, New Zealand has pioneered the concept of seismic isolation. Although a type of seismic isolation was proposed in 1929 [Buckle and Mayes, 1990], practical implementation of the method began in 1973 with the construction of an isolated bridge [McKay et al, 1990]. The first seismically isolated building was constructed in 1981. The increasing number of seismically isolated structures in New Zealand indicates the wide acceptance of this innovative concept to account for earthquakes in an area of very high seismicity. The lead rubber bearing system is the most commonly used seismic isolation system, especially in bridge construction where its use clearly dominates over other systems.

William Clayton Building

The William Clayton Building in Wellington, (Figure 5.6.a), is the first seismically isolated building in New Zealand and the first in the world to incorporate lead rubber bearings. Its construction started in 1978 and was completed in 4 years [McKay et al, 1990]. The four story reinforced concrete frame building, (97 m by 40 m in plan), is mounted on 80 square lead rubber bearings. Each column of the structure is supported on a 600mm by 600mm and 207mm tall bearing with a 105mm in diameter lead cylindrical plug. The vertical load carried by each bearing varies from 1 to 2 MN. A gap around the structure and
special details of non-structural elements and internal services crossing the isolation level, allow 150 mm relative displacement in any horizontal direction. The effective fundamental period of the building is between 0.8 and 2.0 seconds, depending on the level of applied forces and corresponding displacements, while the level of the yield forces is 7% of the weight of the building. The design of this isolated structure was conservative due to the proximity of the building to the active Wellington fault and due to the then limited experience with seismic isolation.

![Figure 5.6](image)

Figure 5.6: (a) William Clayton Building (b) Wellington Central Police Station.

**Wellington Central Police Station**

The 10-story, 38m by 31m, reinforced concrete building was seismically isolated to ensure that it will remain fully operational under a major earthquake and fulfill its special functions in such emergencies (Figure 5.6.b). Three different construction schemes were initially considered [McKay et al, 1990]; seismic isolation finally dominated with 10% savings on the structural cost and with a higher earthquake resistance capacity. The conventional earthquake resisting schemes would require large structural elements to meet the rigorous strength and deflection requirements for such a special building. Seismic isolation could not be implemented with elastomeric bearings due to the height and the plan dimensions of the building. Thus, a different isolation scheme was considered. Long piles enclosed in large dimensions casings, to provide horizontal separation gap from the surrounding soil, were used combined with lead extrusion dissipators. The latter are hyster-
etic energy dissipation mechanisms based on the plastic deformations of lead during the change of its shape when forced or extruded through a hole. The piles were also necessary for the foundation of the building to reach the bedrock below the 15 m layer of weak soil deposits. The structure is located within a few hundred meters from the active Wellington fault. The adopted seismic isolation scheme not only resulted in considerable savings on the initial cost, but also ensured that the building would remain operational under a severe earthquake.

**New Zealand Parliament Buildings**

The 165 million dollars strengthening project of the New Zealand’s Parliament is the largest such project in New Zealand [Gifford, 1993]. Two historic New Zealand buildings, the Parliament House built in 1912 and the Parliamentary Library built in 1898, have to be seismically strengthened to be able to withstand major earthquakes and avoid damage and human casualties. Strengthening is imperative, considering the high seismicity of the site and the proximity of the buildings (within 400 meters) to the nearby Wellington active fault as well as to other active faults. The historic and architectural value of the two structures implies the high priority and dominance of preservation of their architectural character. Seismic isolation has been selected among several alternative strengthening design schemes, due its limited impact on architectural characteristics. Both buildings will be placed on rubber bearings, mainly high damping rubber bearings with some lead cores (LHDRB). More than 75% of the four hundred bearings are LHDRBs and it will be the first time that this hybrid isolation system, which has been described in detail in Chapter 4, will be used. The isolation system will be placed at the lower level of the five story Parliament House, while floor diaphragms and internal walls will be stiffened and columns and walls in the basement will be strengthened. The Parliamentary Library will be seismically isolated by an hybrid isolation system installed at its basement. Concrete facing will be added to the non-architectural side of selected walls. Over four hundreds bearings will be inserted in the foundation walls at the basement level to provide the required flexibility. Numerical simulations of the hybrid isolation system and the isolated structure indicate the superiority of the hybrid system over common lead rubber bearings. In addition, experimental tests have been performed with successful results, to ensure matching of the
stiffness characteristics of the analysis model and the original bearings. The fundamental periods of both buildings will be lengthened from approximately 0.50 sec to 3.5 sec, reducing substantially the seismic forces and interstory deflections [D.I.S., 1994]. Although the alternative conventional strengthening resistant option was 3% cheaper, seismic isolation was preferred, since it provided a higher earthquake resistance capacity and limited impact on the valuable architectural characteristics of both buildings.

![Figure 5.7: New Zealand Parliament Buildings.](image)

5.3 Observed Performance of Seismically Isolated Structures

Several isolated structures are instrumented with equipment to record their response and thus provide information on their performance. As the number of isolated structures increases, more information will be collected from isolated structures subjected to strong ground motions. The recorded information consistently indicates reduced accelerations above the isolation level in comparison to adjacent conventionally strengthened structures. The latest available information is from the two most recent earthquakes, the 1994 Northridge earthquake in California and the 1995 Kobe earthquake in Japan. The information on the performance of the U.S.C. subjected to the Northridge earthquake has already been presented.

Seismically Isolated Computer Center Experienced Kobe Earthquake

One year after the Northridge earthquake, an earthquake stroke the city of Kobe in Japan, resulting in a large number of casualties and severe damage. Hundreds of buildings of all types have been heavily damaged or have collapsed. However, one of the world's largest seismically isolated structure, the West Japan Computer Center of the Ministry of Post and Telecommunications, located 35 km from Kobe, did not experience any damage [Moehle., 1995]. The maximum floor accelerations were recorded and indicated the effec-
tiveness of seismic isolation through the reduced ground accelerations. The recorded maximum floor accelerations in both horizontal directions and the corresponding measured peak ground accelerations are shown in Figure 5.8:

![Figure 5.8: Recorded floor accelerations in Computer Center from Kobe earthquake.](image)

**Seismically Isolated Bridge in New Zealand Subjected to Strong Earthquake**

The Rangitaki River Bridge at Te Teko, New Zealand, experienced a 6.3 Richter earthquake motion in 1987. At the epicenter of the earthquake, located 11 km from the bridge, the recorded peak ground acceleration was 0.33g. The bridge has five twenty-meter spans supported by five meter high piers. Lead rubber bearings were installed at the piers and standard rubber bearings at the abutments. The overall performance of the isolation system of the bridge was satisfactory. However, the bridge suffered minor damage due to the slippage of one of the two rubber bearings in one of the abutments. The bearing was not properly restrained against sliding and during the excitation it moved away from its original location. This resulted in small permanent displacement of the superstructure which required temporary disruption of the bridge’s operation for repairs. Nevertheless, the seismic isolation system performance was successful. It is estimated that if the bearing was adequately restrained to avoid slippage, the superstructure would not have experienced any damage [McKay et al, 1990]. This incident demonstrates the importance of careful construction and installation of all details to ensure the effectiveness of seismic isolation.

**5.4 Configuration of Seismic Isolation System**

Prior to the selection of the plan layout of the isolation devices to avoid torsional effects, a decision must be taken on the level where seismic isolation is to be applied. Among the
several factors which must be taken into account are the site constraints, the type and nature (new construction or rehabilitation) of the structure, the ability of inspection, maintenance and replacement of the system in case of malfunction. In addition, the building internal services, such as elevators and pipes, must be considered since the location of the isolation system must not affect this operation. Finally, it is necessary to have a rigid diaphragm directly above the isolation level to ensure uniform deformations at each isolator.

![Typical seismic isolation configurations.](image)

The two most commonly used seismic isolation systems are shown in Figure 5.9. Both configurations allow continuity of services, such as the operation of elevators and access to stairways. In addition, a backup system, required as a temporary support for the building, is easily installed. The configuration with the isolation system at the basement requires an additional cost, in case a basement has not been included in the design. Adopting this configuration involves an additional cost also for the required independent retaining wall. Finally, the installation at the basement requires special care to ensure a dry area for the isolation system. The configuration with the isolation system at the ground level can easily provide access for inspection and removal of an isolator. There are several other, less attractive, configurations for isolation systems such as their insertion at any other level, typically the top of the first story. However, most of these configurations involve additional practical problems and an additional cost. One issue is the continuity of services, that of the elevator in particular, which can accounted for by constructing a cantilevered elevator hanging from the isolated superstructure. In addition, special cladding construction details may be necessary, increasing the cost of seismic isolation. Finally, fire protection of the installed system may also require an additional cost.
5.5 Practical Issues and Problems

A clearance must be ensured for the lifespan of the structure, to accommodate the maximum expected relative displacements at the isolation level, between the isolated superstructure and the foundation level or the non-isolated part of the structure, as shown in Figure 5.10.

![Section of a Typical Seismically Isolated Building](image)

**Figure 5.10**: Section of a Typical Seismically Isolated Building.

When the isolation system is installed below the ground level, a moat wall around the building is required to provide this clearance. The isolation gap must be covered to protect both the gap and the isolation system and as a safety measure against a possible accident due to the presence of this space. Exterior or interior stairs and elevators crossing the isolation level must be cantilevered to allow relative displacement of the superstructure. The utilities and internal services which intersect the isolation level, must also allow the maximum expected displacement. Water, gas, drainage pipes and cable connections must be designed with flexible joints at that level to accommodate the latter displacement. No future modification should impose any restriction or obstacle to the superstructure’s relative movement. Although there is an additional cost for the flexible joints and the special design and construction details, there are significant advantages due to the substantially reduced interstory drifts of the superstructure. These allow a more flexible design of the internal utilities which do not have to be able to withstand large deformations. Thus, the possibility of gas leakage, which in some cases may be more destructive that the actual earthquake damage, also decreases. During inspection, after a strong earthquake, any leakage at the isolation level due to over stressing of the connections must be detected.
Considering the uncertainties associated with the seismic input and the assumptions and approximations used in the design and analysis, it is reasonable to use backup systems for the case of a higher than expected structural response. Backup systems may be used for both vertical loads and horizontal displacements. They may be used to carry the vertical loads in case of bearing malfunction, by redistributing the corresponding vertical loads to temporary supports to allow the replacement of the bearing (Figure 5.11). The temporary supports must be designed to carry vertical loads and transfer them to the foundation. The difficulty involved in the replacement of a bearing depends on the configuration of the isolation system and the location of that bearing. Careful selection of the bearing connections, e.g. dowels, may simplify its removal.

**Figure 5.11:** Backup system for temporary vertical support.

In addition, constraints as shown in Figures 5.12 and 5.13, may be used as backup systems to restrict the relative displacements at the isolation level within the clearance provided around the structure. This restriction is necessary to avoid collision of the building with adjacent structures or with the moat wall in the case of a more severe earthquake than that anticipated in the design. Such collisions may result in local damage, excitation of higher modes, increased floor accelerations and consequently, building contents damage. Although the constraints will increase floor accelerations and the probability of building contents damage, in the case of an extreme event it is preferable to protect the isolation system and the superstructure from damage due to abrupt shocks. The proposed constraints may be of two types, namely stoppers (Figure 5.12) and bumpers (Figure 5.13). The backup stoppers may be composed of a lead or steel bar, of sufficiently large diameter, surrounded by a cylindrical bearing with inner dimension slightly less than twice the sum of the radius of the solid bar and the width of the seismic gap. To minimize the shock it is better to design each layer of the laminated cylindrical bearing to have a smaller inner diameter, from bottom to top. Each steel plate and adjacent rubber sheet slightly sticks out.
from the lower one by a small indent, for the constraint to be gradually applied instead of abruptly minimizing the resulting mass acceleration increase.

**Figure 5.12: Backup Stopper System.**

As shown in Figure 5.13, pieces of flexible material, e.g. elastomer, preferably with viscous properties, may be installed at particular locations in the perimeter of the building as shock absorbers. They must protrude from the edges of the building in order to be hit first and avoid the abrupt direct collision of the structure with the moat wall or adjacent structures. The buffers must be installed at locations where structural members are strong enough to avoid local damage. Additional reinforcement, in the case of reinforced concrete structures, may locally increase their strength.

**Figure 5.13: Bumpers as Shock Absorbers.**
Another issue of concern is the uplift of the bearings due to overturning moments. The bearings are not designed to carry tensile stresses and thus such stresses must be avoided or, at least, limited to levels that cannot affect the performance of the bearings. Tensile forces are induced in the bearings when the axial forces due to overturning moments are greater than the forces due to gravity loads. Careful configuration of the bearings and distribution of the gravity loads, which the higher they are the most beneficial, may eliminate the problem of uplift. Bearings at the edges of shear walls or at corner columns are usually the most vulnerable to uplift. Whenever the tensile forces cannot be reduced to acceptable levels, special connection details may be used to avoid the transmission of tensile forces. In that case, special details must be added next to the bearings to bring them back to their original configuration after uplift movements. Finally, the sequence of the installation of the isolators must be considered, to ensure its symmetry and avoid torsional response in the case of an earthquake excitation or any other strong lateral loading during construction.

5.6 Maintenance and Management of the Isolation System

The isolation system must remain operational for the whole expected lifetime of the structure under all possible environmental effects. These may corrode the metallic parts of the isolation system and deteriorate the elastomer; they may be reduced by using a protective rubber cover. The maintenance of the isolation system, and especially that of the seismic gap, must be frequent to ensure the proper function of seismic isolation. Inspections must be performed on a regular basis but also after strong seismic events. They must involve examination of metallic parts for corrosion, tightness of bolts, rubber deterioration and any other malfunction. In addition, a water drainage system must be provided to eliminate water as well as trash from the isolation system and thus reduce the possibility of its environmental deterioration. Inspections are required to ensure that no objects are placed in the gap, which may restrict the relative movement of the superstructure and, consequently, the decoupling of the structure from the ground motion. Therefore the residents of isolated buildings must be informed of the main features of the isolation system and the importance of maintenance of the seismic gap. Any arbitrary construction or modification which applies constraints to the relative movement of the superstructure must be prevented.
5.7 Code Requirements

In 1980 in the United States, the Structural Engineers Association of Northern California (SEAONC) formed a group to work on design guidelines for seismic isolation. A brief document called ‘Tentative Seismic Isolation Design Requirements’ was published in September 1986, and was later included in the SEAOC’s ‘Blue Book’ under the title “Recommended Lateral Design Requirements and Commentary”. In these provisions, emphasis was given on simple equivalent static lateral force procedures. In 1988, a subcommittee from the SEAOC seismology committee developed a design document called ‘General Requirements for the Design and Construction of Seismic Isolated Structures’. This was included in the fifth edition of the Blue Book [SEAOC, 1989] and later as an appendix to Chapter 23 of the 1991 Version of the Uniform Building Code (UBC) entitled ‘Earthquake Regulations for Seismic Isolated Structures’ [UBC, 1991]. The latest code emphasizes the use of dynamic analysis, and reduce the cases where equivalent simple static procedures can be used. Similar provisions were published in 1989 by the California Office of Statewide Health Planning and Development (OSHPD) for application to seismically isolated hospitals, entitled ‘An Acceptable Procedure For the Design and Renew of California Hospital Buildings Using Base Isolation’ [OSHPD, 1989]. In 1991, the American Association of State Highway Officials (AASHTO) adopted design requirements for application to seismically isolated bridges under the title ‘Guide Specifications For Seismic Isolation Design’ [AASHTO, 1991].

The objectives of Uniform Building Code regulations and, in general, all related to seismic isolation codes, are:

- to resist minor and moderate levels of earthquake ground motion without structural or non-structural damage as well as without building contents damage.
- to resist severe earthquakes without failure of the isolation system, with limited structural and non-structural damage and without the loss of the functionality of the building.

The code is based on two levels of earthquake motion:

- Design Basis Earthquake(DBE), which has a 10% chance to be exceeded in 50-year time period and a corresponding 475-year return period.
- Maximum Credible Earthquake(MCE), which has a 10% chance to be exceeded in
250-year time period and a corresponding 2400-year return period.

The isolation system including utilities crossing the isolation level, must be designed for the Maximum Credible Earthquake and the superstructure must remain 'essentially elastic' for the Design Basis Earthquake.

Uniform Building Code uses two design procedures:

- A static procedure which is based on simple equations to describe the maximum values of displacement and base shear. It is used for low-rise stiff structures, on stiff soils, located away from active faults.
- A dynamic procedure which uses input earthquake excitations and dynamic analysis to determine the maximum values of the response. The dynamic procedure can be either a linear response spectrum method or a time history analysis, using at least 3 pairs of horizontal ground motion. In addition, when the structure is founded on soft soil or when very close to an active fault, a site specific ground motion is necessary in the design.

An independent engineer peer review must verify the validity of the design and construction procedure of a seismically isolated structure.

In Japan, a subcommittee for Base-Isolated Structures of the Architectural Institute of Japan published a book called “Recommendation for the design of Base Isolated Buildings”, which presents some design guidelines for seismically isolated structures. This document includes design guidelines, commentary, case studies and design information which are useful and informative, but still of no legal standing. It is interesting to note that in Japan, where seismic isolation is widely used, no established codes exist and the construction of a seismically isolated structure requires special permission from the Minister of Construction. The approval of an application and the provision of the relevant permission is assigned to the Building Center of Japan (BCJ), [Kani and Manabu, 1992]. The application is evaluated by the Committee for Evaluation of Base-Isolated Structures.
Chapter 6

Structural Characteristics of Seismically Isolated Structures

6.1 Modeling of Response

The complexity of the model to be used in the various stages of design and analysis of a seismically isolated structure depends on the purpose of the model at a particular stage, and the desired accuracy of the results. The simplest model is a single degree-of-freedom (DOF) system, which assumes that the superstructure can be represented as a single mass and the stiffness is equal to that of the isolation system. This model, shown in Figure 6.1a, approximates sufficiently well the overall behavior of the isolated structure at the preliminary design stage. The response of a seismically isolated system is much more significantly dominated by its fundamental eigenmode than the response of the same system but fixed supported. Thus, single DOF modeling is actually more appropriate for a seismically isolated structure than for a fixed supported one. This may be demonstrated by examining the eigenmodes of a structure for both cases of boundary conditions.

A finer model is the “shear building”, shown in Figure 6.1b. It assumes that the overall deformation of the building may be modeled as a shear beam and that the mass of the building is lumped at the floor levels. The model is a lumped mass multi-DOF system and assumes that the beams of the structure are sufficiently rigid to prevent rotations of the masses at the floor levels about the horizontal axis. In general, it is an appropriate model

![Figure 6.1: Models for the Analysis of Isolated Structures](image-url)
for relatively stiff buildings, i.e. low- to medium-rise, which are also those for which seismic isolation is suitable. The formulation of the required matrices and the methodology for the numerical computation of the dynamic response for such a model are described in Appendices A and B. Input earthquake excitation is applied only in one direction, since each mass has only one horizontal dynamic degree of freedom.

A three dimensional (3-D) model with the masses lumped at the nodes, as shown in Figure 6.1c, is required to compute detailed local response effects and quantities, such as vertical displacements, torsional effects due to eccentricities and the horizontal distribution of seismic forces. The superstructure is modeled as a 3-D frame with the mass concentrated at the nodes. The high planar rigidity of the slab of each floor allows constraint of the corresponding degrees of freedom of each node which lies at the level of the slab, thus reducing the required computational work.

In addition to the appropriate choice of a model for the seismically isolated structure, another issue of concern, either at the preliminary design stage or at the final analysis, is the nonlinear behavior of some isolation systems. The nonlinear behavior at the isolation level may be approximately linearized. Linearizations are reasonable for some isolation systems, especially when considering the inherent uncertainties involved in the dynamic response due to an earthquake excitation. Note that some isolation systems are characterized by a highly nonlinear behavior and hysteretic dissipation of energy through cycles of loading and unloading which cannot be neglected, at least at the final analysis stage. All the above presented models may be used to consider such local non-linearities; the procedures used to account for the non-linearities for both the single-DOF system and the multi-DOF shear beam system are described in Appendix B.

6.2 Natural Modes of Vibration

The response of a linear-elastic structure with \( N \) degrees of freedom may be computed through modal analysis as the summation of the responses of \( N \) independent single DOF systems corresponding to the \( N \) normal modes of vibration of the system. The participation factors of each mode indicate the importance of the fundamental eigenmode and the actually rigid body motion of the superstructure. In addition, insight on the response of an isolated building can be gained by considering that the eigenmodes are the deformed
shapes of the oscillating building. Finally, the single-DOF approximation in the preliminary design may be verified by the high contribution of the first mode. Modal analysis and the mode superposition method are used extensively to perform linear elastic analysis of multi-DOF structures; the system of differential equations of motion may be uncoupled as a set of single-DOF differential equations, for which the response is usually available in the form of a response spectrum.

The system of equations of motion for a lumped multi-DOF system, such as that in Figure 6.1b, excited by an earthquake is:

\[ [\mathbf{M}] \ddot{\mathbf{U}}(t) + [\mathbf{C}] \dot{\mathbf{U}}(t) + [\mathbf{K}] \mathbf{U}(t) = -[\mathbf{M}] \dot{\mathbf{U}}_g(t) \]  

(6.1)

When the system is linear elastic, mode superposition is appropriate. Using this method (Appendix A) the obtained independent equations, which correspond to the eigenmodes of the structure, are of the form

\[ \ddot{\psi}_i + 2 \xi_i \omega_i \dot{\psi}_i + \omega_i^2 \psi_i = -\ddot{u}_g \]  

(6.2)

where \( i \) is the index of a particular mode.

The displacements and accelerations of the masses of the structure may be computed using mode superposition

\[ \mathbf{U}(t) = \mathbf{[\Phi] \, y(t)} = \sum_{i=1}^{N} \phi_i \mathbf{y}_i(t) = \sum_{i=1}^{N} \phi_i \Gamma_i \psi_i(t) \]  

(6.3)

\[ \ddot{\mathbf{U}}_{\text{total}}(t) = \ddot{\mathbf{U}}(t) + \ddot{\mathbf{U}}_g(t) = \sum_{i=1}^{N} \phi_i \Gamma_i \ddot{\psi}_i(t) + \dddot{U}_g(t) \]  

(6.4)

The participation factor \( \Gamma_i \) for each mode \( i \) is defined as:

\[ \Gamma_i = \frac{\phi_i^T \mathbf{M} \mathbf{e}}{\phi_i^T \mathbf{M} \phi_i} = \frac{L_i}{m_i} \]

The effective modal mass \( M_i^* \) is the percentage of mass oscillating with the \( i \)th mode, and is expressed by the following expression:
Modal analysis shows the higher contribution of the lower eigenmodes, the fundamental mode in particular; higher modes may therefore be neglected, at least in the preliminary design. This is evident from Figure 6.2 which shows the variation of the effective modal mass $M_i^*$ for the first two modes, expressed as a percentage of the total mass $M_{total}$, as the degree of isolation changes. The latter is expressed as the ratio of the fundamental mode of the isolated structure to the fixed supported superstructure.

![Figure 6.2: Variation of modal contributions with degree of isolation.](image)

The eigenproblem for the 3-story structure, discussed in Chapter 3, has been solved for two types of boundary conditions, namely fixed supported and seismically isolated. The structural characteristics of the building have been presented in Table 3.1. The results of the eigenproblem analysis are summarized in Table 6.1 for the fixed supported system and in Table 6.2 for the isolated system. The eigenmodes of the two systems are shown in Figures 6.3 and 6.4, respectively.

<table>
<thead>
<tr>
<th>Normal Mode</th>
<th>Period [sec]</th>
<th>Frequency [rad/sec]</th>
<th>Mode Shapes ([M] normalized) mass displacement x 10000</th>
<th>Effective mass [tons]</th>
<th>% of total mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.21</td>
<td>29.22</td>
<td>6.00 11.28 14.11 771.5</td>
<td>90.23</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0.08</td>
<td>79.35</td>
<td>-13.65 5.52 -11.41 73.40</td>
<td>8.58</td>
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</tr>
<tr>
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<td>0.06</td>
<td>113.03</td>
<td>10.28 -13.90 6.92 10.1</td>
<td>1.18</td>
<td></td>
</tr>
</tbody>
</table>

Table 6.1: Natural characteristics and effective modal mass: fixed supported system
The first and usually most essential mode of the isolated structure is characterized by the rigid body motion of the superstructure, as shown in Figure 6.4, and the deformation of the isolation system. Inertia forces due to the seismic excitation are uniformly distributed which in turn implies lower overturning moments. In contrast, in the case of a fixed supported structure, the distribution of these forces increases linearly with height. The fundamental eigenmode is essentially independent of the stiffness and damping properties of the superstructure, considering the actually rigid body motion of the latter due to the large difference between its stiffness and that of the isolation system. Therefore, to a first approximation, a seismically isolated structure may be represented by a single DOF model with the mass of the whole structure above the isolation level and the stiffness and damping properties of the isolation system.

Table 6.2: Natural characteristics and effective modal mass: isolated system

<table>
<thead>
<tr>
<th>Normal Mode</th>
<th>Period [sec]</th>
<th>Frequency [rad/sec]</th>
<th>Mode Shapes ([M] normalized) mass displacement x 10000</th>
<th>Effective mass[tons]</th>
<th>% of total mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.26</td>
<td>4.97</td>
<td>9.00 9.15 9.26 9.31</td>
<td>1189.7783</td>
<td>99.9817</td>
</tr>
<tr>
<td>2</td>
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<td>48.12</td>
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<td>0.0173</td>
</tr>
<tr>
<td>3</td>
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<td>89.61</td>
<td>8.49 -9.57 -8.80 9.87</td>
<td>0.0101</td>
<td>0.0008</td>
</tr>
<tr>
<td>4</td>
<td>0.05</td>
<td>115.66</td>
<td>-4.49 11.50 -12.44 5.78</td>
<td>0.0010</td>
<td>&lt;0.0001</td>
</tr>
</tbody>
</table>
Although the fundamental mode dominates the response of a seismically isolated structure, the participation of higher modes increases with the viscous damping of the isolation system. Considering the shape of the displacement response spectrum (Figure 2.4), the displacements, those of the isolated system in particular, are completely dominated by the fundamental mode. However, the acceleration response spectrum (Figure 2.5) indicates that higher modes have higher spectral values than the fundamental mode of the isolated system, and thus their contribution may be increasingly significant. Despite this fact, though, the small participation factors of these higher modes suppress their potential importance.

6.3 Linear Systems

Some isolation systems may be considered linear-elastic with viscous damping properties, while others are inherently non-linear inelastic with a hysteretic energy dissipation mechanism. The most commonly used linear isolation systems are the natural rubber bearings (NRB) combined with an auxiliary viscous damper, and the high damping rubber bearings (HDRB). Although the latter involve hysteretic dissipation of energy and a nonlinear behavior, they may still be treated as linear, using equivalent stiffness and damping properties. Linear systems have a constant, low isolation stiffness (Figure 6.5a) which makes them vulnerable to dynamic loadings with dominant frequencies close to the fundamental frequency of the isolated structure. The viscous energy dissipation mechanism of linear systems is constant and continuously available for both low and large strains. The differ-
The differential equation of motion of a linear, viscously damped single-DOF system is

\[ M \ddot{U} + C \dot{U} + K U = -m \ddot{U}_g \]

which can alternatively be written as

\[ \ddot{U} + 2 \xi \omega \dot{U} + \omega^2 U = -\ddot{U}_g \]

with \( \xi = \frac{C}{C_{cr}} = \frac{C}{2M \omega} \) the damping coefficient,

\[ \omega = \sqrt{\frac{K}{M}} \]

the frequency of an undamped single degree of freedom system, and

\[ \omega_d = \sqrt{\left( \frac{K}{M} \right)^2 - \left( \frac{C}{2M} \right)^2} = \omega \sqrt{1 - \xi^2} \]

the frequency of a damped single DOF system.

The shear stiffness of an elastomeric bearing may be computed by the following formula, which considers only shear deformations of the rubber and neglects bending deformations as well as the influence of axial (normal) forces. The influence of the axial load is negligible when the axial stresses are relatively low, e.g. well below the critical for buckling value. Shear modulus of typical elastomeric bearings are of the order of 0.7 to 1.0 MPa.

\[ K = \frac{G_r A_r}{h_r} \]  \hspace{1cm} (6.6)

The viscously dissipated energy depends on the velocity of the mass and is characterized by smooth elliptical hysteresis loops, as depicted in Figure 6.5b.

\[ \text{Figure 6.5: Structural characteristics of a linearly isolated system.} \]
As damping increases, the displacements of the structure are reduced; the accelerations decrease until the damping ratio reaches approximately 20-30%, and then increase only slightly, for higher damping ratios. The occurrence of this phenomenon is demonstrated here through a dynamic time history analysis of a linearly isolated 4-mass model. The structural characteristics of the model are shown in Figure 6.6.a. The introduced damping ranges from 2 to 60% for the fundamental mode, and 2% for the other modes. The dynamic analysis is performed for two ground accelograms, the El-Centro and Olympia earthquakes, scaled to have PGA equal to 0.50 g. The maximum relative displacement (Figure 6.6.b) and the normalized mass acceleration (Figure 6.6.b) of the structure are plotted as a function of the varying damping ratio.

![Graphs showing structural characteristics, maximum displacements, and normalized mass accelerations](image)

**Figure 6.6:** Effect of viscous damping on a linear isolation system.

### 6.4 Bilinear Systems

The most commonly used isolation systems that exhibit bilinear behavior are the lead rubber bearings (LRB) and the natural rubber bearings (NRB) combined with an auxiliary lead or steel damper. Bilinear systems have a high initial rigidity which is very useful for service lateral loads.

Bilinearly isolated structures typically have a more efficient and reliable energy dissipation mechanism. However, this mechanism is not always available since it is incorporated only when a yield force is exceeded. In addition, highly nonlinear isolation systems may result in higher mode effects which may, in turn, significantly increase the masses’ accelerations and consequently the inertia forces induced in the structure and its contents.
This situation is worth considering when the prevention of building contents damage is the dominant design requirement. The contribution of higher modes increases with the sudden changes of the isolation system stiffness during reversed loading. The procedure used to perform a step-by-step time history analysis considering the local non-linearities is described in appendix B.

6.5 Comparative Study of a Linearly and a Bilinearly Isolated Structure

The response of a typical three-story structure, mounted on a linear, a bilinear isolation systems and a hybrid isolation system is investigated. Linear isolation is typically implemented using high damping rubber bearings. The most commonly used bilinear system is one containing lead rubber bearings. A linearly isolated structure has higher displacements than a bilinear one because the response of the latter is dominated by the prior to yielding high rigidity. This generates higher mode effects due to the coincidence of the fundamental frequency of the initially stiff isolated structure and the superstructure. A hybrid isolation system, combining high damping rubber with lead plugs (LHDRB), is an alternative and more efficient system. The characteristics of this system have been discussed in detail in Chapter 4.

Tables 6.3-6.5 summarize the characteristics of a structure which have been isolated with a linear, a bilinear and a hybrid isolation system, respectively.

<table>
<thead>
<tr>
<th>Level</th>
<th>Masses [Tons]</th>
<th>Stiffness [MN/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Isolation</td>
<td>335</td>
<td>50</td>
</tr>
<tr>
<td>1</td>
<td>305</td>
<td>1250</td>
</tr>
<tr>
<td>2</td>
<td>285</td>
<td>1125</td>
</tr>
<tr>
<td>3</td>
<td>265</td>
<td>1125</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mode</th>
<th>Period [sec]</th>
<th>Frequency [rad/sec]</th>
<th>Effective mass [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.98</td>
<td>6.37</td>
<td>99.948</td>
</tr>
<tr>
<td>2</td>
<td>0.12</td>
<td>48.38</td>
<td>0.048</td>
</tr>
<tr>
<td>3</td>
<td>0.07</td>
<td>89.69</td>
<td>0.002</td>
</tr>
<tr>
<td>4</td>
<td>0.05</td>
<td>115.68</td>
<td>&lt;0.001</td>
</tr>
</tbody>
</table>

Table 6.3: Structural characteristics of the linearly isolated structure.
Table 6.4: Structural characteristics of the bilinearly isolated structure (Fy/W=0.05).

<table>
<thead>
<tr>
<th>Level</th>
<th>Masses</th>
<th>Stiffness</th>
<th>Mode</th>
<th>Period</th>
<th>Frequency</th>
<th>Effective mass [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Initial - Post</td>
<td>a/a</td>
<td>Initial - Post</td>
<td>Initial - Post</td>
<td>Initial - Post</td>
</tr>
<tr>
<td>Isol.</td>
<td>335</td>
<td>250</td>
<td>25</td>
<td>1</td>
<td>0.47</td>
<td>1.38</td>
</tr>
<tr>
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<td>305</td>
<td>1250</td>
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<td>2</td>
<td>0.12</td>
<td>0.13</td>
</tr>
<tr>
<td>2</td>
<td>285</td>
<td>1125</td>
<td></td>
<td>3</td>
<td>0.07</td>
<td>0.07</td>
</tr>
<tr>
<td>3</td>
<td>265</td>
<td>1125</td>
<td></td>
<td>4</td>
<td>0.05</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Table 6.5: Structural characteristics of the structure with hybrid isolation (Fy/W=0.03).

<table>
<thead>
<tr>
<th>Level</th>
<th>Masses</th>
<th>Stiffness</th>
<th>Mode</th>
<th>Period</th>
<th>Frequency</th>
<th>Effective mass [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Initial - Post</td>
<td>a/a</td>
<td>Initial - Post</td>
<td>Initial - Post</td>
<td>Initial - Post</td>
</tr>
<tr>
<td>Isol.</td>
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<td>150</td>
<td>25</td>
<td>1</td>
<td>0.58</td>
<td>1.38</td>
</tr>
<tr>
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<td>305</td>
<td>1250</td>
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<td>2</td>
<td>0.13</td>
<td>0.13</td>
</tr>
<tr>
<td>2</td>
<td>285</td>
<td>1125</td>
<td></td>
<td>3</td>
<td>0.07</td>
<td>0.07</td>
</tr>
<tr>
<td>3</td>
<td>265</td>
<td>1125</td>
<td></td>
<td>4</td>
<td>0.05</td>
<td>0.05</td>
</tr>
</tbody>
</table>

The same structure is analyzed for these cases under an earthquake excitation. The time history analysis is performed using the El-Centro earthquake scaled for two levels of peak ground acceleration, pga equal to 0.50 and 0.10 g, respectively, in order to consider the effectiveness of each system under both a severe and a small to moderate earthquake. The damping for the linear system is assumed to be 15% for the first mode and 2% for higher modes, while for the bilinear system, the damping matrix is constructed assuming Rayleigh damping with 2% damping ratios. The damping matrix for the hybrid system is constructed for damping ratios 10% and 2% for the lowest and the highest frequencies, respectively. The yield force of the bilinear system is 5% of the total weight of the structure while that of the hybrid system is 3%. Accordingly, the prior to yielding stiffness of the bilinear system is ten times the post-yield stiffness, while that of the hybrid system is six times the post-yield stiffness.
The maximum values for the relative displacement at the isolation level, interstory deflection, floor acceleration, and base shear force for the three isolation systems and for both pga=0.10g and pga=0.50g are given in Tables 6.6 and 6.7, respectively.

<table>
<thead>
<tr>
<th>Isolation type</th>
<th>Base displacement [cm]</th>
<th>Interstory deflections [cm]</th>
<th>Floor accelerations [m/sec^2]</th>
<th>Base shear Force [MN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear</td>
<td>1.87</td>
<td>0.056</td>
<td>0.87</td>
<td>0.94</td>
</tr>
<tr>
<td>Bilinear</td>
<td>1.27</td>
<td>0.067</td>
<td>1.46</td>
<td>0.84</td>
</tr>
<tr>
<td>Hybrid</td>
<td>1.12</td>
<td>0.041</td>
<td>0.85</td>
<td>0.57</td>
</tr>
</tbody>
</table>

Table 6.6: Maximum response values for El-Centro scaled to pga=0.10g.

<table>
<thead>
<tr>
<th>Isolation type</th>
<th>Base displacement [cm]</th>
<th>Interstory deflections [cm]</th>
<th>Floor accelerations [m/sec^2]</th>
<th>Base shear Force [MN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear</td>
<td>9.37</td>
<td>0.278</td>
<td>4.33</td>
<td>4.69</td>
</tr>
<tr>
<td>Bilinear</td>
<td>10.0</td>
<td>0.206</td>
<td>3.66</td>
<td>3.03</td>
</tr>
<tr>
<td>Hybrid</td>
<td>8.80</td>
<td>0.163</td>
<td>2.79</td>
<td>2.49</td>
</tr>
</tbody>
</table>

Table 6.7: Maximum response values for El-Centro scaled to pga=0.50g.

The story shear force-interstory deflections for all isolation systems for both levels of earthquake excitation are shown in Figures 6.8 and 6.9.

Figure 6.7: Base shear force - Relative displacement diagrams for pga=0.10g.
The higher mode effects may be reduced by designing the isolation system to have its initial frequency away from the higher frequencies of the isolated structure in order to avoid resonance. The fundamental frequency of the superstructure with fixed supports may be used as an approximate upper bound from which the initial stiffness of the isolated structure must differ significantly. Initial rigidity is necessary to prevent displacements under other lateral loads, e.g. wind loads, and thus to avoid the discomfort of the structure’s inhabitants. An alternative to the provision of high initial stiffness is to incorporate an additional energy dissipation mechanism in order to suppress higher mode effects. The hybrid isolation system (LHDRB) provides relatively reduced initial rigidity and an energy dissipation mechanism for minor lateral loads. The LHDRB system reduces higher mode acceleration responses which occur when lead rubber bearings are used. In contrast to lead rubber bearings which dissipate energy only once the isolation system yield force has been exceeded, the system provides a continuous energy dissipation mechanism.

### 6.6 Linearization of Bilinear Behavior

The bilinear behavior of some highly nonlinear isolation systems, e.g. lead rubber bearings (LRBs), may be linearized for preliminary design purposes. Figure 6.10 shows the typical force-displacement relation of a bilinear system. Non-linearities due to the hysteretic dissipation of energy may be linearized through the following damping linearization procedure. This is based on the determination of equivalent viscous damping to represent the hysteretic energy dissipation.
6.6.1 Effective Stiffness

The initial elastic stiffness provides the necessary initial rigidity for minor lateral loads up to the level of design yield force $F_y$ of the lead plugs. The initial and the post-yield stiffness may be computed considering shear deformations and assuming that there are no bending deformations and effects of axial forces:

$$K_{el} = \frac{G_l A_l}{h_{tot}} + \frac{G_r A_r}{h_r} \quad K_{pl} = \frac{G_r A_r}{h_r} \quad (6.7)$$

where $G_l$ and $G_r$ are the lead and rubber shear moduli, $A_l$ and $A_r$, the lead and rubber areas, and $h_l$ and $h_r$, the lead and (total) rubber heights, respectively.

An effective stiffness $K_{eff}$ and an effective damping coefficient $\xi_{eff}$ are very often used to describe the bilinear behavior of the isolation system and allow an equivalent linear elastic analysis which is useful at least in the preliminary design stage. However, both effective parameters depend on the earthquake excitation and its severity; their determination, therefore, involves several uncertainties.

Figure 6.9: Force-displacement bilinear relation and its linearization.

The effective stiffness is defined as the secant modulus joining the extreme positive and negative displacements.

$$K_{eff} = \frac{F_y + K_p \left( u_{max} - u_y \right)}{u_{max}} = \frac{F_y (1 - K_p/K_e)}{u_{max}} + K_p \frac{u_{max}}{u_{max}} \quad (6.8)$$

Subsequently, the effective period is given by the expression
\[ T_{\text{eff}} = 2 \pi \sqrt{\frac{M}{K_{\text{eff}}}} = 2 \pi \sqrt{\frac{M u_{\text{max}}}{F_y \left(1 - K_p/K_e\right) + K_p u_{\text{max}}}} \] (6.9)

### 6.6.2 Effective Damping

Effective viscous damping may be defined on an energy equivalence base so as to represent the actual hysteretically dissipated energy. The equivalent damping coefficient \(\xi_{\text{eff}}\) may be obtained by equating the hysteretically dissipated energy over a complete cycle to the viscously dissipated energy. However, energy dissipation through viscous damping is proportional to velocity, while hysteretic energy dissipation is related to the offsets of reversed cycles and is equal to the kinetic energy converted to thermal energy and dissipated.

A single DOF system with viscous damping \(C_{\text{eff}}\) and stiffness \(K_{\text{eff}}\), shown in Figure 6.10.a, is used to determine an equivalent linear elastic system which in turn represents the hysteretic energy dissipation.

![Diagram of a single DOF system](image)

Figure 6.10: Viscously damped linear elastic single DOF system.

Assuming that the displacements for a free vibration of the mass \(M\) are of the form \(u(t) = u_o \sin(\omega t)\), the velocity is given by: \(\dot{u}(t) = u_o \omega \cos(\omega t)\), where \(\omega\) is the frequency of oscillation. The total force (damping and elastic), is shown in Figure 6.10.c; it is equal to

\[ P_{\text{return}} = C_{\text{eff}} \omega u_o \cos(\omega t) + K_{\text{eff}} u_o \sin(\omega t) \] (6.10)

The loss factor \(h\) is defined as the ratio

\[ h = \frac{(\text{Area of hysteresis loop})}{4 \pi (\text{Elastic energy area})} = \frac{E_h}{4 \pi E_e} \] (6.11)
and may be determined experimentally by the above-mentioned oscillations.

The hysteretically dissipated energy per cycle is

$$E_d = \int_{0}^{T} P \frac{du}{dt} dt .$$

Substituting the expressions for the displacement and velocity we obtain

$$E_d = \pi \int_{0}^{\frac{2\pi}{\omega}} C_{eff} \omega u_{o} \cos (\omega t) + K_{eff} u_{o} \sin (\omega t) \omega u_{o} \cos (\omega t) dt \quad (6.12)$$

Integration of Equation 6.12 gives the area of the hysteresis loop per cycle

$$E_d = \pi C_{eff} \omega u_{o}^2 .$$

The maximum elastic potential energy is equal to

$$E_e = 0.50 K_{eff} u_{o}^2 .$$

Substituting in Equation 6.11, and considering that the natural frequency of the system is equal to $\omega_N = \sqrt{K_{eff}/M}$, the loss ratio $h$ may be related to the damping ratio $\xi_{eff} = C_{eff}/C_{cr} = C_{eff}/2\omega_N M$, as follows:

$$h = \frac{\pi C_{eff} \omega u_{o}^2}{2 \pi K_{eff} u_{o}^2} = \frac{C_{eff} \omega}{2 K_{eff}} = \frac{2 \xi_{eff} \omega_N M \omega}{2 \omega_N^2 M} = \xi_{eff} \left( \frac{\omega}{\omega_N} \right) \quad (6.13)$$

The frequency of oscillation for the free response is approximately equal to the natural frequency of the system, i.e. $\omega = \omega_N$, and thus the damping ratio $\xi_{eff}$ is, in turn, equal to the loss factor $h$. The latter is computed experimentally by free vibration tests and is of the order of 10-15% for high damping rubber.

The above ‘linearization’ of damping is based on several assumptions which may be incorrect in practice. The substitution of hysteretic with equivalent viscous damping, for highly nonlinear systems, involves several uncertainties. First, the hysteretic damping depends on the displacement magnitude since it is equal to the area of cycles, and thus on the input excitation, while viscous damping depends on velocity. In addition, the linearization is based on periodic complete cycles of oscillation while the expected irregular response due to an earthquake excitation contains many short cycles with little hysteretic energy dissipation. Small incomplete cycle viscous damping may overestimate the actual energy dissipation. The experimental results are for free vibration which is not what an isolated structure undergoes. However, the use of an equivalent viscous damping based on
an approximate energy equivalence allows a rough approximation of the isolated structure response for preliminary design purposes.

6.6.3 Study on the Effectiveness of Bilinear Behavior Linearization

We want to investigate the possibility of using an equivalent viscously damped linear single DOF system to represent the actual nonlinear behavior of a structure seismically isolated structure with lead rubber bearings. The results indicate that the response is affected by several factors such as the characteristics and intensity of the earthquake, as well as by the system characteristics. This shows the necessity of nonlinear dynamic analysis of the isolated structure at the final design and analysis stages.

A single-DOF system with a bilinear behavior is used in the study, assuming it is subjected to six different earthquake excitations (Figure 3.2), scaled for different levels of peak ground acceleration (pga), ranging from 0.10 to 0.70 g.

The system has the following characteristics:

- Mass, \( M = 1672.0 \) tons
- Elastic stiffness, \( K_e = 165.0 \) MN/m
- Post-yield stiffness, \( K_{pl} = 16.5 \) MN/m

Using the Newmark step-by-step direct integration method, described in Appendix B, the response of several single-DOF systems with the above characteristics, but with different \( F_y/W \) ratios, are calculated. The maximum relative displacements for these systems for all six excitations are shown in Figure 6.11a.

The equivalent viscous damping ratio \( \xi_{eq} \), required to ensure same maximum relative displacements \( U_{max} \) as that of an equivalent viscously damped single-DOF system with the same mass and equivalent effective stiffness \( K_{eff} \), varies with the magnitude of the seismic excitation and the ratio \( F_y/W \). This variation for six different earthquakes is shown in Figure 6.11b.
Figure 6.11: Maximum relative displacements and equivalent viscous damping ratio $\xi$.

We notice from the results obtained from several numerical simulations that the required damping ratio $\xi_{eq}$ of an equivalent linear system, varies considerably. In general, the required viscous damping, for an equivalent linear system, is higher under weaker earthquake excitations than under stronger ones, in the range of very low yielding force levels due to the dominance of the prior to yielding high initial stiffness of the isolation system. Systems with high levels of yield force are more effective, considering the displacements under stronger earthquake excitations. This is indicated by the higher viscous damping required by an equivalent linear system to have the same maximum relative displacement. However, systems with high level of yield force do not have an energy dissipation mechanism for lower loads, since the hysteretic damping is activated once the isolation system has yielded. In addition, the higher modes may be excited when the degree of non-linearity is also high. Finally, for an intermediate level of yield force, the required viscous damping and the overall effectiveness of the isolation varies considerably according to both structural and earthquake characteristics. In Figure 6.11 we note the reduction of the effectiveness of seismic isolation for very low $F/W$ ratios, due to the lim-
ited dissipation of energy provided. For a specific earthquake excitation, an optimum value of the yield force $F_y$ can be computed to minimize a particular response quantity. When the quantity of interest is the displacement, the optimum point increases with the severity of the excitation (Figure 6.11.b). Given the uncertainties and the variations of the characteristics and severity of the expected earthquakes, it is impossible to determine a single optimum damping point.

For a typical lead rubber bearing isolation system, i.e. $F_y/W=0.05$ and $K_e/K_{pl}=10$, the damping ratio is approximately 10-30%. The results show that an equivalent linear elastic, viscously damped system cannot accurately model the actual bilinear behavior of an isolated structure with LRBs. The maximum relative displacements have been computed for a wide range of single-DOF systems with bilinear behavior, assuming for $F_y/W$ the two common cases of 0.02 and 0.05. The previously presented earthquake excitations are used, scaled by the pseudovelocity $S_v$, given by the ATC3-06 design spectrum for peak ground acceleration (pga), ranging from 0.30g to 0.60g. The ATC3-06 is an elastic design spectrum generated in 1984 by [Applied Technology Council, 1984], using records which were scaled to have pga equal to 0.40g.

The results for the normalized maximum relative displacements $U_{max}$ are compared in Figure 6.12.a with the corresponding values of $(S_d/pga)$ used for a conventionally designed earthquake-resistant structure, for $\xi$ equal to 5% and 10%, respectively. Also, the resulting maximum accelerations of the bilinear model are compared to the corresponding values of a viscously damped system with $\xi$ equal to 5% and 10%.
We conclude that an elastic design spectrum may be used at the preliminary design to take into account the constraint imposed by the maximum relative displacement accommodated for a specific structure. In addition, the results of several linear and non-linear studies indicate that the variation between the linear and non-linear analysis is in many cases less pronounced than the variations among non-linear analyses for different earthquake excitations. Considering the uncertainties associated with the expected earthquake excitations and their specific characteristics, as well as the difficulty involved in a non-linear analysis, we may conclude that the results of a linear analysis are in many cases acceptable. However, higher mode effects and the resulting increased accelerations of the masses cannot be observed using an equivalent linear-elastic model. A nonlinear analysis is required to investigate such effects, in particular for structures with acceleration requirements.

### 6.7 Effect of the Degree of Non-Linearity

The degree of non-linearity of bilinear systems may be defined as the ratio of the initial stiffness $K_{ei}$ to the post-yield stiffness $K_{pl}$. High degree of non-linearity may induce higher
mode effects and result in large floor accelerations. The degree of non-linearity is also related to the yield force $F_y$ and consequently to the ratio $F_y/W$. Its influence on the system's response is studied using the 4-mass system discussed in Section 6.5, the performance of which is compared to that of a typical lead rubber bearing system with a $K_{el}/K_{pl}$ ratio equal to 10.0 and a yield force equal to 5% of the weight of the building. Both $K_{el}/K_{pl}$ and $F_y/W$ vary linearly between 1%-10%, and 2-20, respectively. Three different earthquake excitations are used in the analysis, scaled for 0.3, 0.5 and 0.7 g. The structural characteristics of the superstructure are given in Section 6.5. The post-yield stiffness of the isolation system is 25 MN/m. The maximum relative displacements at the isolation level for various degrees of non-linearities, normalized with the corresponding maximum relative displacement for the investigated system with $K_{el}/K_{pl}=10$ and $F_y/W=0.05$ are shown in Figure 6.13. The linear variation of the ratios $K_{el}/K_{pl}$ and $F_y/W$ is plotted on the horizontal axis. We note that they decrease as the non-linearity of the isolation system increases, keeping the post-yield stiffness of the isolation system constant, due to the dominance of the prior to yielding high stiffness $K_{el}$. As the severity of the earthquake decreases, the influence of the non-linearity on the reduction of the maximum relative displacement is more pronounced. This is due to the fact that the effective stiffness of the isolation system is more influenced by the high initial stiffness than by the post-yield stiffness.

![Figure 6.13: Normalized maximum relative displacement at isolation level.](image)

Floor accelerations increase with the increase of the degree of non-linearity, due to the higher effective stiffness of corresponding non-linear systems and due to the increased
effective stiffness and higher mode effects excited by the sudden stiffness changes. This is illustrated in Figure 6.14, where the maximum normalized floor accelerations, are plotted:

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure6_14.png}
\caption{Maximum floor accelerations normalized with peak ground acceleration.}
\end{figure}

A compromise must be made to keep both the maximum relative displacement at the isolation level and the maximum floor accelerations within acceptable limits. The reduction of the former is associated to the maximum displacement to be accommodated by the gap around the structure and the connections of external services; the latter must be controlled to avoid damage of the contents of the structure. The specific project characteristics and design requirements determine which of the two dominates the design.

As previously mentioned, a hybrid model may reduce both floor accelerations and suppress the relative displacement at the isolation level, through additional viscous damping. This is illustrated in Figure 6.15 where the viscous damping of the system is increased linearly from 2% at \( K_{ef}/K_{pl}=10 \), to 10% at \( K_{ef}/K_{pl}=6 \). Additional viscous damping may be provided either by the inherent damping properties of high damping rubber of an elastomeric bearing, or via the incorporation of an actual viscous damper.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure6_15.png}
\caption{Normalized maximum relative displacements (additional viscous damping).}
\end{figure}
The increase of the relative displacements at the isolation level with the decrease of the degree of non-linearity, may be suppressed, as illustrated in Figure 6.15. Also reductions of floor accelerations may be achieved by decreasing the non-linearity of the isolation system, as shown in Figure 6.16.

![Figure 6.16: Maximum floor accelerations normalized with pga (additional viscous damping).](image)

It is evident from the above discussion that when the reduction of floor accelerations is an important design parameter, the degree of non-linearity must be reduced as well as \( F_y/W \). The latter is defined by the minor lateral loads for a specific project. Thus, using a particular \( F_y/W \) ratio, a proper \( K_e/K_{pl} \) ratio must be selected. When building contents protection dominates the design requirements, a lower degree of non-linearity must be used, combined with additional viscous damping to suppress the large relative displacement at the isolation level. The use of highly nonlinear isolation systems may result in increased higher mode effects which may significantly increase mass accelerations and consequently the inertia forces induced in the structure and its contents. The contribution of higher modes is enhanced by sudden changes of the isolation system stiffness during reversed loads. Higher mode effects may be reduced by designing the isolation system to have its initial, i.e. prior to yielding frequency away from the higher frequencies, to avoid resonance. These, together with the resulting increased mass accelerations, cannot be observed using an equivalent linear-elastic model.

### 6.8 Effect of Superstructure Stiffness

The stiffness of the superstructure and in particular its relative difference from the stiffness of the isolation system significantly affect the efficiency of seismic isolation. The
degree of isolation, defined as the ratio of the period of the isolated structure to the period of the superstructure when fixed supported, is directly related to the ratio of the corresponding stiffness for both types of boundary conditions. In general, as the stiffness of the superstructure increases, the large relative displacement at the isolation level slightly increases, due to the increase of the fundamental period. The interstory displacements are substantially reduced due to the increased stiffness of the superstructure; floor accelerations are reduced due to the increase of the fundamental period. Finally, the base shear forces slightly increase due to the increase of the relative displacement at the isolation level.

A 3-DOF system mounted on a linear isolation system, with characteristics as given in Figure 6.17, is analyzed under the El-Centro earthquake, to illustrate the above effects as the stiffness of the superstructure changes.

<table>
<thead>
<tr>
<th>Level</th>
<th>Masses [Tons]</th>
<th>Stiffness [MN/m]</th>
<th>Damping ratio ξ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Isolation</td>
<td>335</td>
<td>50</td>
<td>15</td>
</tr>
<tr>
<td>1</td>
<td>305</td>
<td>varying from 500 to 5000</td>
<td>2</td>
</tr>
<tr>
<td>2</td>
<td>285</td>
<td>varying from 500 to 5000</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>265</td>
<td>varying from 500 to 5000</td>
<td>2</td>
</tr>
</tbody>
</table>

**Figure 6.17: Structural characteristics of the isolated structure**

The ratio of the superstructure stiffness $K_s$ to the isolation system stiffness $K_i$ varies from 10 to 100. The maximum values of the quantities of interest are given in Figures 6.18 and 6.19. In particular, the change of the fundamental period of the isolated structure as a function of $K_s/K_i$, normalized by the value of the fundamental period for $K_s/K_i=10$, is shown in Figure 6.18.a. The fundamental period of the multi-DOF system approaches the value of the period of a single-DOF system when the superstructure is assumed infinitely rigid, and is indicated with a dashed line. The maximum values of relative displacements at the isolation level and the interstory deflections, normalized with the corresponding value at $K_s = 10 \cdot K_i$, are also shown. The relative displacement at the isolation level
increases by less than 5% when the superstructure stiffness becomes ten times its initial value, while the interstory displacements substantially decrease by 90%.

Figure 6.18: Effect of superstructure stiffness on displacements.

The maximum floor accelerations and base shear force, with varying degree of isolation, are shown in Figure 6.19. They decrease by approximately 10%, while the base shear forces increase by less than 5%.

Figure 6.19: Effect of superstructure stiffness on floor accelerations and base shear forces.

It is evident that the superstructure must be as stiff as possible to ensure a better performance when seismic isolation is used. The larger the difference between the fundamental frequency of the seismically isolated structure and its fundamental period when fixed supported, the better the response of the isolated structure.

6.9 Ductility of Isolated Structures

The seismic loads considered in conventional earthquake-resistant design are substantially reduced using the reduction factors provided in the codes. The reduction of the seismic
forces is mainly due to the inelastic deformations which dissipate energy. The superstructure of an isolated structure may also have some inelastic deformations under a very strong excitation. In addition, the softening of the superstructure from the inelastic deformations lengthens the period of the superstructure. In contrast, in the case of a seismically isolated structure under a very strong earthquake, although energy is dissipated through inelastic deformations when the superstructure begins to yield, the softening of the superstructure has negative effects on the efficiency of seismic isolation. The ductility capacity of a seismically isolated structure is much lower than that of a fixed supported one. This difference is reflected on the different reduction factors adopted for conventionally fixed supported and seismically isolated structures. Appropriate factors for the latter are under investigation and no agreement on their values has been reached. My opinion is that the reduction factors may be used only for the maximum credible earthquake; since no damage is accepted for the design earthquake, no reduction factors must be used.
Chapter 7

A Preliminary Design Methodology

7.1 Introduction

A preliminary design methodology for bilinear elastomeric isolation systems, those with lead plugs in particular, is presented here. The purpose of the methodology is to provide only a preliminary guidance for the design of an isolated structure, which involves several dependent on each other parameters. A trial and error procedure with adjustments and corrections may be required to finally obtain an adequate design of the isolation system. Although a creative procedure, the design of an isolated structure is complicated and requires significant experience and extensive engineering judgement. The dominant design parameters of the proposed methodology are the level of yield force $F_y$ of the isolation system, the maximum allowable relative displacement at the isolation level $U_{max}$, the location and dimensions of the isolators, the non-linearity ratio $K_e/K_{pl}$ and the distribution of seismic forces at the isolators.

The level of the yield force $F_y$ of the isolation system is determined by the minor lateral loads to be sustained prior to yielding, and is usually expressed as a percentage of the weight of the structure. Practical restrictions constrain the level of maximum allowable relative displacement at the isolation level $U_{max}$ and consequently the degree of isolation. The ratio $K_e/K_{pl}$ is an important factor for the reduction of floor accelerations and must be selected according to the design requirements. As discussed in Chapter 6, this ratio must be relatively low when protection of building contents dominate the design. Finally, proper selection of locations and dimensions of the isolators, considering their axial forces under service loads, can suppress torsional and rocking effects. Torsional effects result from eccentricities between the centers of stiffness and mass of the building. These eccentricities are due to the asymmetric configuration of the structural members, especially the very stiff elements such as the elevator core and the stairwells, and the asymmetric horizontal distribution of gravity loads. Although the proposed preliminary design methodology involves isolation systems with bilinear behavior applied to relatively stiff structures, the primary issues of concern are directly pertinent to most isolation systems.
7.2 A Feasibility Study for the Use of Seismic Isolation

At the preliminary design stage, a feasibility study must be carried out to investigate the plausibility and effectiveness of seismic isolation for the structure of concern. When the protection of a building’s contents and the avoidance of loss of functionality are the dominant design objectives for a building located in a high seismicity area, and thus suitable to be seismically isolated, seismic isolation is almost always the most effective and, in the long term, most economic design method. However, it is not suitable for all types of structures since its effectiveness is closely related to the superstructure characteristics. As previously mentioned, it is suitable for stiff low- to medium-rise buildings, bridges, and some special relatively stiff structures, such as nuclear reactors; it is usually not appropriate for flexible high-rise buildings. For seismic upgrading of existing structures for which seismic isolation is to be used, the difficulty and extent of modifications involved in its incorporation must also be considered in the feasibility study. Moreover, the seismicity of the area indicates the significance of seismic loads and the necessity for the incorporation of an appropriate system to account for these loads. Other minor loads such as wind loads must be also considered to determine the applicability of seismic isolation. In addition, site and foundation soil characteristics reveal the expected dominant frequencies of earthquake excitations and indicate the suitability and effectiveness of the method. Finally, even when the structure of concern is suitable to be seismically isolated, the distance of surrounding structures from it may prevent isolation. As previously mentioned, a seismic gap at the isolation level must be ensured and it may not be available if adjacent structures are too close to the structure of concern.

7.3 Determination of Peak Ground Acceleration

The level of seismic loading indicates its importance and must, therefore, be determined. The seismic input generally adopted consists of two levels of seismic excitation: a design earthquake, which is likely to occur once or twice during the useful life of the structure, and a maximum credible earthquake, which is the most severe earthquake expected at a particular region given the a priori knowledge of the local geological history. According to the provisions of most codes for seismically isolated structures, these two levels of seismic excitation must be considered in the design. The Uniform Building Code requires the
isolated structure to be designed and analyzed using a design earthquake loading with a 10% probability to be exceeded in 50 years, and a maximum credible earthquake with a 10% probability to be exceeded in 250 years. The isolated structure must respond elastically, i.e. without damage, for the design earthquake. For the maximum credible earthquake, limited inelastic deformation is allowed in the superstructure. The earthquake loadings must be determined considering the seismicity of the region, the importance of the structure and site conditions.

In a preliminary design procedure, based on the maximum expected displacement at the isolation level, it is adequate to use a linear design spectra, having first determined the peak ground acceleration \((pga)\) for the maximum credible earthquake. The estimation of \(pga\) may be done from acceleration attenuation curves, by locating the active seismic sources of the area and using the magnitude of the maximum credible earthquakes and the corresponding epicentral distances. The acceleration attenuation curves have been derived after extensive seismological research and studies on the soil characteristics of different regions, and provide \(pga\) as a function of earthquake magnitude and epicentral distance. Attenuation curves for different regions with similar geological characteristics may instead be used when ones for a particular area are not available. Alternatively, the available expected \(pga\) given in the codes, used in conventional earthquake-resistant design, may be used to determine the \(pga\) for the design and maximum credible earthquakes of the isolated structure.

7.4 Design of the Superstructure

The superstructure must be initially designed independently of the isolation system, although some of its structural characteristics may influence the efficiency of the system. In Chapter 6 it has been demonstrated that the stiffer the superstructure, the more efficient the seismic isolation system is, since the difference between the fundamental period of the isolated structure and the fixed supported superstructure increases with the inserted flexibility. The deformations of the superstructure are substantially decreased according to this difference, due to the actually rigid body motion of the whole structure. The masses accelerations are also reduced with the inserted flexibility, due to the avoidance of resonance. Thus, the superstructure must be designed to be as stiff as possible within reasonable eco-
nomical cost and architectural impact.

The superstructure must be analyzed for gravity dead and live loads, used to generate the required loading combinations in the analysis and design stages. Two loading combinations, namely maximum gravity loads and service gravity loads, must be determined by simple superposition of appropriately factored dead and live loads. Service gravity loads must be also combined with seismic, wind and/or any other non-gravity loads to generate the corresponding loading combinations. When special design details of the bearings do not allow the transmission of bending moments from the columns to the bearings, which is typically the case, the degrees of freedom corresponding ones to the supports’ displacements must be restrained. In contrast, the corresponding to the three rotations must be released. This modeling is based on the assumption that there is a diaphragm above the isolation level, which is sufficiently rigid in its plane to prevent relative displacements of the support nodes.

The configuration of the vertical members of the superstructure determines the potential locations of the isolators. Torsional effects are avoided by coinciding the center of stiffness with the center of mass of the superstructure. In practice, such effects cannot be completely eliminated. Therefore, in order to minimize them it is preferable to have the isolators at the outer edges of the structure, as far away as possible from the center of stiffness. The vertical configuration of the isolation system must be determined and a fully stiff diaphragm must be constructed above the isolation level, to distribute lateral loads uniformly to the isolators, taking advantage of the large in plane stiffness of the diaphragm. The plan dimensions of the bearings may be computed from the corresponding vertical loads which they must sustain. When the construction of a diaphragm is not possible, the bearings must be designed in proportion to the magnitude of the lateral forces carried by the members above them. It is difficult to take into account such design issues due to the uncertainties involved; therefore, the construction of a diaphragm above the isolation level must be anticipated. Note that here, the existence of a rigid diaphragm is assumed. Finally, the selected location of the bearings must be such that access to them is enabled for inspection and possible replacement purposes.
7.5 Selection of a Suitable Isolation System

The type and structural characteristics of the building under consideration, as well as the given design requirements must be considered in the selection of an appropriate isolation system. Non-linear systems, e.g. those with hysteretic or frictional behavior, are more effective in reducing large relative displacements and interstory deflections, particularly under extreme seismic loads. However, highly non-linear isolation systems are not appropriate when the protection of the structure’s contents is an essential design parameter, since they may produce high mode effects resulting in high floor accelerations. The latter may be reduced when the high initial, i.e. prior to yielding, stiffness of the isolation system is significantly lower than the stiffness of the superstructure. This ensures a sufficient degree of isolation for the loading level prior to yielding. This difference in stiffness is easily attained in very stiff superstructures with a high fundamental frequency. Alternatively, linear systems may be used to suppress higher mode vibrations and reduce floor accelerations. In addition, the continuous presence of a dissipation mechanism, even for minor lateral loads, may be essential in some cases where protection must be ensured under microtremors. The designer must also consider availability and cost of each different type of seismic isolation system as well as the experience of those who provide it. The most commonly used seismic isolation systems have been presented in Chapter 4 where the use of a hybrid isolation system has been proposed, to achieve an optimum compromise between the extreme cases of an approximately linear elastic and a highly non-linear hysteretic system. The hybrid isolation system which has relatively low initial stiffness has an increased degree of isolation prior to yielding, compared to a highly nonlinear system with significant abrupt stiffness changes. Finally, the objective and available funds for a particular project also determine which isolation system should be used. For example, cost considerations dominate the selection of an appropriate design in low cost dwellings, while for very important structures with expensive equipment, such as nuclear plants and computer centers, seismic performance and protection requirements prevail the design.

7.6 Preliminary Design of the Isolation System

The response of a seismically isolated structure under a strong earthquake excitation is approximately equal to that of the structure with the superstructure above the isolation
level treated as rigid. This has been demonstrated in Chapter 6 for the case of an isolation system inserted at the base of a structure. Thus, a single degree-of-freedom system, where the mass is that of the superstructure and the stiffness is that of the isolation system, may represent the structure adequately, at least for preliminary design purposes. Note that the results of a non-linear dynamic analysis results may be very different for the same structure and isolation system, under different seismic excitation scaled for the same level of pga. Therefore, considering the uncertainties associated with both the seismic loading and the response of a seismically isolated structure, it is sufficient to use an equivalent linear elastic viscously damped model and a design response spectra in the preliminary design stage.

The design of the isolation system must be based on a compromise between the degree of the isolation to be provided to the structure and the maximum allowable relative displacement at the isolation level. The more isolated a structure is, i.e. the larger the flexibility is inserted at the isolation level, the lower the seismic forces, interstory deflections and floor accelerations. On the other hand, the relative displacements are larger and thus practical constraints are imposed on the degree of isolation. In addition, the yield level is constrained by minor lateral loads, such as wind loads, to avoid the residents’ discomfort whenever the structure is subjected to such loads. When significant constraints are imposed on the acceptable maximum relative displacement, a high degree of isolation may be reached using auxiliary energy dissipation mechanisms, such as viscous dampers, to suppress the displacement at the isolation level. However, excessive damping may excite higher modes and result in higher floor accelerations which must be avoided.

7.6.1 Estimation of the Maximum Allowable Displacement
At the preliminary design stage, the maximum relative displacement of a structure may be approximately determined, considering practical restrictions due to the location of the structure and the preserve of facilities, such as elevators, pipes, etc. Reduction of the maximum allowable relative displacement at the isolation level may result in savings due to the reduced required seismic gap around the structure, and the flexibility of the connections of external services to accommodate such displacement.

The maximum allowable shear strains of the elastomeric bearings may be determined by the manufacturer or by laboratory tests and must be used to check the selected dimen-
sions of the isolators. The allowable shear strain used in the design of bridge bearings is 20% of the failure strain, \( \varepsilon_f \), in a simple tension test [Skinner et al., 1993]. However, bridge bearings have to carry well-defined service loads which act continuously. Instead, the loads used for the design of seismically isolated structures are extreme seismic events which are unlikely to happen more than once or twice during the lifetime of the structure. Therefore, for earthquake loadings the allowable shear strains may be higher than the shear strains under frequent loadings. Allowable shear strains are typically of the order of 0.5 for design earthquakes and higher for extremely rare events; tests indicate rupture of the rubber for strains of the order of 500%.

7.6.2 Estimation of Yield Force \( F_y \) of Isolators

The yield force of the isolation system must be determined considering minor lateral loads such as wind loads. It is, indeed, usually considered to be equal to these loads. Initial high stiffness is provided by the lead of the LRBs which may be used either under each column or only under selected columns at the perimeter of the building. The latter case is preferable when relatively low prior to yielding stiffness is anticipated, since it provides higher resistance to torsion. When LRBs are used under each column, the lead plug dimensions may be easily determined using the axial forces for service gravity loads. The analysis of the superstructure under the service gravity loading combination, defines the axial loads at each bearing location which may be used to properly determine the size of the lead plugs to avoid torsional effects. The diameter of each lead plug may be determined considering the minor loads to be carried by the lead plugs and using the lead yield strength, in order to have initial rigidity under these loads. The yield strength of lead \( \sigma_{ly} \) is approximately 10.5 MPa, while the ratio of the yield force \( F_y \) to the total weight of the structure under service loads \( W_s \), is typically chosen to be 5%. As shown in Chapter 6, a lower value of \( F_y / W_s \) ratio, e.g. 3%, may be more appropriate particularly for floor acceleration reductions. The required area and diameter of the lead plug may be determined using the computed axial load at the location of each bearing and the ratio of \( F_y \) to the total weight of the structure under service loads, \( W_s \).

\[
A_{lead}^i = \frac{F_y}{\sigma_{lead, y}} \frac{W_s^i}{W_s}
\]  

(7.1)
Using the above formula to compute the required lead plug diameter, the center of mass for the service loading combination will automatically coincide with the center of stiffness prior to yielding. [Mayes et al, 1984] estimate that the lead plug diameter must lie in the range of one third to one sixth of the bearing diameter, i.e. $\frac{D}{6} < d_{\text{lead}} < \frac{D}{3}$, and not more than 2/3 of the total rubber height $h_{\text{tot}}$, values which are not yet known at this stage of design, and must therefore checked at a later stage.

### 7.6.3 Determination of Required Effective Stiffness

Seismic isolation is more effective when the fundamental period of the fixed supported superstructure and that of the isolated structure are well separated, which in turn results in rigid body motion of the superstructure with small floor accelerations and interstory deflections. The objective of the design is to make seismic isolation more efficient by increasing the inserted flexibility to ensure a large effective fundamental period, under the restriction of maximum acceptable displacement at the isolation level. The preliminary design must primarily address this issue, using a simplified single DOF system. To determine the required characteristics of the isolation system, the effective stiffness and damping may be used at the preliminary design stage to provide rough approximations of the overall response of the isolated structure.

Having estimated the maximum acceptable displacement ($U_{\text{max}}$), the $pga$ for the maximum credible earthquake, the ratio $F_y/W$, the soil coefficient $S$ and the damping coefficient $B$, are used to compute the required effective period $T_{\text{eff}}$ from a normalized design spectra. The design spectra, according to the Uniform Building Code (UBC), is defined by the following expressions:

\[
C = \min \left( \frac{1.25 S}{T^2/3}, 2.75 \right) \quad S_a = \frac{C \cdot pga}{B} \quad (7.3)
\]

where $S$ is the site coefficient to consider the foundation soil characteristics, and $B$ the damping coefficient. The site coefficient takes values of 1.0, 1.2 and 1.5 for soil types $S_1$, $S_2$ and $S_3$, respectively. Although these values have been adopted by the ‘Earthquake Design’ of UBC for conventional fixed supported structures, they are more appropriate in
the preliminary design than those given in provisions for seismically isolated structures. The latter have been conservatively overestimated for the case of a simple static response procedure and their use leads to the selection of ineffective seismic isolation systems. The damping coefficient may be determined using an approximate estimation of the minimum equivalent viscous damping ratio $\xi$. In particular, $B$ may be considered to be 1.0, 1.2, 1.5 and 1.7 for damping ratio $\xi$ equal to 5%, 10%, 20% and 30%, respectively. Considerable research has been done on the expected seismic events, the results of which are currently used in the conventional earthquake-resistant design, and which may also be used in the preliminary design of seismically isolated structures. Once $T_{\text{eff}}$ has been determined and the ratio $(K_e/K_{pl})$ had been specified, the initial stiffness $K_{ei}$ and the post-yield stiffness $K_{pl}$ of the isolation system may be subsequently computed.

When a design spectrum is used, the equivalent viscous damping must also be determined. Although some inelastic deformation of the superstructure is allowed for the maximum credible earthquake, the energy dissipation mechanism must be limited to that absorbed by the seismic isolation system, not allowing ductility reductions for safety purposes. As discussed in Chapter 6, we have tried to investigate whether the bilinear response of a seismically isolated structure with lead rubber bearings can be characterized by an equivalent viscously damped system. A single degree-of-freedom system has been used to model the physical problem; the obtained results showed pronounced variations. However, when we compared the computed maximum displacements with those obtained from the ATC3-06 design spectra for 10% damping ratio, we observed that the computed displacements were well enveloped by those obtained from the design spectra. The variation of the equivalent viscous damping for a hybrid system is less than that of a LRB which has a purely hysteretic energy dissipation mechanism. Having estimated a damping ratio $\xi_{\text{eff}}$, the damping coefficient $B$ may be subsequently determined.

A near-fault effect coefficient $N$ is also defined in the current provisions for the static procedure used for seismic isolation, to consider the effect of long components of the ground motion in the design displacements, due to the proximity of the structure to an active fault. This coefficient increases the design displacement by 50% and 20% when the structure is located within 5 or 10 km, respectively, from the active fault. This coefficient is conservatively high, mainly to limit the use of the simple static procedure. In this pre-
liminary design procedure it has been ignored since its use requires high stiffness due to the substantial increase of the estimated maximum displacement. The possibility of dominance of long components of the ground motion must be taken into account during the dynamic analysis of the isolated structure, using excitations with such characteristics. Adoption of the relatively high values of the near-fault coefficient would result in a significant increase of the expected displacement, using the design spectrum, and consequently to less flexible isolation systems. In most cases, lower flexibility of the isolation system not only reduces its effectiveness but may also result in resonance between the isolation system and the ground motion. In extreme cases, e.g. very soft soils, where the dominant ground motion period is so high that it is impossible to reach the necessary difference from the fundamental period of the structure, seismic isolation should not be used.

The pseudo-velocity $S_v$ in the range of the fundamental period of a typical seismic isolated structure is constant. The spectral acceleration is therefore proportional to the inverse of the period $T$ and not to $T^{2/3}$. Thus, in the denominator of the expression for the coefficient $C$ in equation 7.3, $T$ should be used instead of $T^{2/3}$. The latter has been conservatively adopted in codes for very flexible structures. In addition, for a typical fundamental period of an isolated structure the coefficient $C$ is less than 2.75. Therefore, the pseudo-velocity $S_v$ may be computed as a function of $C$, by the expressions

$$ C = \frac{1.25}{T} \quad S_v = \frac{1.25}{2 \pi B} \frac{S}{pga} \quad (7.4) $$

The maximum relative displacement is then computed as:

$$ U_{max} = S_d = \frac{S_v}{\omega_{eff}} = \frac{1.25}{4 \pi^2 B} \frac{S}{pga} \frac{T_{eff}}{T} \quad (7.5) $$

Having determined the maximum acceptable relative displacement $U_{max}$, the ratio of the yield force to the weight of the structure $F_y/W_s$, $pga$ and equivalent viscous damping, the required effective period $T_{eff}$ may be computed as

$$ T_{eff} = \frac{4 \pi^2 B U_{max}}{1.25 S pga} \quad (7.6) $$

The effective stiffness $K_{eff}$ may be expressed using the force-displacement relation of a bilinear system, as a function of $U_{max}$:
The effective period $T_{\text{eff}}$ is then given by

$$T_{\text{eff}} = 2 \pi \sqrt{\frac{M}{K_{\text{eff}}}} = 2 \pi \sqrt{\frac{M U_{\text{max}}}{\left(1 - \frac{K_{\text{pl}}}{K_{\text{el}}}ight) F_{y} + K_{\text{pl}} U_{\text{max}}}}$$

(7.8)

We set Equations 7.6 and 7.8 to be equal and obtain

$$T_{\text{eff}} = \frac{4 \pi^{2} B U_{\text{max}}}{1.25 S \text{pga}} = 2 \pi \sqrt{\frac{M U_{\text{max}}}{\left(1 - \frac{K_{\text{pl}}}{K_{\text{el}}}ight) F_{y} + K_{\text{pl}} U_{\text{max}}}}$$

(7.9)

A $K_{\text{el}}/K_{\text{pl}}$ ratio must be assumed and then the above equation must be solved for the post-yield stiffness $K_{\text{pl}}$. For lead rubber bearings, the ratio $K_{\text{el}}/K_{\text{pl}}$ is typically 10, while for lead high-damping rubber bearings it may be lower (close to 6).

$$K_{\text{pl}} = \frac{M S^{2} \text{pga}^{2} - 25.266 B^{2} F_{y} U_{\text{max}} \left(1 - \frac{K_{\text{pl}}}{K_{\text{el}}}ight)}{25.266 B^{2} U_{\text{max}}^{2}}$$

(7.10)

The post-yield stiffness $K_{\text{pl}}$ may be specified for each particular isolator using the axial load $W_{s}^{i}$ which corresponds to a service load combination, since the isolators act as springs in parallel. It is convenient to express the derived equation as the ratio of the post-yield stiffness $K_{\text{pl}}$ to the axial load at a particular isolator:

$$\frac{K_{\text{pl}}^{i}}{W_{s}^{i}} = \frac{S^{2} \text{pga}^{2} - 247.86 B^{2} \left(F_{y}/W_{s}\right) U_{\text{max}} \left(1 - \frac{K_{\text{pl}}}{K_{\text{el}}}ight)}{247.86 B^{2} U_{\text{max}}^{2}}$$

(7.11)

For a very low effective period, which is almost never the case, the post-yield stiffness of each isolator may be determined by using the following equation, obtained using $C=2.75$, to determine the maximum displacement:
Once the maximum acceptable relative displacement $U_{max}$, and the yield force $F_y$ have been determined, and assuming a particular non-linearity ratio $K_{el}/K_{pl}$, the required post-yield stiffness may be calculate from the above formulas. The global parameter $K_{pl}^i/W_s^i$, may be used to compute the required stiffness of each individual isolator, by multiplying the computed ratio by the corresponding vertical loads. Then, the center of post-yield stiffness $K_{pl}$ will coincide with the center of mass of the superstructure under service loads, thus avoiding torsional effects.

The variation of the required ratio of post-yield stiffness to axial load $K_{pl}/W_s^i$ is shown in Figure 7.1 for two different cases, as a function of the maximum displacement $U_{max}$. In the first case (Figure 7.1.a), the assumptions are: stiff and shallow foundation soil, i.e. $S_1$ soil coefficient, minor lateral loads equal to 3% of the total service weight of the structure, a ratio of initial to post-yield stiffness equal to 6, approximately 10% equivalent viscous damping, and that the site is located more than 15 km from the active fault of concern. In the second case (Figure 7.1.b), it is assumed that the structure is founded on a soil profile of soft to medium stiff clay with limited depth of soft clay, i.e. $S_3$ soil coefficient, minor lateral loads are of the order of 5% of the total service weight, $K_{el}/K_{pl}$ is equal to 10, equivalent viscous damping is 20% and the active fault is situated within less than 10 km from the structure. The required values of $(K_{pl}/W_s^i)^i$ are plotted in Figure 7.1 for each of the above mentioned cases, for three different peak ground accelerations, 0.3, 0.4, and 0.5 g.
Figure 7.1: Design charts for \( (K_p/W_s)^j \), based on maximum allowable displacement. The values of the ratio of post-yield stiffness \( (K_p/W_s)^j \) to the maximum allowable displacement \( U_{max} \), as derived by the above charts, give the effective period values \( T_{eff} \) using equation 7.8. The charts are shown in Figure 7.2.

Figure 7.2: Design charts for required effective period \( T_{eff} \) based on \( U_{max} \).

Then, the horizontal frequency of the structure \( f_h \) may be computed as

\[
 f_h = \frac{1}{T_{eff}} \quad (7.13)
\]
7.6.4 Determination of Plan Size Dimensions of Isolators

The plan size dimensions of the isolators must be determined by the required vertical capacity, using the maximum axial forces on each bearing under various load combinations. The easiest way to compute the required area of each isolator $A_i$ is to use the maximum axial forces from the maximum gravity loads combination. In addition, when the shape of the structure indicates the plausibility of occurrence of substantial additional axial forces, from overturning moments due to seismic loads, their effects may be approximately taken into account by their equivalent static application. When maximum axial loads from seismic combinations are used to determine the plan size dimensions of isolators, the effective area $A_{eff}$ may be used instead of the actual area $A$, to take into account the deformed shape of the bearing. The effective area $A_{eff}$ has been defined in Chapter 4 and is typically of the order of 0.6 to 0.8 of the area $A$. The vertical load capacity of an elastomeric bearing is determined by the plan dimensions of the bearing and the internal layer thickness. In addition, the displacement of the top layer with respect to the bottom reduces the area of the ‘effective column’, which may conservatively be considered to transfer vertical loads to the foundation.

The plan dimensions of the isolators must be chosen to ensure an approximately uniform distribution of vertical stresses well below the allowable compressive stress. As mentioned in Chapter 4, the plan area and the primary shape factor $S_1$, and consequently, the internal layer rubber thickness, influence the vertical stiffness of the bearing $f_v$. This must be sufficiently large to avoid rocking of the superstructure. For most buildings, $f_v$ is in the range of 8-12 Hz [Kelly, 1993]. The ratio of the vertical to the horizontal frequency may be expressed using Equations 4.20 and 6.6 and may be simplified as follows:

\[
\frac{f_v}{f_h} = \frac{1}{2 \pi} \sqrt{\frac{K_v}{W_s}} = \frac{K_v}{K_h} = \sqrt{\frac{6 \cdot G \cdot S_1^2 + 3 \cdot G \cdot A_r / h_r}{G \cdot A_r / h_r}} \quad (7.14)
\]

\[
\Rightarrow S_1 = \sqrt{\frac{(f_v/f_h)^2 - 3}{6}}
\]
When typical values of the ratio $f_v/f_h$ are used, the above expression may be simplified to the following expression which may anyway be determined using Equation 4.18 instead of 4.20.

$$\frac{f_v}{f_h} = \frac{K_v}{K_h} = \sqrt{\frac{6 G S_I^2 A_r/h_r}{G A_r/h_r}} \Rightarrow S_I = 0.408 \frac{f_v}{f_h} \quad (7.15)$$

Therefore, once the desired vertical frequency has been determined, the primary shape factor may in turn be computed. This factor is necessary in the determination of the plan dimensions of the bearings as it effects the bearings' vertical capacity.

The seismic effect on the maximum axial force on each bearing may be approximately taken into account, using equivalent static seismic forces combined with service loads. Subsequently, the superstructure may be analyzed as fixed supported for additional seismic loading combinations composed of service gravity loads and seismic loads in each horizontal direction. These loadings are based on the uniform distribution of the design base shear force $F_{max}$ on the superstructure. The following expression may be used to compute $F_{max}$:

$$F_{max} = F_y \left( 1 - \frac{K_{pl}}{K_{el}} \right) + K_{pl} U_{max} \quad (7.16)$$

As already mentioned, the distribution of the inertia seismic forces on an isolated structure have an approximately constant distribution, while that on a fixed supported structure linearly increases according to the shape of the fundamental mode. These forces must be applied in both the positive and the negative horizontal directions. Therefore, the equivalent static lateral forces applied to each mass $m_j$ are given by

$$F_{hj}^i = F_{max} \frac{m_j^s}{\sum m_j^s} \quad (7.17)$$

Using the maximum calculated axial loads at the bearing locations from all the load combinations and the allowable stresses at the bearings, with appropriate safety factors, the required plan size dimensions of each bearing are computed. The allowable compres-
sive stress is the minimum of 6.89 MPa and 0.689, times the product of the shear modulus $G$ and the primary shape factor $S_f$ [Roeder et al., 1987].

$$\sigma_{all} [MPa] = \left( \min(6.895 [MPa], 0.6895 G [MPa]) S_f \right) / SF$$ (7.18)

A safety factor $SF$ may be used for possible variations of the compressive strength. Typical values of the primary factor $S_f$ are in the range of 10 to 20.

When circular bearings are used, and similarly for square bearings, we may obtain the required diameter as a function of the corresponding to the bearing maximum applied vertical load $W_{max}$:

$$D_i [m] = \frac{4 W_{max} [MN]}{\pi \sigma_{all} [MPa]}$$ (7.19)

As previously mentioned, for the maximum axial loads from seismic loading combinations, the reduction of the area to the effective area $A_{eff}$ must be taken into account. This may be achieved by multiplying of the above derived dimension by the square root of the ratio of $A$ to $A_{eff}$. A typical $A_{eff}/A$ ratio of 0.7 will lead to an increase of 20% of the above-computed required diameter. Once the diameter has been determined, it may be used, together with the primary shape factor, to determine the internal rubber sheet thickness $t_r$.

Typical values of the latter are within 5-15 mm.

$$t_r = \frac{D}{4 \cdot S_f}$$ (7.20)

### 7.6.5 Determination of Required Total Rubber Thickness

The total rubber thickness must be determined to provide the required flexibility to the structure. Increasing the total height of the rubber increases the inserted flexibility; in other words it decreases the stiffness of the isolation system. Having computed the post-elastic stiffness $K_{pl}^i$ and the area $A_r^i$ of each bearing, the total height of the rubber $h_r^i$ may be determined by the following formula:

$$h_r^i = \frac{G_r A_r^i}{K_{pl}^i}$$ (7.21)
Then, the number of required rubber sheets $n_r$ for each bearing may be determined using the internal rubber sheet thickness $t_r$ and the total height of the rubber $h_r^i$. The number of steel internal plates $n_s$ is also derived from $n_r$.

$$n_r = \frac{h_r}{t_r} \quad \text{and} \quad n_s = n_r - 1 \quad (7.22)$$

The thickness of the internal steel plates $t_s$, typically of the order of 3mm, must be selected. The total height of the rubber $h_{tot}$ of each bearing may then be computed using the previously determined dimensions and the thickness $t_p$ of the top and bottom plates, which is typically 20-40 mm.

$$h_{tot} = n_r \cdot t_r + n_s \cdot t_s + 2 \cdot t_p \quad (7.23)$$

### 7.6.6 Design Corrections and Adjustments

This stage requires engineering judgement since it involves adjustments of the selected parameters to obtain an acceptable preliminary design. There are several issues to be checked. First, bearings must be checked to ensure that the maximum axial loads do not exceed the critical for buckling loading. In Chapter 4, the following expression has been derived for the critical load

$$P_{cr} = \frac{0.134 \cdot G_r \cdot D_r^5}{h_r \cdot t_r^2} \quad (7.24)$$

A safety factor $SF_b$ for buckling must be used, i.e

$$P_{max}^i \leq \frac{0.134 \cdot G_r \cdot D_r^5}{SF_b \cdot h_r^i \cdot t_r^2} \quad (7.25)$$

Then, the selected dimensions must be checked to ensure that they fall within acceptable ranges; otherwise they must be modified. Shear strains of the isolators $\gamma_i$ must be computed from the total rubber thickness $h_r^i$ of each bearing, and the maximum displacement $U_{max}^i$, and be subsequently checked to ensure that they are below the allowable strains $\gamma_{all}$, i.e
\[ \gamma_i = \frac{U_{\text{max}}}{h_r} \leq \gamma_{\text{all}} \]  

(7.26)

As previously mentioned, the diameter of the lead plug of each bearing \( d_i \) must also be checked to lie within the range of one third to one sixth of the bearing diameter \( D_r \) and to be less than two thirds of the total rubber height \( h_{\text{tot}} \).

\[ \frac{D_r}{6} < d_i < \frac{D_r}{3} \quad \text{and} \quad 1.5 \cdot d_i < h_{\text{tot}} \]  

(7.27)

When one tries to adjust the dimensions of the lead plugs of some bearings, it is possible that these dimensions may be too small. In such cases, it is preferable to have the bearings without lead plugs. The latter may instead be distributed only to external columns to satisfy the initial stiffness requirement and increase the torsional resistance.

In what regards the prior to yielding stiffness of the isolation system, one must first determine the stiffness of each bearing \( K_{el}^i \). The following expression may be used:

\[ K_{el}^i = \frac{G_l A_l^i}{h_i^i} + \frac{G_r A_r^i}{h_r^i} \]  

(7.28)

The system’s overall prior to and post-yield stiffness, \( K_{el} \) and \( K_{pl} \), respectively, is then calculated by summing the individual stiffnesses, i.e.

\[ K_{el} = \sum K_{el}^i \quad K_{pl} = \sum K_{pl}^i \]  

(7.29)

The center of mass of the superstructure must be determined in order to check eccentricities. This is achieved using the axial column forces under the service loads and an auxiliary coordinate system, i.e.

\[ X_{SM} = \frac{\sum W_s^i \cdot x_i^i}{\sum W_s^i} \quad Y_{SM} = \frac{\sum W_s^i \cdot y_i^i}{\sum W_s^i} \]  

(7.30)

Then, the eccentricities are determined by a cartesian coordinate system with its origin at the center of mass \( SM \), as shown in Figure 7.3.
The eccentricities $e_{el}^x$ and $e_{el}^y$, between the center of stiffness prior to yielding and the center of mass, may be computed as follows:

$$
e_{el}^x = \frac{\sum K_{el}^i \cdot x_i}{\sum K_{el}^i} , \quad e_{el}^y = \frac{\sum K_{el}^i \cdot y_i}{\sum K_{el}^i}$$

(7.31)

Similarly, the eccentricities $e_{pl}^x$ and $e_{pl}^y$ are

$$
e_{pl}^x = \frac{\sum K_{pl}^i \cdot x_i}{\sum K_{pl}^i} , \quad e_{pl}^y = \frac{\sum K_{pl}^i \cdot y_i}{\sum K_{pl}^i}$$

(7.32)

From the discussed preliminary procedure we deduce that the designer must experiment with different values of the above-mentioned critical parameters, the dimensions of the components of the isolation system in particular. A trial and error process is necessary to ultimately satisfy the different design requirements. Once this procedure has been completed, the designer will have initial values of these parameters which are needed in the final and more detailed analysis.

### 7.7 Issues of Concern at the Final Analysis Stage

A three-dimensional time history dynamic analysis, for several appropriately selected accelerograms, must be performed in the final stage of the design to provide highly accurate results, which cannot be obtained with simpler models. This analysis is required to take
into account the vertical displacements, possible uplifts of the isolators, horizontal distribution of seismic forces, especially at the isolation level, torsional effects due to eccentricities and, in general, local results for individual structural elements and isolators. Finally, a non-linear dynamic analysis is necessary when the isolation system is highly non-linear, to explicitly take into account the hysteretic energy dissipation mechanism, and investigate the possibility of higher mode effects. The results of this more detailed analysis will indicate necessary adjustments to the isolation system.

When strong vertical ground motion components are expected, it would be useful to study the effects of a vertical excitation on the isolated structure. The selection of the accelerograms must be based on site characteristics, design earthquake magnitude, expected predominant period and distance from active faults. Thus, the availability of a seismic database with strong excitations would be useful for the appropriate selection of ground motion time histories. The dominant period of the ground motion tends to increase with the increase of the layer depth and the decrease of the soil stiffness. Stiff soils therefore amplify the high frequency components of ground motion, while deep soft soils amplify the low frequency ones. The high frequency components have a tendency to attenuate faster than the low ones, thus resulting in a dominant period which lengthens with the epicentral distance. The depth of the seismic fault also influences the expected pga. Long components of the earthquake excitation may also be present due to near-fault effects, or when the epicentral distance is large and seismic waves propagate through deep soft layers of soil. In the case that pertinent information is not available in the earthquake database, excitations may be scaled by an appropriate time scale factor corresponding to the expected dominant period. The selection of the appropriate scaling of excitations is of particular importance for seismically isolated structures. This is because when accelograms with short predominant period are used in the analysis, and long components of ground motion are in contrast expected, the effectiveness of seismic isolation is overestimated. Under the design earthquake, yielding of the superstructure must be avoided; for the maximum credible earthquake, limited ductile inelastic deformations in the superstructure may be accepted only when it is unavoidable. The different effect of yielding on a seismically isolated structure from that of a fixed supported one has been discussed in Chapter 6 and must be taken into account.
When a 3-D analysis is performed, the isolated structure is more accurately modeled and more reliable results are obtained. When a non-linear isolation system is used, the hysteretic energy dissipation may now be directly considered using the nonlinear behavior of the isolators. The higher-mode effects due to the bilinear behavior of the isolation system may also be studied to ensure the prevention of high floor accelerations. Torsional effects due to accidental eccentricities may also be more accurately assessed. Under the design earthquake, it must be ensured that the maximum computed lateral displacement is well below the distance provided by the seismic gap. Under the maximum credible earthquake, the corresponding displacement must be again be checked to ensure that it still is below the seismic gap. A safety margin must be included to allow additional displacements due to torsional effects since in practice they are almost never completely avoided. From the maximum displacement and the bearings’ dimensions, the shear deformations may be computed and checked to be within allowable limits. Necessary adjustments must be made according to corresponding guidelines to those mentioned at the preliminary design procedure.

Axial forces must be determined and checked to ensure that the isolators will not be damaged, neither due to buckling resulting from compressive axial stresses, nor due to extensive tensile stresses. In Chapter 4 the critical, for buckling, loading has been determined. It may be used here as it has also been used in the preliminary design procedure. The tensile stresses may result in overturning moments on columns with low gravity loads. When the axial stresses on the bearings are checked for seismic loads, the reduced effective area $A_{\text{eff}}$ must be used. Tensile stresses may be introduced to elastomeric bearings due to uplift forces, which cancel the gravity loads. The possibility of isolators uplift must be checked for seismic loads and reduced dead loads. High tensile stresses may generate flaws in the elastomer which may progressively grow with reversed loading, leading to failure. Appropriate location and design of the bearings is required to reduce or avoid tensile stresses. In addition, construction details may prevent the transmission of tensile forces.

The superstructure must be designed to sustain at least the design earthquake with no damage, i.e. without inelastic deformations. The current ductility requirements used for conventionally designed earthquake-resistant structures must not be relaxed for seismi-
cally isolated structures, even though ductility requirements are limited for them. As previously mentioned, the reasons are the possibility of vertical and long period components, the near-fault effects and the limited experience with seismic isolation. In addition, the reduction of the effectiveness of seismic isolation, when the superstructure experiences inelastic deformation due to its softening, may be counterbalanced by additional energy dissipation when ductility capacity is available. Under the maximum credible earthquake the superstructure must exhibit limited or no inelastic deformations. The isolation system must be less conservatively checked for this loading, to withstand the excitation without failure. Also, the maximum displacement must not exceed the provided seismic gap. The possibility of such excess displacement may be accounted for by introducing stoppers and bumpers, as described in Chapter 5. Whenever necessary, adjustments on the preliminary selected design must be made to satisfy the design requirements. Once the design has been significantly modified, a new 3-D analysis must be performed and the new response be checked again.
Chapter 8

Conclusion

The thesis has explored seismic isolation as a performance-based design method to account for earthquake excitations in low- to medium-rise buildings and bridges. The limitations of conventional earthquake-resistant design and the resulting damage, which is not anymore acceptable, has motivated the exploration of the characteristics and the potential of this design method. Conventional earthquake-resistant design has been described in Chapter 2. Its limitations and disadvantages, particularly for stiff structures, have been discussed. The focus of conventional design on the prevention of structural collapse and the avoidance of casualties has been emphasized. The disadvantage of a conventionally designed structure to be in resonance with the usual ground motions has also been demonstrated. The fact that this method relies on the ability of the structure, when properly designed and constructed, to provide high levels of ductility, has been pointed out, as well as the fact that damage under strong excitations is anticipated, as a mean to absorb energy and avoid structural collapse. Subsequently, it has been argued that since the previously accepted damage level is no longer tolerated, a different design methodology is needed to design earthquake-resistant structures. Seismic isolation has then be presented, as an alternative way of design and construction to address the issue of damage. Its main feature to decouple the structure from the destructive ground motion has been explained, as well as its success in reducing induced seismic forces on the superstructure. In Chapter 3, the basic components, advantages, constraints and limitations of seismic isolation have been described. A comparative study of a system with two boundary conditions, namely fixed supported and seismically isolated, has been performed to demonstrate the better performance of seismic isolation. In particular, the reduced interstory deflections and floor accelerations, at the cost of large relative displacement at the isolation level, have been reported. In addition to the protection of the structure and its contents, avoidance of business interruption and loss of functionality of essential public facilities have been considered as important reasons for the employment of the method. Factors which enable the use of seismic isolation have been presented, as well as economic aspects related to the practi-
cal implementation of the method, for both new construction and seismic upgrading of existing structures. Finally, a brief account of the reasons due to which seismic isolation is not yet extensively used has been given.

Commonly used seismic isolation systems have been described in Chapter 4. The discussion has focused on elastomeric bearings, LRBs, HDRBs and LHDRBs in particular. The potential of a hybrid system, consisting of high damping rubber with a lead plug, has been assessed. This system has both initial rigidity and a continuous energy dissipation mechanism which is less temperature dependent than that of a HDRB system. In addition, it allows lower prior to yielding stiffness which provides higher initial degree of isolation and lower high mode effects, due to smoother stiffness changes. The use of high damping rubber bearings with no lead plugs, under internal columns, has been identified as a means of increasing torsional resistance while reducing the prior to yielding stiffness. The mechanical and physical characteristics of elastomeric bearings have then been presented, followed by an investigation of several issues useful in the design of isolated structures. Expressions for the vertical stiffness, buckling load and effective column area of elastomeric bearings have been derived and have been subsequently used in the proposed preliminary design procedure.

A brief account of the current status of seismic isolation in countries where it is extensively used has been given in Chapter 5. Selected examples of seismically isolated structures have been presented. Emphasis has been placed on the observed performance of the structures which have already been subjected to earthquake excitations. Then, practical issues and problems arising when seismic isolation is employed have been identified and ways to address them have been proposed. First, the configuration of a seismic isolation system has been described as well as the physical maintenance requirements which must be met upon its installation. Among them, the importance of the clearance around the structure and the possible necessity of mechanisms such as stoppers and bumpers, to constrain the relative displacement of the superstructure and avoid collisions with adjacent structures, have been emphasized. Finally, code requirements for seismic isolation, particularly in the United States, have been mentioned.

The structural characteristics of seismically isolated structures have been investigated in Chapter 6. A modal analysis of a seismically isolated structure has first been performed
to demonstrate the dominance of the fundamental mode of the structure on its response. The characteristics of linear isolation systems have been subsequently described and the effect of viscous damping has been assessed. As shown, by increasing the viscous damping, the displacements under an earthquake excitation are reduced, while the accelerations are reduced for damping only up to approximately 20-30% of the critical damping; for higher values the accelerations increase. It has, therefore, been deduced that an optimum viscous damping may be computed for the reduction of floor accelerations. A description of bilinear systems and a comparative study of a linearly and a bilinear isolated structure has then been performed. The relative performance of the two systems and that of a hybrid system with both viscous and hysteretic damping have shown the better performance of the hybrid system. Then, the linearization of the bilinear behavior of an isolation system has been attempted. As shown, the values of the different parameters of concern, when such a linearization is made, vary significantly with both the structural and earthquake characteristics. Next, a study of the effect of non-linearities has indicated that a bilinear hysteretic system with moderate non-linearity and relatively low prior to yielding stiffness and yielding force, combined with a viscous damping, may take advantages of both linear and non-linear systems. The effect of the superstructure stiffness has also be discussed and it has been demonstrated that its increase affects positively the efficiency of seismic isolation, especially the reduction of interstory deflections. Finally, the issue of ductility has been discussed, emphasizing its influence on a seismically isolated and a fixed supported structure. Inelastic deformations in a fixed supported structure are beneficial, due to energy dissipation and softening of the structure which drives it away the dangerous for resonance range. In contrast, in the case of a seismically isolated structure, the corresponding effects are contradictory. On one hand, the inelastic deformations provide an additional energy dissipation mechanism, but on the other hand, the softening of the superstructure reduces the effectiveness of seismic isolation.

Finally, a preliminary design methodology for seismically isolated structures has been proposed in Chapter 7. It may be used to obtain parameter values which are necessary in the final and more detailed analysis. This procedure has been based on bilinear isolation systems, elastomeric bearings with lead plugs in particular, although the primary issues addressed are pertinent to most systems. A feasibility study has been suggested at the first
stage of the preliminary design, to investigate the plausibility and effectiveness of seismic isolation. The level of the expected earthquake excitation and in particular the peak ground acceleration must be also estimated. Then, the superstructure configuration must be determined and analyzed for both dead and live loads. The next step is to select an appropriate isolation system given the particular design requirements. Finally, the preliminary design of the isolation system must follow, which in turn involves several stages. Due to the several interrelated parameters of concern, it may require a trial and error procedure which will ultimately lead to an optimum design. First, the maximum allowable displacement must be estimated, considering practical constraints. Then, the yield force of the isolation system must be determined according to the minor lateral service loads and the diameter of the lead plugs at each bearing must be computed. The required post-yield stiffness may be computed from a design spectrum after the estimation of the equivalent viscous damping of the isolation system, the non-linearity ratio $K_e/K_{pl}$ and site characteristics. The corresponding fundamental period and frequency of the isolated structure may then be computed. Subsequently, the plan dimensions of each bearing may be determined using the requirement for the maximum allowable compressive stress and the vertical stiffness to avoid rocking. Finally, the total rubber thickness is determined to provide the computed post-yield stiffness. Having obtained the primary shape factor, the thickness of the internal rubber sheets may be computed, and subsequently the number of the rubber layers. The thickness of the steel plates may also be selected. These values are used to determine the total thickness of the bearing. Adjustments and corrections must also be performed to ensure the satisfaction of the design requirements. First, the axial forces on the bearings must be checked to ensure that they are well below buckling load. Then, shear strains may be computed and checked against the maximum allowable shear strains. Finally, the prior to and post yielding stiffness must be checked to ensure the desired stiffness and avoid torsional effects. The procedure may require also several adjustments and corrections to obtain an adequate design of the isolation system. Issues of concern in the final and more detailed analysis have also been discussed. The importance of appropriately selected and scaled ground motion has been emphasized, since it has significant effects on the response of the isolated structure. Although seismic forces and required amount of ductility in a seismically isolated structure are substantially reduced,
the current design details to ensure ductility capacity must not be relaxed, until sufficient knowledge on the behavior of seismically isolated structures, and a reliable estimate of the expected earthquake excitation is obtained.

It is evident from the discussion that seismic isolation is an efficient design scheme, which successfully addresses earthquake loadings, and not only provides safety but also prevents damage. However, it is not applicable to all structures; therefore a feasibility study is imperative at the early stages of design to assess its effectiveness according to particular design requirements. In addition, different design requirements need different isolation systems; thus, the appropriate selection must be made based on knowledge and understanding of the characteristics, advantages and limitations of each system. As with any other innovative design method, seismic isolation may require some time before it becomes established as a method to account for seismic loads. Currently it is not widely accepted mainly due to the engineers' conservation in adopting innovative construction methods, particularly when there are no immediate cost savings. Employment of seismic isolation requires an additional cost, although in the long term it ensures savings due to reduced damage and uninterrupted function of the structure. Recent seismic events have shown the necessity of more stringent codes, imposing performance requirements to avoid damage and ensure safety. It is therefore expected that seismic isolation will be more extensively used in the future.

Authorities may also promote the use of seismic isolation as well as any other method with enhanced seismic performance, since its application will result in reduced damage and corresponding reimbursements. In addition to the modification of codes, which will favor the use of seismic isolation, information on the advantages of the method may also be more broadly available. Insurance companies may also provide special premiums for structures which have been seismically isolated, due to their better performance.

Isolation systems must also be standardized to allow their easier use. Also, future civil engineers must be exposed to the method, its characteristics and purpose, during their studies. Finally, as the number of seismically isolated structures increases, more experience will be gained on the performance of the method and its benefits.

The establishment of seismic isolation may just be the starting point. Other innovative design approaches, such as passive and active control, may follow. Also, it may be possi-
ble that seismic isolation can be combined with a particular control scheme, for example one to account for wind loads, to address the issue of multiple natural loadings.
References


Appendix A

Linear Elastic Systems

A.1 Single Degree of Freedom Systems

The motion of the foundation ground due to an earthquake excitation results in inertia forces acting on the structural masses. The response of many buildings is mainly characterized by their fundamental eigenmode. The response of a seismically isolated building is dominated even more significantly by its fundamental eigenmode, considering the difference between the stiffness at the superstructure level and that at the isolation level, which results in rigid body motion.

Consider the case of a damped single degree of freedom system shown in Figure A.1.

![Figure A.1: Single DOF system subjected to support ground motion.](image)

We apply Newton’s law to the free body diagram in Figure A.1 and use D’Alembert’s principle to obtain the governing equation of motion:

\[
M \dddot{u}(t) + C \dot{u}(t) + K u(t) = -M \dddot{u}_g(t) \tag{A.1}
\]

where \(M\), \(C\) and \(K\) are the mass, damping and stiffness of the structure, respectively, \(\dddot{u}(t)\), \(\dot{u}(t)\) and \(u(t)\) are the mass acceleration, velocity and displacement, and \(\dddot{u}_g(t)\) is the ground acceleration.

For a specific ground excitation, the equation of motion may be solved using different methods. The Newmark \(\alpha-\delta\) method, an implicit, direct integration method has been selected. In the special case of the method which is based on the trapezoidal rule, \(\alpha\) and \(\delta\)
have the values 0.25 and 0.50, respectively. Then, the method assumes that the velocity and displacement at time \((t+\Delta t)\) are given by

\[
\begin{align*}
    u(t + \Delta t) &= u(t) + \frac{\dot{u}(t + \Delta t) + \ddot{u}(t)}{2} \Delta t \\
    \dot{u}(t + \Delta t) &= \dot{u}(t) + \frac{\ddot{u}(t + \Delta t) + \dddot{u}(t)}{2} \Delta t
\end{align*}
\]

(A.2) (A.3)

Substitute (A.3) in (A.2) to obtain

\[
\begin{align*}
    u(t + \Delta t) &= u(t) + \dot{u}(t) \Delta t + 0.25 \frac{\Delta t^2}{2} (\ddot{u}(t) + \dddot{u}(t + \Delta t))
\end{align*}
\]

(A.4)

and finally

\[
\begin{align*}
    \ddot{u}(t + \Delta t) &= \frac{4}{\Delta t^2} \left( u(t + \Delta t) - u(t) - \Delta t \dot{u}(t) - 0.25 \Delta t^2 \dddot{u}(t) \right)
\end{align*}
\]

(A.5)

The resulting expression for the acceleration is substituted in (A.3) to obtain

\[
\begin{align*}
    \ddot{u}(t + \Delta t) &= -\dddot{u}(t) + 2 \left( \frac{u(t + \Delta t) - u(t)}{\Delta t} \right)
\end{align*}
\]

(A.6)

Finally, the last three expressions (A.4) to (A.6) are used in the equation of motion to solve for the displacement \(u\) at time \((t + \Delta t)\); which is given by

\[
\begin{align*}
    u(t + \Delta t) &= \left[ M \left\{ \frac{4}{\Delta t^2} u(t) + \frac{4}{\Delta t} \dot{u}(t) + \ddot{u}(t) - \frac{g}{\Delta t} (t + \Delta t) \right\} + C \left\{ \dot{u}(t) + \frac{2}{\Delta t} u(t) \right\} \right] + \frac{4M + 2C}{\Delta t^2 + 2C \Delta t + K}
\end{align*}
\]

(A.7)

Once this has been obtained under appropriate initial conditions, the displacement, velocity and acceleration of the system may be obtained iteratively, using Equations A.7, A.3 and A.5, respectively.

### A.2 Multi Degree of Freedom Systems

Most low- to medium-rise buildings may be modeled as shear buildings, assuming that their floors are sufficiently rigid so that they do not allow rotation about the horizontal axis, and that the columns are axially non-deformable. Consequently, the mass of the structure may be treated as lumped at the floor levels, having only three dynamic degrees of freedom per floor (two horizontal translations and a rotation about a vertical axis). The ‘shear building’ approximation assumes that the whole building deforms as a shear beam,
although the actual ‘internal’ deformations are mainly due to bending. In addition, if we
neglect eccentricities which result in torsional effects, and consider the earthquake excita-
tion to be only in one direction, the building may be modeled by lumped masses with one
translational degree of freedom per floor.

![Multi DOF system modeled as shear beam.](image)

When the structure is assumed to be linear elastic, its stiffness and damping properties
are considered constant and independent of the prior response history, i.e. the stiffness and
damping are independent of structural deformations.

The equation of equilibrium for a mass $M_i$ may be written as

$$
M_i \ddot{u}_{tot} + C_i \left( \ddot{u}_i - \dot{u}_i - 1 \right) - C_i + 1 \left( \ddot{u}_i + 1 - \dot{u}_i \right) + K_i \left( u_i - u_i - 1 \right) - K_i + 1 \left( u_i + 1 - u_i \right) = 0
$$

(A.8)

where the total displacement is equal to the sum of the relative and ground displace-
ments, i.e. $(u_{tot} = u + u_g)$.

The above equation holds for all masses $M_i$. We can therefore write the equilibrium
equation of the system in matrix form:

$$
[M] \dot{U}_{tot}(t) + [C] \dot{U}(t) + [K] U(t) = 0
$$

(A.9)

where $[U_{tot}] = [U(t + \Delta t)] + [e] u_g(t + \Delta t)$, with $[e]$ a unit vector. Thus,

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

(A.10)

The above system of differential equations may be solved using one of several numer-
ic methods. When the system is linear elastic, the mode superposition analysis method
may be used, since superposition of individual mode responses is allowed. When the prob-
lem involves non-linearities, a time step-by-step history analysis, such as the Newmark method, must be performed.

For the particular case of a 3-mass model shown in Figure A.2, the system matrices are:

\[
[M] = \begin{bmatrix}
m_1 & 0 & 0 \\
0 & m_2 & 0 \\
0 & 0 & m_3
\end{bmatrix}
\]

\[
[C] = \begin{bmatrix}
(c_1 + c_2) & -c_2 & 0 \\
-c_2 & (c_2 + c_3) & -c_3 \\
0 & -c_3 & c_3
\end{bmatrix}
\]

\[
[K] = \begin{bmatrix}
(k_1 + k_2) & -k_2 & 0 \\
-k_2 & (k_2 + k_3) & -k_3 \\
0 & -k_3 & k_3
\end{bmatrix}
\]

(A.11) (A.12) (A.13)

A.2.1 Modal Analysis

The response of a multi-DOF structure may be computed as the sum of the responses of independent single-DOF systems, corresponding to the eigenmodes of the structure. The system of differential equations of motion may be uncoupled by means of a transformation of coordinates which incorporates the orthogonality property of modal shapes.

\[
[M] \phi_i + [K] \phi_i = 0 \text{ when } i \neq j
\]

The eigenmodes of an undamped system may be computed for the free vibration of the system. In such a case the equation of motion of the system is

\[
[M] [\dot{U}] + [K] [U] = 0
\]

Assuming that the displacements may be expressed as

\[
[U] = [\phi] \sin(\omega t)
\]

\[
[K] [\phi] = \omega^2 [M] [\phi]
\]
The eigenproblem may be solved numerically to obtain the $N$ natural frequencies and mode shapes. Inverse iteration is used, combined with ‘sweeping’ for convergence purposes. The process is illustrated below:

**Figure A.3:** Inverse iteration procedure.

Introducing the transformation $[U] = [\Phi] [y]$, the system of equations becomes

$$
[M] [\Phi] \ddot{y} + [C] [\Phi] \dot{y} + [K] [\Phi] y = -[M] [e] \ddot{u}_g (t + \Delta t)
$$

(A.14)

Premultiplying by the transpose of each mode $\phi_i$ we obtain

$$
\phi_i^T [M] [\Phi] \ddot{y} + \phi_i^T [C] [\Phi] \dot{y} + \phi_i^T [K] [\Phi] y = -\phi_i^T [M] [e] \ddot{u}_g (t + \Delta t)
$$

(A.15)

Considering the orthogonality property of modes, and assuming that the system is damped proportionally to the mass and stiffness, the $(N \times N)$ system of equations may be uncoupled in a set of $N$-independent differential equations of the form
which may be simplified as

\[ m_i \ddot{y}_i + c_i \dot{y}_i + k_i y_i = -\left[ \phi_i^T \right] \begin{bmatrix} M \ & C \\ C \ & K \end{bmatrix} \left[ \phi_i \right] \ddot{u}_g(t) \] (A.17)

where \( m_i = \left[ \phi_i^T \right] M \left[ \phi_i \right] \) is the modal mass, \( c_i = \left[ \phi_i^T \right] C \left[ \phi_i \right] \) the modal damping, and \( k_i = \left[ \phi_i^T \right] K \left[ \phi_i \right] \) the modal stiffness.

By dividing each of the \( N \)-differential equations by \( m_i \), we obtain:

\[ \ddot{y}_i + 2 \xi_i \omega_i \dot{y}_i + \omega_i^2 y_i = -\Gamma_i \ddot{u}_g(t) \] (A.18)

where \( 2 \xi_i \omega_i = \frac{c_i}{m_i} \), \( \omega_i^2 = \frac{k_i}{m_i} \), and \( \Gamma_i = \left[ \phi_i^T \right] M \left[ \phi_i \right] \frac{c_i}{m_i} = \frac{L_i}{m_i} \).

We can simplify the derived equations further by introducing the transformation \( y_i = \Gamma_i \psi_i \), which transforms the set of \( N \) equations to the same form as the differential equation of motion of a single DOF system subjected to an earthquake excitation \( \ddot{u}_g(t) \):

\[ \ddot{\psi}_i + 2 \xi_i \omega_i \dot{\psi}_i + \omega_i^2 \psi_i = -\ddot{u}_g(t) \] (A.19)

The resulting displacements, velocities and accelerations may be computed via modal superposition as

\[
\begin{align*}
[U(t)] &= [\Phi] [\gamma(t)] = \sum_{i=1}^{N} [\phi_i] y_i(t) = \sum_{i=1}^{N} [\phi_i] \Gamma_i \psi_i(t) \\
[\dot{U}(t)] &= [\Phi] [\dot{\gamma}(t)] = \sum_{i=1}^{N} [\phi_i] \dot{y}_i(t) = \sum_{i=1}^{N} [\phi_i] \Gamma_i \dot{\psi}_i(t) \\
[\ddot{U}(t)] &= [\Phi] [\ddot{\gamma}(t)] = \sum_{i=1}^{N} [\phi_i] \ddot{y}_i(t) = \sum_{i=1}^{N} [\phi_i] \Gamma_i \ddot{\psi}_i(t)
\end{align*}
\] (A.20-22)
\[ \begin{bmatrix} \ddot{U}_{\text{total}}(t) \end{bmatrix} = \begin{bmatrix} \ddot{U}(t) \end{bmatrix} + \begin{bmatrix} \ddot{U}_g(t) \end{bmatrix} = \sum_{i=1}^{N} \phi_i \Gamma_i \ddot{\psi}_i(t) + \begin{bmatrix} \ddot{U}_g(t) \end{bmatrix} \quad (A.23) \]

The forces may then be computed from the displacements as
\[ \begin{bmatrix} F \end{bmatrix} = \begin{bmatrix} K \end{bmatrix} \begin{bmatrix} \ddot{U} \end{bmatrix} = \begin{bmatrix} K \end{bmatrix} \sum_{i=1}^{N} \phi_i \Gamma_i \ddot{\psi}_i = \sum_{i=1}^{N} \Gamma_i \ddot{\psi}_i \omega_i^2 \begin{bmatrix} M \end{bmatrix} \begin{bmatrix} \phi_i \end{bmatrix} \quad (A.24) \]

Finally, the base shear force then be calculated as the sum of the horizontal forces acting on the structure.
\[ Q = \sum F_i = \begin{bmatrix} e \end{bmatrix}^T \begin{bmatrix} F \end{bmatrix} = \sum_{i=1}^{N} \Gamma_i \ddot{\psi}_i \omega_i^2 \begin{bmatrix} e \end{bmatrix}^T \begin{bmatrix} M \end{bmatrix} \begin{bmatrix} \phi_i \end{bmatrix} = \sum_{i=1}^{N} \left( \frac{L_i^2}{m_i} \right) \ddot{\psi}_i \omega_i^2 \quad (A.25) \]

### A.2.2 Newmark Method

The Newmark method may be used to perform a time history analysis of a multi-DOF system. For the case where \( \alpha = 0.25 \) and \( \delta = 0.50 \), the expressions for the displacement, velocity and acceleration vectors, are the same as those for a single-DOF system, but in vector form. The governing equation of motion finally obtained is:
\[ \begin{bmatrix} M \end{bmatrix} \left( \frac{4}{\Delta t^2} \begin{bmatrix} U(t + \Delta t) \end{bmatrix} - \begin{bmatrix} U(t) \end{bmatrix} - \begin{bmatrix} \ddot{U}_t \end{bmatrix} \Delta t - 0.25 \Delta t^2 \begin{bmatrix} \dddot{U}(t) \end{bmatrix} \right) + \begin{bmatrix} C \end{bmatrix} \left( \frac{2}{\Delta t} \begin{bmatrix} U(t + \Delta t) \end{bmatrix} - \begin{bmatrix} U(t) \end{bmatrix} \right) \begin{bmatrix} U(t) \end{bmatrix} + \begin{bmatrix} K \end{bmatrix} \begin{bmatrix} U(t + \Delta t) \end{bmatrix} = - \begin{bmatrix} M \end{bmatrix} \begin{bmatrix} e \end{bmatrix} \ddot{U}_g(t + \Delta t) \quad (A.26) \]

Equation A.26 can be solved for \( U(t + \Delta t) \) :
\[ \begin{bmatrix} U(t + \Delta t) \end{bmatrix} = \left\{ \frac{4}{\Delta t^2} \begin{bmatrix} M \end{bmatrix} + \frac{2}{\Delta t} \begin{bmatrix} C \end{bmatrix} + \begin{bmatrix} K \end{bmatrix} \right\}^{-1} \left\{ \begin{bmatrix} M \end{bmatrix} \begin{bmatrix} U(t) \end{bmatrix} + \frac{4}{\Delta t} \begin{bmatrix} \dddot{U}_t \end{bmatrix} - \begin{bmatrix} \dddot{U}_g(t + \Delta t) \end{bmatrix} \begin{bmatrix} e \end{bmatrix} \dddot{U}_g(t + \Delta t) \right\} \quad (A.27) \]
Appendix B

Bilinear Systems

Many isolation systems are characterized by a highly nonlinear behavior and hysteretic dissipation of energy through cycles of loading and unloading. The most commonly used isolation systems, such as lead rubber bearings, behave bilinearly. The lumped mass models described in Appendix A may be easily modified to consider local non-linearities.

B.1 Single Degree of Freedom Systems

The equation of motion of a bilinear single-DOF system is given by

\[ M \ddot{u}(t) + C(t) \dot{u}(t) + K(t) u(t) = -M \ddot{g}(t) \]  

(B.1)

where the stiffness \( K \) and the damping \( C \) of the single DOF system are not constant anymore but depend on the previous response history of the system.

The stiffness \( K \) at any time \((t+\Delta t)\) is based on the prior response history of the system. The stiffness may be determined at each step, from which the damping may be easily computed. Assume that a particular damping ratio \( \xi \) is given. The damping is expressed as \( c = 2 \xi \omega M \), where \( \omega = \sqrt{\frac{K(t+\Delta t)}{M}} \).

A time history analysis may now be performed, to compute the displacement as

\[ u(t+\Delta t) = \frac{M \left\{ \frac{4}{\Delta t^2} \frac{u(t)}{\Delta t} + \frac{4}{\Delta t} \ddot{u}(t) + \ddot{u}(t) - \ddot{g}(t+\Delta t) \right\} + C(t) \left\{ \dot{u}(t) + \frac{2}{\Delta t} \frac{u(t)}{\Delta t} \right\}}{\left( \frac{4}{\Delta t^2} + \frac{2}{\Delta t} \frac{C(t)}{\Delta t} + \frac{K(t)}{\Delta t} \right)} \]  

(B.2)

The velocity and acceleration may be then determined using Equations A.3 and A.5.

B.2 Multi Degree of Freedom Systems

Consider now the case of a building modeled as a shear beam with lumped masses at the floors and local non-linearities at the isolation level due to the bilinear behavior of the isolation system. The model is shown in Figure B.1.
Figure B.1: Bilinear multi-DOF system

The equations of motion of such a bilinear multi-DOF system may be written as

\[
\begin{bmatrix}
F_I
\end{bmatrix} + \begin{bmatrix}
F_D
\end{bmatrix} + \begin{bmatrix}
F_E
\end{bmatrix} = \begin{bmatrix}
0
\end{bmatrix} \tag{B.3}
\]

where, at time \((t + \Delta t)\) we have the following forces:

- **Inertia forces:** \(\begin{bmatrix}
F_I(t + \Delta t)
\end{bmatrix} = [M][\ddot{U}(t + \Delta t)]\)

- **Damping forces:** \(\begin{bmatrix}
F_D(t + \Delta t)
\end{bmatrix} = [C(t + \Delta t)] [\ddot{U}(t + \Delta t)] \tag{B.4}\)

- **Deformation forces:** \(\begin{bmatrix}
F_E(t + \Delta t)
\end{bmatrix} = [K(t + \Delta t)] \left(\begin{bmatrix}
U(t + \Delta t)
\end{bmatrix} - [U(t)]\right) \tag{B.5}\)

For the 4-DOF model shown above, the system matrices are

\[
[M] = \begin{bmatrix}
0 & 0 & 0 & m_{isol} \\
0 & m_1 & 0 & 0 \\
0 & 0 & m_2 & 0 \\
0 & 0 & 0 & m_3
\end{bmatrix} \tag{B.6}
\]

\[
[C(t + \Delta t)] = \begin{bmatrix}
(c_{isol}(t + \Delta t) + c_1) & -c_1 & 0 & 0 \\
-c_1 & (c_1 + c_2) & -c_2 & 0 \\
0 & -c_2 & (c_2 + c_3) & -c_3 \\
0 & 0 & -c_3 & c_4
\end{bmatrix} \tag{B.7}
\]
The equations of motion may therefore be written in the form

\[
[M] \begin{bmatrix} \ddot{U}_{tot}(t) \end{bmatrix} + \begin{bmatrix} C(t + \Delta t) & \left[ U(t + \Delta t) - U(t) \right] \end{bmatrix} + \begin{bmatrix} F_E(t) + \left[ K(t + \Delta t) \right] \left( \left[ U(t + \Delta t) - U(t) \right] \right) \end{bmatrix} = \begin{bmatrix} 0 \end{bmatrix}
\]  

(B.9)

Substituting the expressions for the velocity and the acceleration, assumed in the Newmark method previously described, we obtain

\[
[M] \left( \frac{4}{\Delta t^2} \left( \left[ U(t + \Delta t) - U(t) \right] - \left[ \ddot{U}_{tot}(t) \right] \Delta t - 0.25 \Delta t^2 \left[ \dddot{U}_{tot}(t) \right] \right) \right) +
\]

\[
[C(t + \Delta t)] \left( 2 \left( \frac{\left[ U(t + \Delta t) - U(t) \right]}{\Delta t} \right) - \left[ \ddot{U}_{tot}(t) \right] \right) + F_E(t) +
\]

\[
[K(t + \Delta t)] \left( \left[ U(t + \Delta t) - U(t) \right] \right) = -\left[ M \right] \left[ \varepsilon \right] \dddot{U}_g(t + \Delta t)
\]

This may be used to solve for the displacement at time \((t+\Delta t)\):

\[
\left[ U(t + \Delta t) \right] = \left\{ \frac{4}{\Delta t^2} \left[ M \right] + \frac{2}{\Delta t} \left[ C(t + \Delta t) \right] + \left[ K(t + \Delta t) \right] \right\}^{-1}
\]

\[
\left\{ \left[ K(t + \Delta t) \right] \cdot \left[ U(t) \right] - F_E(t) + C \left( \left[ \ddot{U}_{tot}(t) \right] + \frac{2}{\Delta t} \left[ U(t) \right] \right) \right.
\]

\[
-\left[ M \right] \left[ \varepsilon \right] \dddot{U}_g(t + \Delta t) + \left[ M \right] \left( \frac{4}{\Delta t^2} \left[ U(t) \right] + \frac{4}{\Delta t} \left[ \dddot{U}_{tot}(t) \right] \right) \}
\]

Equation B.11 may be used to obtain the solution to the problem by computing the displacement, velocity and acceleration at each step. The issues of concern related to the bilinear behavior are the loading and unloading paths and the possibility of exceeding the yield force level. The loading and unloading paths are specified by the sign of the velocity of the mass at the isolation level. A change in velocity sign indicates reversion of the load-
ing or unloading paths, while if the yield force is exceeded, a reduction of stiffness of the isolation system, from the initial elastic stiffness $K_{isol,el}$ to the post-yield stiffness $K_{isol,pl}$ is implied. An algorithm has been developed to compute the exact point of either zero velocity or yield point, using a linear interpolation procedure. When a change in the sign of velocity occurs, a smaller integration time step is assumed and the iteration is repeated using a corresponding linearly interpolated value of ground acceleration. The process is repeated until the velocity is approximately equal to zero. The time history analysis then continues with an updated stiffness and damping matrices for the reversed path. Similarly, when the force at the isolation level exceeds the yield force, subsequent iterations are performed to find the time at which the force is approximately equal to the yield force. Once the time of yielding has been determined, the time history analysis continues with the updated post yield stiffness. Finally it may also be possible that both the yield force is exceeded and the velocity sign is reversed. In such cases, the earliest of the two events must be detected. An automatic procedure has been developed to perform such an analysis.