SEISMIC RESPONSE ANALYSIS OF ISOLATED AND NON-ISOLATED BRIDGES

BY

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I hereby declare that the research work reported in this thesis has been performed by me and that this work has not been submitted elsewhere for any other purpose, except for publications.

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ABSTRACT

Earthquake resistant bridge design must ensure that bridge piers withstand the lateral forces generated during earthquakes. In recent years, base isolation has emerged as an effective technique for reducing the lateral forces acting on bridge piers by placing suitable isolators in between the deck and pier. Base isolation results in increasing the natural period and damping characteristics of the bridge.

In this study the seismic response of bridge structures subjected to moderate and strong earthquakes is evaluated using nonlinear finite element method. Time history analysis is performed employing Newmark's step wise time integration. Both conventional non-isolated and isolated bridges are considered, but emphasis is primarily given on evaluating the seismic response of isolated bridges. A simple dynamic single pier model consisting of deck mass, isolator unit and pier is used after verification with a more detailed model of a four-span regular bridge. Complete bridge model is used when study is performed on transverse deck vibration. Longitudinal and transverse vibrations of the bridge are studied separately. For most part of the study non-linearity is restricted to the isolation unit, while rest of the structure is assumed to remain elastic at all times. However, non-linearity of pier is considered in the case where ductility requirements for bridge piers are studied, in which case the pier is simplistically modeled as a bi-linear elastic spring element. The study concentrates mainly on evaluating the peak shear forces, which the bridge pier needs to resist during earthquakes, and isolator displacement, which is important for design of isolation system and for allowing relative movement of the deck.

Extensive parametric studies are performed to show the influence of key bridge and isolator parameters on the seismic response of bridges. The distribution of bridge mass over its components, shows significant influence on pier force and ductility requirements for piers. For strong motions, the ductility requirements for piers are found to be reduced significantly with increase in pier period. However, it shows little or no effect on isolator displacement. Among the isolator parameters, isolator yield force and post-yielding stiffness of isolator are found to have considerable influence on peak shear force, ductility demand and transverse mid-span deck displacement.

As a case study, the recently commissioned 4.8 km long Bangabandhu (Jamuna Multipurpose) Bridge has been considered for seismic response analysis. This bridge has seismic pintles for protection against a design ground acceleration of 0.2g. Analysis results show that the pintles are effective in keeping the pier forces within the elastic limit.
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NOTATION

μ  Ductility demand
β  Newmark's operator
ε  Pier strength ratio
Δ_u  Maximum lateral deflection at the end of plastic deformation
Δ_v  Lateral deflection at the yield load
q  Vectors of increments of nodal displacement
q̇  Vectors of velocity increment
q̈  Vectors of increment of nodal acceleration
A  Acceleration coefficient
A_d  Deck cross sectional area
A_p  Pier cross sectional area
C_s  Seismic response coefficient
E_p  Elasticity of pier
F_y  Isolator yield force
F_{yp}  Pier yield force
\dot{g}  Acceleration due to gravity
H_p  Pier Height
\dot{\dot{q}}  Vectors of velocity increment
\dot{q}  Vectors of nodal acceleration
C  Damping matrix
K_E  Linear stiffness matrix
K_G  Geometric stiffness matrix
I_d  Deck moment of inertia
I_p  Pier moment of inertia
K_h  Post-yielding stiffness of isolator
K_i  Initial isolator stiffness
K_L  Stiffness in longitudinal direction
K_{pL}  Post yielding stiffness of pier
K_{pi}  Initial stiffness of pier
K_T  Equivalent Stiffness in Transverse direction
M  Consistent mass matrix
m_d  Deck mass
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<td>$\omega$</td>
<td>Natural frequency</td>
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<tr>
<td>$\gamma$</td>
<td>Newmark's operator</td>
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<td>$\delta$</td>
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<td>$V$</td>
<td>Base shear</td>
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CHAPTER 1
INTRODUCTION

1.1 GENERAL

From the past historical records of earthquake occurrence, it is seen that earthquake is one of the most feared natural disasters which has caused incalculable destruction of properties and injury and loss of lives to the population. Earthquakes occur due to the instability of the earth crust and the sudden release of accumulated stress deep inside the crust. The sudden release of energy during an earthquake may lead to ground shaking, surface faulting, ground failures and tsunamis. Stresses are generated in structures due the ground shaking and if a structure is incapable of resisting these additional stresses, it will suffer damage. The current philosophy behind earthquake resistant design of common structures is to ensure that:

- there are no damages (or only slight but repairable non-structural damage) due to design earthquakes and

- Collapse is prevented during more severe earthquakes, which is achieved by ensuring ductile, rather than brittle behavior of the structural response.

Collapse of structures, which do not have adequate seismic resistance, result in human deaths and injuries. The major cause of death or injuries due to earthquakes is the collapse of buildings or bridges. In case of urban areas, fires triggered by earthquakes (e.g. from short circuits, gas explosions) may prove to be more disastrous. For example, during the 1921 Kanto Earthquake in Tokyo-Yokohama, 100,000 people lost their lives due to fire.

The behavior of bridge structures under the influence of seismic load has been a major point of interest for engineers over a long period of time. The 1971 San Fernando earthquake was a major turning point in the development of seismic design criteria for bridges, in as much the same way as the 1933 Long Beach earthquake was for the earthquake response of buildings. Similarly, the 1929 Murchison earthquake in New Zealand altered the view prevailing then that earthquakes were of scientific interest only. Although significant advances have been achieved since that time in the design and construction of an earthquake resistant bridge, numerous gaps still remain in the understanding of the seismic behavior of bridges.

The damage to bridge structures during an earthquake may be classified into three categories: (i) Damage due to inadequate strength of foundation (ii) Damage due to soil liquefaction. (iii) Damage to columns and bearing support. In recent earthquakes, significant
damage due to shortage of foundation strength and effect of soil liquefaction did not occur in bridges designed and constructed in accordance with the recent design specification after 1971 (Kawashima et al, 1991). However, damages to reinforced concrete columns and bearing supports, which rarely suffered in earlier earthquakes, occurred. This is due to the fact that old failure modes such as tilting or movement of the foundations, soil liquefaction and falling-off of superstructures were prevented by the new design recommendations.

Through these evidences of seismic damage and effect of countermeasures, it is becoming apparent that certain types of failure which developed in recent years at bearing supports and column might not be prevented by only increasing the design lateral forces, as have been adopted for seismic design of highway bridges. It might be more effective to allow some relative displacement between deck and substructures for preventing the damage of bearings. Because lateral force developed during a destructive earthquake with the magnitude over 8.0 would be more than 3 times larger than the force level considered in the current seismic design criteria (Kawashima et al, 1991). Incorporation of structural members which develop stable energy dissipation under destructive seismic loading might be required. This is one of the motivations for incorporating seismic isolation device in highway bridges.

The basic intent of seismic isolation is to increase the fundamental period of vibration such that a structure is subject to lower earthquake forces. However, this reduction in force is accompanied by an increase in displacement which must be accommodated within the flexible support. To control these increased deflections within desirable limits, damping is included as part of the isolation system. Studies have shown that the cost of this isolation hardware can be offset by the savings in the substructures and foundations (because of reduced forces) and the long term reduction in repair costs for seismic damage. Bridge structure are particularly suitable for isolation and literature surveys indicate that more than 90% of the base-isolated structures in the world are bridge structures (Buckle and Mayes, 1990).

Seismic Isolation system for bridges has special significance in Bangladesh. Several earthquakes of large magnitude (Richter magnitude 7.0 or higher) with epicenters within Bangladesh and in India close to Indo-Bangladesh border have occurred (Ali and Choudhury, 1994). Table 1.1 provides a list of these major earthquakes that have affected Bangladesh. Moreover, there are faults within Bangladesh and in neighboring India and Burma that may be sources of earthquakes affecting Bangladesh (Fig.1.1). Table 1.2 (Ali and Choudhury, 1992) shows the probable magnitudes of operational basis earthquakes and maximum credible earthquakes, along with depth of focus in these fault zones. Furthermore, the country is divided into three zones determined from the earthquake magnitude for various return periods and the acceleration attenuation relationship (Ali and Choudhury, 1994) namely
zones 1, 2 and 3, with zone 3 and zone 1 being the most and least severe respectively (Fig 1.2). This has been included in the Bangladesh National Building Code (BNBC, 1993). These information clearly signifies that the probability of occurrence of earthquakes of large magnitudes is considerable in this country.

The 4.8km Bangabandhu (Jamuna Multipurpose) Bridge, which has been commissioned in June, 1998, was designed for a 0.2g ground acceleration input. Seismic pintles have been used which act as isolation device in the event of major earthquakes. With the envisaged construction of large number of bridges like Bhairab Bridge, Padma Bridge, Paksey Bridge, the economic seismic design of such bridges and probable use of base isolation device may become an important issue.

Table 1.1 List of Major Earthquakes Affecting Bangladesh (After Ali and Choudhury, 1992)

<table>
<thead>
<tr>
<th>Date</th>
<th>Name of Earthquake</th>
<th>Magnitude (Richter)</th>
<th>Epicentral Distance form Dhaka (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 January, 1869</td>
<td>Cachar Earthquake</td>
<td>7.5</td>
<td>250</td>
</tr>
<tr>
<td>14 July, 1885</td>
<td>Bengal Earthquake</td>
<td>7.0</td>
<td>170</td>
</tr>
<tr>
<td>12 June, 1897</td>
<td>Great Indian</td>
<td>8.7</td>
<td>230</td>
</tr>
<tr>
<td></td>
<td>Earthquake</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8 July, 1918</td>
<td>Srimongal Earthquake</td>
<td>7.6</td>
<td>150</td>
</tr>
<tr>
<td>3 July, 1930</td>
<td>Dhubri Earthquake</td>
<td>7.1</td>
<td>250</td>
</tr>
<tr>
<td>15 January, 1934</td>
<td>Bihar-Nepal Earthquake</td>
<td>8.3</td>
<td>510</td>
</tr>
<tr>
<td>15 August, 1950</td>
<td>Assam Earthquake</td>
<td>8.5</td>
<td>780</td>
</tr>
</tbody>
</table>

Table 1.2 Tectonic Provinces and their Earthquake Potential (After Ali and Choudhury, 1992)

<table>
<thead>
<tr>
<th>Location</th>
<th>Operating Basis Magnitude (Richter)</th>
<th>Maximum Credible Magnitude (Richter)</th>
<th>Depth of focus (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Assam fault zone</td>
<td>8.0</td>
<td>8.7</td>
<td>0-70</td>
</tr>
<tr>
<td>Tripura fault zone</td>
<td>7.0</td>
<td>8.0</td>
<td>0-70</td>
</tr>
<tr>
<td>Sub-Dauki fault zone</td>
<td>7.3</td>
<td>7.5</td>
<td>0-70</td>
</tr>
<tr>
<td>Bogra fault zone</td>
<td>7.0</td>
<td>7.5</td>
<td>0-70</td>
</tr>
</tbody>
</table>
Fig 1.1 Faults in the Precambrian and Depth Contour of Basement Inside Bangladesh (After Ali and Choudhury, 1992)
Fig 1.2 Seismic Zoning Map of Bangladesh (After Ali and Choudhury, 1992)
1.2 OBJECTIVES OF THE RESEARCH

The principal objectives of the present research are:

- To conduct a thorough literature survey on available information on bridge dynamics and base isolation devices used for protection of bridges.

- To perform non-linear dynamic analysis on appropriate dynamic models of bridge, subjected to a variety of earthquakes, differing in magnitude and frequency content.

- To examine the effectiveness of isolation system as an earthquake protection device through comparisons between isolated and non-isolated bridges.

- To perform extensive parametric study of the influence of isolator properties and bridge characteristics on the dynamic response of bridges.

- To study the requirements for ductility demand of piers under isolated and non-isolated conditions considering non-linear behavior of column.

- To examine the effect of vertical ground motion associated with horizontal ground shaking in an earthquake on the peak response of bridges.

- To study the earthquake response of Bangabandhu (Jamuna Multipurpose) Bridge using simplified dynamic models subjected to the design earthquake.

1.3 SCOPE OF THE WORK

This research work involves numerical study of seismic response of bridges with and without seismic isolation devices. However, the emphasis is primarily given on isolated bridges. A non-linear finite element program was used in the study. The bridge was modeled using beam-column element with lumped masses and the Isolation unit was represented by non-linear truss elements. Time history analysis was performed using Newmark’s β-method.
The study concentrated mainly on evaluating the peak shear forces which the bridge pier needs to resist during an earthquake. Also, the peak isolator displacement is estimated which is important for design of isolation system. The peak deck vibration is studied for the case of transverse motion.

Both linear and non-linear behavior of bridge pier is considered. Bridge piers are likely to sustain some plastic deformation in the event of severe earthquakes even for isolated bridges. Plastic (permanent) deformation is represented simplistically in this project by the ductility demand. The ductility demand of isolated bridges is compared with that of non-isolated bridges, in order to assess the effectiveness of the isolation system.

1.4 ORGANIZATION OF THE THESIS

In this study the results of research carried out have been divided into different topics and presented in seven chapters.

A brief introduction to the damage of bridges during earthquakes and seismic isolation as an earthquake protection device is presented in the first chapter with special emphasis on earthquake risks involved in Bangladesh. The objectives and scope of the work are also outlined in this chapter.

Chapter 2 presents the different aspects of dynamic behavior of bridges. It also presents the seismic design considerations for bridges and AASHTO design methods.

Chapter 3 deals elaborately with topics related to seismic isolation systems. Descriptions are provided in this chapter on basic elements of isolation system, its working principle and the benefits that are associated in using isolation systems in bridges. A few examples of different types of isolation systems currently used in bridges are also briefly described in this chapter.

In Chapter 4 the features and limitations of ANSR, the computer program used in this study for numerical analysis, are outlined. Brief descriptions are also provided on the solution technique used in the program.
The results of extensive parametric study on both isolated and non-isolated bridges are presented in Chapter 5.

In Chapter 6 the seismic response of Bangabandhu (Jamuna Multipurpose) Bridge is evaluated using design earthquake.

The conclusions of the entire study and some recommendations for further research are presented in Chapter 7.
CHAPTER 2
BRIDGE DYNAMICS AND DESIGN CONSIDERATIONS

2.1 GENERAL

The basic aim of seismic design, as in any engineering design, is to ensure that the resistance of the structure is greater than the loads applied to it. This is complicated in seismic design by the fact that earthquake loads are dynamic and not deterministic, i.e. they cannot be determined in an explicit manner in the same way that dead loads, vehicle loads and other environmental loads may be computed. It is therefore clearly important to be able to analyze a bridge for dynamic loads; the intent of this chapter is to outline basic principles of dynamic behavior of bridges and to present the procedures for determination of the magnitude of design loads, component design forces and associated detailed design requirements. Primary emphasis is given to the design philosophy and design requirements of the AASHTO Guide Specifications (AASHTO, 1991).

2.2 BASIC DYNAMICS

Dynamic loads are loads that vary with time. Bridges are subjected to several kinds of dynamic loads ranging from wind and vehicle effects to earthquakes. Response to these loads can be markedly different to that under static loads. It is possible that bridges that have repeatedly withstood static loads may collapse under dynamic loading of similar or smaller magnitude.

The essential difference between static and dynamic loads is the time varying nature of the dynamic loads. If the frequency content of the applied load is close to the frequency of vibration of the bridge, the structure will amplify the loading (Clough and Penzien, 1975; Biggs, 1964) and large and potentially destructive forces will be generated within the bridge. Therefore, problems arise when frequency matching occurs. This is the basis of all resonance phenomena. Load which is applied very slowly causes response which is virtually identical to static loading. On the other hand, cyclic load which is applied very rapidly has negligible effect on a structure (Biggs, 1964). Amplification of load only occurs when the rate of application of load is near one of the natural frequencies of one of the modes of vibration of for the bridge. Different bridges will therefore respond differently to the same load because their natural frequencies will be different.
2.2.1 SDOF Representation of Bridge

The study of simple models which approximate the behavior of actual bridge is a widely recommended procedure to analyze the dynamic response of bridges. The primary factors influencing response (weight, stiffness and damping) are more easily identified from these models. The simplest of all bridge models is the single mass model restrained by an elastic spring. In this model, the mass is assumed to have one degree of freedom i.e. it can move in only one direction. This assumption is reasonably accurate for the longitudinal (span wise) response of straight continuous bridges. However, its reliability for transverse behavior is conditional on several typical factors and it may not be satisfactory for complex bridges. Nevertheless, for typical highway structures, the single degree of freedom (SDOF) model is sufficiently accurate for design purposes, especially if care is taken in the selection of the equivalent mass and stiffness parameters. To illustrate the spring-mass model, a bridge example is shown in Fig. 2.1. The two span bridge has seat type abutments with sliding bearings to permit longitudinal movement but the superstructure is monolithic with a multi-column bent. Transverse shear keys are provided at each abutment to prevent transverse movements across the abutments and to lock the superstructure (deck) to the abutments in this direction. The abutments and the foundation structure below the pier are all assumed to be rigid in both directions.

**Longitudinal Mode**

Fig. 2.2(a) shows an elevation of this bridge and its equivalent spring-mass system. The mass \( M_L \), represents the total mass of the deck and representative fraction of the column mass.

The longitudinal movement of the superstructure mass, \( d \), becomes the displacement degree-of-freedom of the spring-mass system.

Spring constant (stiffness \( K_L \)) is given by the sum of the stiffnesses of all the components effective in this direction. Since free sliding is presumed to occur at the abutments, only the columns contribute to \( K_L \).
Fig 2.1 Bridge Modeling as SDOF System
$K_L = \text{stiffness in longitudinal direction}$

$M_L = \text{total mass of superstructure}$

*Fig 2.2 (a) Equivalent mode for Longitudinal Response*

$K_T = \text{equivalent stiffness is transverse direction}$

$M_T = \text{total mass of superstructure}$

*Fig 2.2 (b) Equivalent mode for Transverse Response*
Transverse Mode

Fig. 2.2(b) shows a plan view of the bridge and its equivalent spring-mass system. The effective mass, $M_T$, appropriate to this model needs to be calculated according to the method of analysis used.

The transverse movement of this effective mass, $d$, becomes the displacement degree-of-freedom of the spring-mass system.

If the abutments are both assumed to be perfectly rigid (infinite lateral stiffness) the deck does not displace as a rigid body (as in the longitudinal case) but rather deflects as a beam spanning from one fixed abutment to the other.

The lateral stiffness, $K_T$, is then a combination of the in-plane flexural stiffness of the superstructure and the lateral stiffness of the columns.

2.2.2 Damping

All structural systems exhibit damping to varying degrees and its effect on dynamic response is generally beneficial. Structural damping is assumed to be viscous by nature. If the damping coefficient, which relates force to velocity is sufficiently large, it is possible to totally suppress the oscillatory motion, is called the critical damping. A convenient measure of damping is then possible by comparing actual values of the damping coefficient with this critical value. This ratio is frequently expressed as a percentage and typical values for bridge structures fall in the range 2 to 10 percent. Although these might appear to be small values, they are nevertheless very important in controlling peak displacement and forces, especially near resonance and bringing the bridge to rest at the end of an earthquake. Since the natural frequency of a damped system is only slightly different from an undamped system, it is usual to neglect damping in frequency and period calculations unless it exceeds about 20 percent.

2.2.3 Period of Vibration

The term natural frequency is used to mean the frequency at which a bridge will vibrate freely, without being forced in any way. Free vibrations are most commonly measured or calculated by initially deflecting the structure, releasing it to vibrate without interference, and recording the time it takes for the bridge to complete a given number of cycles. The number of cycles per unit time is then a measure of the natural frequency. The reciprocal of frequency is the time the bridge takes to complete one cycle of vibration. Called the period of vibration, this
interval of time is used more commonly than frequency to describe a vibratory motion or a bridge response to excitation. The equation,

$$T = 2\pi \sqrt{\frac{M}{K}}$$ (2.1)

may be used to calculate this period for any vibrating system, including a bridge structure, considered as a single degree of freedom system.

2.2.4 Response Spectra

Earthquakes subject structures to time-varying forces which, in turn, produce time-varying displacements and stresses within these structures.

From a design viewpoint, only the maximum values of displacement and stress are of interest—the variation with time of these quantities is of little consequence. For survival, the structure must withstand the peak value whenever that may occur. Therefore, only the peak value to is needed to be known to successfully design a bridge and this information is made available in the form of response spectra.

It is possible to generate curves which give peak displacements for any structure subject to a given earthquake. These curves are called response spectra because they give the response (e.g. maximum displacement) of a wide spectrum of structures as defined by their frequency (or period) and damping ratio.

To generate spectra for real earthquake time histories, Duhamel's integral is commonly used (Clough and Penzien, 1975) and maximum values recorded and later plotted against frequency (or period) and damping ratio. Because there is no closed-form solution to the integral equation each point on each curve in the spectra is computed numerically.

Fig. 2.3 shows a typical response spectra for El Centro (1940 NS) earthquake.

2.2.5 Ductility Demand

Economic seismic design throughout the world in both bridges and buildings is achieved by permitting flexural yielding of the supporting piers. Flexural yielding in the piers implies deformation beyond the yield capacity of the pier. The extent of deformation beyond yield is referred to as the ductility demand on the pier. Consequently, it is important to understand the definition of ductility and what design parameters impact the ductility capacity of a column.
Fig. 2.3 Acceleration Spectra for the El Centro (1940 NS) Earthquake
Fig. 2.4(a) shows a single degree of freedom system responding elastically to an earthquake and the resulting load-deflection curve. Here, $b$ represents the maximum response of the system and the area $abc$ is a measure of the potential energy stored in the system at the time of maximum deflection. As the mass returns to the initial "at rest" position, this energy is converted into kinetic energy.

Now if the column is not strong enough to withstand the full elastic load implied by $b$, a plastic hinge will develop and the load-deflection curve will be as shown in figure 2.4 (b). When the limiting moment capacity is reached in the hinge, deflection proceeds along the path $de$ and $e$ in that case represents maximum displacement response. The potential energy stored in the system is given by “a-d-e-g” is dissipated in plastic deformation (mainly as heat) and is therefore irrecoverable. Hence, although the strength is less and plastic deformation implied a large deflection for negligible additional load, the maximum deflection of an elasto-plastic system is not significantly different to that of a pure elastic one. This is because less energy is being fed back into the system on return cycle.

Now, if ductility is expressed in displacement terms, a ductility factor $\mu$ may be defined by:

$$\mu = \frac{\Delta u}{\Delta y}$$

(2.2)

where, $\Delta u$ is the maximum lateral deflection at the end of plastic deformation and $\Delta y$ is the lateral deflection when yield in the column is first reached.

For majority of bridges, where ductility is provided by flexural plastic hinging of the columns, the ductility capacity will be limited by the ultimate displacement $\Delta u$ that can be sustained by the bridge columns without collapse. Definition of $\Delta u$ is somewhat subjective, but a recommended approach for reinforcing concrete members is to define $\Delta u$ as the displacement corresponding to either the first fracture of the confining reinforcement in a column plastic hinge (which results in rapid degradation of performance), or to a 20 percent drop in the lateral load capacity after the maximum strength has been reached.
(a) Elastic Response

(b) Elasto-Plastic Response

Fig. 2.4 Idealized Response of a Single Column Pier
2.3 BASIS FOR BRIDGE SEISMIC DESIGN PHILOSOPHY

For permanent loads (dead loads), or frequently occurring loads (live loads), engineering design is based on elastic principles so that the capacity of the structure is sufficient to resist all loads with a specified margin of safety. The magnitude of earthquake loads is such that this principle would be unrealistic for most bridges. Accordingly, a commonly accepted seismic design philosophy for bridges is as follows:

- For low to moderate earthquakes, which may be expected to occur several times throughout the life of a bridge, the structure is designed to resist these loads with only minor damage.

- For severe earthquakes which may occur once in the lifetime of a bridge, some structural damage is accepted but controlled so as to prevent collapse and preserve public safety. Where possible, damage that does occur should be readily detectable and accessible for inspection and, if feasible, repair.

These concepts can be illustrated by means of the simple bridge example shown in Fig. 2.5, and the two response spectra given in Fig. 2.6 (Buckle, Mayes and Button, 1987). The lower level spectrum in Fig. 2.6 is representative of a low to moderate earthquake whereas the higher level spectrum is representative of a more severe event at the same site which is assumed to be in a high seismic zone.

If the period of this bridge, with single column piers, is 0.3 sec. and if it is to remain elastic during the severe earthquake for the site, it will need to be designed for a lateral seismic shear force of 1.0W.

However, it is uneconomical to design bridge to remain elastic under such a high lateral load, and a reduced value is used instead. The consequential damage is accepted provided the total collapse is prevented and public safety is preserved.

The permitted reduction depends on the ability of the substructures to withstand this damage without collapse. For single column piers of the type illustrated in Fig. 2.5, a response reduction factor of 3 is judged appropriate. Therefore, design force spectrum for the column is one-third of the elastic spectrum as shown in Fig. 2.6 is an elastic spectrum for a low-to-moderate earthquake for the same site. It falls below the design curve for the column and therefore, the column will not be damaged (but will remain elastic) during this low-to-moderate event.
Fig. 2.5 Simplified Model for Calculation of Transverse Period of Vibration

\[ T = 2\pi \sqrt{\frac{W}{gK}} \]

\[ K = \frac{3EI}{H} \]
Fig. 2.6 Comparison of Realistic and Design Earthquake Forces
In many of the early bridge design codes the lateral design force (F) was expressed as a fraction of the weight (W) of the structure. This fraction was frequently called a design coefficient (C) which had values in the range of 0.03 to 0.15, depending on the period of the bridge and soil condition of the site. A comparison of this design coefficient against realistic force coefficient for the highest seismic zone of the AASHTO Guide Specification is shown in Fig 2.6. It is evident that the force given by these coefficients are significantly lower than those that can be realistically expected.

Following the catastrophic collapse of many bridges on the Golden State Freeway during the 1971 San Fernando earthquake near Los Angeles, the California Department of Transportation (Caltrans) made major revisions to the lateral force coefficient method. The most important change was to use realistic design forces and displacements. For comparative purposes, the forces and displacements in the new Caltrans design criteria are now similar to the higher level response spectra shown in Fig.2.6. This change resulted in significant increases in forces and displacements for the design of all bridge components. For example, the lateral force coefficient for a typical bridge was raised from 0.12 to above 1.0. However, reductions in these forces are permitted according to the importance, function and type of each component.

2.4 PAST PERFORMANCE IN EARTHQUAKES

Damage to bridge structures may occur in the superstructure, the substructure or the approaches. Typical types of damage are discussed below (Iwasaki et al,1973). Most failures occur from horizontal rather than vertical ground motion.

2.4.1 Superstructure

Loss of support for the girders is the most severe form of superstructure damage, and this may be caused by a lack of continuity in the superstructure, inadequate support lengths for the girders, skew supports which encourage rotation of the superstructure about vertical axis or gross movements at the superstructure supports due to some form of soil failure under the piers or abutments.

2.4.2 Substructure

Substructure damage generally manifests itself in the form of damage to columns, abutments and foundations (piles, footings). Column damage can be caused by flexural failure, shear failure, and anchorage failure of longitudinal reinforcement. These types of failure modes may also cause collapse of the superstructure by removal of support for the superstructure.
2.4.3 Foundations

Seismic damage, particularly to low bridges, is frequently caused by foundation failures which result from excessive ground deformation and/or loss of stability and bearing capacity of the foundation soils. As a result, substructures often tilt, settle, slide, or even overturn, thus experiencing severe cracking or complete failure.

2.4.4 Abutments

By virtue of their high lateral stiffness, abutments may attract the largest share of the seismic inertia forces developed in the superstructure. These forces can be very high and may cause severe failures, often of a brittle nature. The interaction of the abutment with the backfill may also cause the wing walls to break loose from the abutments.

2.5 CURRENT BRIDGE DESIGN CRITERIA

Since the 1971 San Fernando earthquake, the Federal Highway Administration has funded numerous research projects to improve the seismic design of bridges. These culminated in a contract to the Applied Technology Council of California to compile a new set of Design Guidelines based on the results of this research. Published in 1981, the ATC-6 Seismic Design Guidelines for Highway Bridges were adopted by AASHTO in 1983 as “Guide Specifications for the Seismic Design of Highway Bridges”. These specifications represent the state-of-the-art in seismic design for bridges and are recommended for the design of all new bridges throughout the United States.

New Zealand and Japanese engineers have also refined and updated their seismic design criteria for highway bridges in recent years. As a consequence, the seismic design provisions in New Zealand Ministry of Works Highway Bridge Design Brief (New Zealand Ministry of Works and Development, 1985) and the Japanese Specifications (Japan Road Association, 1982) is also amended.

Conceptually, the Caltrans, New Zealand and Japanese seismic design approaches all employ a “force design” concept. The Japanese criteria incorporate the highest levels of design forces and therefore rely less on the ductility of the supporting columns.

In the New Zealand criteria, which also accepts the philosophy that it is uneconomic to design a bridge to resist a large earthquake elastically (without damage), bridges are designed to resist small-to-moderate earthquakes in the elastic range. Design earthquakes are represented by the upper curve in figure 2.6 which may be reduced by a displacement ductility
factor to determine design force levels. This factor performs a similar function to the reduction factors (R) allowed in other codes. The selection of R depends on the ability of the bridge substructure to withstand inelastic deformation and can range from 2 to 6 according to the judgement of the design engineer. The design philosophy is that columns be capable of resisting the higher forces by inelastic or ductile deformation. Thus flexural plastic hinging in the columns is acceptable but the New Zealand code attempts to prevent significant damage to the foundations and other joints. Consequently, as a second step in the design process, maximum forces resulting from plastic hinging in all columns are determined. These forces are then used for the design of all components connected to the columns including the foundations. Hence, critical elements in the bridge are designed to resist the maximum forces to which they will be subjected by flexural yielding of the columns in a large earthquake.

In the Caltrans approach the member forces are determined from an elastic design response spectrum for a maximum credible earthquake, similar to the upper curve in figure 2.6. Although there are several spectra in the Caltrans provisions, they are of the order of, or higher than, curve 1 in figure 2.6. The design forces for each component of the bridge are then obtained by dividing the elastic forces calculated using this curve, by a reduction factor (Z). The Z-factor is 1.0 and 0.8 respectively, for hinge restrainers and shear keys. These components are therefore designed for expected and greater-than-expected (in case of shear keys) elastic forces resulting from a maximum credible earthquake. Well-confined ductile columns are designed for lower-than-expected forces from an elastic analysis as the reduction factor Z-varies from 4 to 8. Thus, in figure 2.6, the column design forces would be obtained by dividing the upper curve by the Z-factor. This assumes that the columns can deform inelastically when the seismic forces exceed these lower design forces. The end result is similar to the New Zealand approach although the procedures used are quite different.

In the development of the AASHTO Guide Specifications, the assessment of many loss-of-span type failures in the past earthquakes was attributed in part to relative displacement effects. Relative displacements between adjacent superstructure segments arise from out-of-phase motion of different parts of a bridge, from lateral displacement and/or rotation of the foundations and differential displacements of abutments. Therefore, in the development of the AASHTO Guide Specification the design displacements were considered to be equally important to the design forces. Thus minimum support lengths at abutments, columns and hinge seats were specified; and for bridges in areas of high seismic risk, ties between non-continuous segments of a bridge are specified. The philosophy for forces in this AASHTO Specification is similar to that of Caltrans. That is, the bridge is analyzed using realistic forces
calculated from a realistic design spectrum and the component forces are then modified by dividing these forces by a reduction factor (R).

2.6 ECONOMIC CONSIDERATIONS

The design philosophy used for bridges clearly has economic implications. A bridge can be designed such that it will suffer only minor damage in a major earthquake if the upper level curve in figure 2.6 is used to design all the components elastically. However, the cost increase will be considerable. Thus, in the development of a design philosophy, clearly stated objectives are required if a compromise between cost and safety is required.

In the development of the AASHTO Guide Specifications, both acceptable and unacceptable types of damage were defined. Detailed design and analysis requirements were then developed to achieve these performance criteria as follows.

2.6.1 Acceptable Damage

The only form of acceptable damage in the piers is flexural yielding of the columns. A well designed and detailed steel or reinforced concrete column can be subjected to many cycles of flexural yielding without risk of collapse. Any resulting damage will be visible and repairable and therefore acceptable.

For concrete columns to be repairable, it is most important that the provisions for confinement of the flexural reinforcement be satisfied in the zones where flexural yielding is expected.

Nominal abutment damage may also be acceptable provided adequate seat widths are used to accommodate the larger movements. Such damage might include shear key failure (in the transverse direction) and/or backwall impact (in the longitudinal direction).

2.6.2 Unacceptable Damage

Loss of Girder Support. Clearly this is the most unacceptable form of damage. To minimize this potential mode of failure, minimum support lengths for the girders are specified.

Column Failure. The two types of reinforced concrete column failures that can lead to a catastrophic collapse are shear failures and pullout of the longitudinal reinforcement. A capacity design approach is adopted in the AASHTO Guide Specification to minimize the possibility of a shear failure. Pullout of the longitudinal reinforcement is addressed with detailed design provisions and the requirement to design connections for the maximum expected forces generated from flexural yielding in the columns.
Foundation Failure. This can manifest itself in several ways. Any damage that does occur will not be readily visible or easily repairable. As a consequence, the AASHTO Guide Specification minimizes the possibility of failure to occur at the foundation. It requires that all foundation structures be designed for maximum forces that can be transferred by the piers, assuming flexural yield in the piers.

Connection Failures. Connections are extremely important in maintaining the overall integrity of the bridge. Consequently, in the AASHTO Guide Specification particular attention is given to the displacement that occur at moveable supports. For the fixed connections, conservative design forces are specified. In addition, positive horizontal linkage is to be provided between adjacent sections of the superstructure.

Liquefaction Failure. Liquefaction of saturated granular foundation soils has been a major source of bridge failures during past earthquakes. Investigations indicated that liquefaction of foundation soils contributed to much of the damage, with loss of foundation support leading to major displacements of abutments and piers. The best design measure is to avoid deep, loose to medium-dense sand sites where liquefaction risks are high. Where dense or more competent soils are found at shallow depths, stabilization measures such as densification may be economical.

2.7 EARTHQUAKE LOADING FOR DESIGN

In the development of the design loads, it must be emphasized that the specification of earthquake ground shaking cannot be achieved solely by following a set of scientific principles, for the following reasons. First, the causes of earthquakes are still not well understood and experts do not fully agree as to how the available knowledge should be interpreted to specify ground motions for use in design. Second, to achieve workable bridge design provisions it is important to simplify the complex matter of earthquake occurrence and ground motions. Finally, any specification of a design ground shaking involves balancing the risk of that motion occurring against the cost to society of requiring that structures be designed to withstand that motion. Hence, judgment, engineering experience and political wisdom are as necessary as scientific knowledge.

In the AASHTO Guide Specifications, the design loads are expressed as a design coefficient or design response spectra which represent the expected realistic force levels for the site. These force levels are derived such that they have a 10 to 20 percent chance of being exceeded every 50 years and are a function of the acceleration coefficient and the site soil conditions.
2.7.1 Zoning Map and Acceleration Coefficient

The first step in the determination of the design loads is the use of seismic zoning or regionalization maps to determine the zone in which the bridge site is located. This defines the level of seismic risks to which the bridge will be exposed.

2.7.2 Design Earthquake Ground Motion

The determination of appropriate seismic design loads, although complex in reality has been significantly simplified for code application. To state the concept rather than providing a definition, the design ground motion for a location is the ground motion that the engineer should consider when designing a structure to provide a satisfactory degree of protection for life safety and to prevent collapse.

At present, the best workable tool for describing design ground shaking is a smoothed elastic response spectrum for single degree-of-freedom systems. Such a spectrum provides a quantitative description of both the intensity and frequency content of ground motion. Smoothed elastic response spectra for 5 percent damping are used as a basic tool for the representation of local ground condition.

2.7.3 Influence of Soil Condition on Ground Motion

At the present time there is a widespread agreement that the characteristics of ground shaking and the corresponding response spectra are influenced by:

- The characteristics of the soil deposits underlying the proposed site.
- The magnitude of the earthquake producing the ground motions.
- The source mechanism of the earthquake producing the ground motions.
- The distance of the earthquake from the proposed site and the geology of the travel path.

While it is conceptually desirable to include specific consideration of all four of factors listed above it is usually not possible to do so in a code environment because of the complexity of the problem.

The fact that the effects of local soil conditions on ground motion characteristics should be considered in structural design has long been recognized in many countries of the world. Most countries considering these effects have developed different design criteria for several
different soil conditions. Typically these criteria use up to three different soil condition which are also included in the AASHTO Guide Specification.

Soil Profile Type I is a profile with either:

- Rock of any characteristic, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 760 m/sec, or by other appropriate means of classification)

- Stiff soil conditions where the soil depth is less than 60 meter and the soil types overlying rock are stable deposits of sands, gravel, or stiff clays.

Soil Profile Type II is a profile with stiff clay or deep cohesionless conditions where the soil depth exceeds 60 meter and the soil types overlying rock are stable deposits of sands, gravel, or stiff clays.

Soil Profile Type III is profile with soft to medium-stiff clays and sands, characterized by 30 meter or more of soft to medium clays with or without intervening layers of sand or other cohesionless soils.

2.8 METHODS OF ANALYSIS

The determination of appropriate seismic design loads, although complex in reality, has been significantly simplified for code application. The AASHTO Seismic Guide Specification suggests for three procedures for analysis.
2.8.1 Statically Equivalent Seismic Force and Coefficient

The elastic seismic response coefficient $C_s$ used to determine the design forces is given by the dimension-less formula:

$$C_s = \frac{1.2AS}{T^{2/3}}$$  \hspace{1cm} (2.3)

Where $A$ is an acceleration coefficient given by seismic zoning map

$S$ is dimensionless coefficient for the soil profile characteristics of the site as given by table 1

$T$ is the period of the bridge in seconds.

<table>
<thead>
<tr>
<th>Soil Profile Type</th>
<th>I</th>
<th>II</th>
<th>III</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S$</td>
<td>1.0</td>
<td>1.2</td>
<td>1.5</td>
</tr>
</tbody>
</table>

*Table 2.1 Site Coefficient*

2.8.2 Single Mode Spectral Analysis

The single mode method described in the following steps may be used for both transverse and longitudinal earthquake motion.

**Step 1** consists of calculating the displacements $V_s(x)$ due to an assumed uniform loading $P_0$. The uniform loading $P_0$ is applied over the length of the bridge. It has units of force/unit length and is arbitrarily set equal to 1. The static displacement $V_s(x)$ has units of length.

In **step 2**, factors $\alpha$, $\phi$ and $\varphi$ are calculated from the following expressions

$$\alpha = \int V_s(x) \, dx$$ \hspace{1cm} (2.4)

$$\phi = \int W(x) \, V_s(x) \, dx$$ \hspace{1cm} (2.5)

$$\varphi = \int W(x) \, V_s(x)^2 \, dx$$ \hspace{1cm} (2.6)
where \( W(x) \) is the weight of the dead load of the bridge superstructure (force/unit length). The computed factors, \( \alpha, \phi \) and \( \varphi \), have units of \((\text{length}^2)\), \((\text{force-length})\), and \((\text{force-length}^2)\), respectively.

**Step 3** consists of calculating the period of the bridge using the expression:

\[
T = \frac{2\pi}{\sqrt{\varphi / P_0 g \alpha}} \tag{2.7}
\]

Where, \( g \) = acceleration due to gravity.

In **step 4** of the procedure the value of \( P_0(x) \), the intensity of the equivalent static seismic loading is determined as \( P_0(x) = W(x) \cdot V_0(x) \cdot C_S \cdot \phi / \varphi \)

Where \( C_S \) is calculated from Eq. (2.3)

In **step 5** of the procedure the loading \( P_0(x) \) is applied to the bridge superstructure to determine the resulting member forces and displacements.

### 2.8.3 Multimode Spectral Analysis

The multimode spectral analysis method applies to bridges with irregular geometry which induces coupling in the three coordinate directions within each mode of vibration. These coupling effects make it difficult to categorize the modes into simple longitudinal or transverse modes of vibration and, in addition, several modes of vibration will in general contribute to the total response of the structure. A computer program with space frame capabilities should be used to determine coupling effects and multimodal contributions to the final response. Motions applied at the support in any one of the two horizontal directions will produce forces along both principal axes of the individual members because of the coupling effects. For curved structures, the longitudinal motion shall be directed along a chord connecting the abutments and the transverse motion shall be applied normal to the chord.
CHAPTER 3

SEISMIC ISOLATION SYSTEMS USED IN BRIDGES

3.1 GENERAL

The principle of seismic isolation is to minimize the transmission of potentially damaging earthquake ground motions into a structure. This is achieved by the introduction of flexibility at the base of the structure in the horizontal direction while at the same time introducing damping elements to restrict the amplitude or extent of the motion caused by the earthquake somewhat akin to shock absorbers. In recent years this relatively new technology has emerged as a practical and economic alternative to conventional seismic strengthening. This concept has received increasing academic and professional attention and is being applied to a wide range of civil engineering structures. To date there are several hundred bridges in New Zealand, Japan, Italy, Iceland and the United States which use seismic isolation principles and technology for their seismic design.

3.2 HISTORICAL BACKGROUND

The concept of separation or isolation of a structure from the damaging effects of an earthquake is not new; the first patent for such a system was filed in 1909 in England. Numerous proposals have been made since that time but until very recently, all have been judged impractical.

Early concerns were focused on the fear of uncontrolled displacements at the isolation interface, but these have been largely overcome with the successful development of mechanical energy dissipators. When used in combination with a flexible device an energy dissipator can control the response of an isolated structure by limiting both the displacements and the forces. Interest in base isolation as an effective means of protecting structures from earthquakes has therefore, been revived in recent years.

In summary, the following are the five recent developments that has enabled base isolation to be a practical reality:

30
1. The design and manufacture of high quality isolation bearings, that are used to support the weight of the structure and at the same time release it from earthquake induced forces.

2. The design and manufacture of mechanical energy dissipators (absorbers) that are used to reduce the movement across the bearings to practical and acceptable levels (4-6 inches) and to resist wind loads.

3. The development and acceptance of computer software for the analysis of base-isolated structure which includes nonlinear material properties and the time-varying nature of the earthquake loads.

4. The ability to perform shaking table tests using real recorded earthquake ground motions to evaluate the performance of structures and provide results to validate computer modeling techniques.

5. The development and acceptance of procedures for estimating site-specific earthquake ground motions for different return periods.

3.3 BASIC ELEMENTS OF BASE-ISOLATION SYSTEM

The prime objective of seismic isolation is to increase the fundamental period of vibration so that the structure is subject to lower earthquake forces. However, the reduction in force is accompanied by an increase in displacement demand which must be accommodated within the flexible mount. Furthermore, longer period bridges can be lively under service loads.

There are therefore three basic elements in a bridge isolation system, as follows:

1. A flexible mounting so that the period of vibration of the bridge is lengthened sufficiently to reduce the force response.

2. A damper or energy dissipator so that the relative deflections across the flexible mounting can be limited to a practical design level.

3. A means of providing rigidity under low (service) load levels such as wind and braking force.
Flexibility: An elastomeric bearing is not the only means of introducing flexibility into a structure, but it certainly appears to be one of the most practical devices. Due to additional flexibility the period of the structure is elongated. From the acceleration response curve shown in Fig. 3.1, it may be observed that reductions in base shear occur as the period of vibration of the structure is lengthened. The extent to which these forces are reduced is primarily dependent on the nature of the earthquake ground motion and the period of the non-isolated structure.

Energy Dissipation: As noted above, the additional flexibility needed to lengthen the period of the structure will give rise to large relative displacement across the flexible mount. Fig. 3.2 shows an idealized displacement response curve from which displacements are seen to increase with increasing period (flexibility). Large relative displacements can be controlled if substantial additional damping is introduced into the structure at the isolation level. This is shown schematically in Fig. 3.3. Also shown schematically in this figure is the smoothening effect of higher damping.

One of the most effective means of providing a substantial level of damping is through hysteretic energy dissipation. The hysteretic refers to the offset between the loading and unloading curves under cyclic loading. Figure 3.4 shows an idealized force-displacement loop where the enclosed area is a measure of the energy dissipated during one cycle of motion. Mechanical devices which use the plastic deformation of either mild steel or lead or simple friction to achieve this behavior have been tested, refined and are now included in several bridge structures.

Rigidity Under Low Lateral Loads: While lateral flexibility is highly desirable for high seismic loads, it is clearly undesirable to have a structural system which will vibrate perceptibly under frequently occurring loads such as wind loads or braking loads. Mechanical energy dissipators may be used to provide rigidity at these service loads by virtue of their high initial elastic stiffness. Alternately, some base isolation systems use a separate wind restraint device for this purpose—typically a rigid component which is designed to fail at a given level of lateral load, under strong earthquake.
ACCELERATION RESPONSE SPECTRUM

**Fig. 3.1 Idealized Force Response Curve**

DISPLACEMENT RESPONSE SPECTRUM

**Fig. 3.2 Idealized Displacement Response Curve**
Fig. 3.3 Response Curve for Increasing Damping
Fig. 3.4 Idealized Hysteresis Loop
3.4 DESIGN PRINCIPLES

The seismic design principles for base isolation are best illustrated by Fig. 3.5. The solid uppermost line (curve 1) is the realistic (elastic) ground response spectrum as recommended in the AASHTO Guide Specifications for the highest seismic zone (AASHTO, 1991). This is the spectrum that is used to determine actual forces and displacements to which a bridge will be subjected. The lowest solid line (curve 4) is the design curve from the AASHTO Standard Specification. It is seen to be approximately one-fifth of the realistic forces given by the Guide Specification. This reduction, to obtain the design forces, is consistent with a reduction factor of 5 for a multicolumn bent.

Also shown in Fig. 3.5 is curve (3), the probable overstrength of a pier bent designed to the AASHTO Standard Specification. This has been obtained by assuming an overstrength factor of 1.5. Curve (3) therefore represents the probable capacity of the bent.

The demand on this bent is represented by curve (1) and the difference between demand and capacity results in damage—possibly in the form of plastic hinging in the bridge piers. This difference is highlighted in Fig. 3.5 by the arrow and noted just above the legend for curve (1). If the bridge is isolated, the actual shear forces that the bridge will be subjected to may be represented by curve (2) (small dashed line). This curve corresponds to the same seismic input as curve (1), but it includes the effect of substantial level of damping inherent in hysteretic base isolation systems. The period of the isolated bridge may be in the 2.0 to 2.5 sec range and it is seen that in this range the overstrength (actual capacity) of the bent exceeds the realistic forces (demand) for the isolated bridge. This region has been shaded in Fig. 3.5. There is therefore no inelastic deformation or ductility requirement of the bent and elastic performance (without damage) is ensured.

3.5 BENEFITS OF SEISMIC ISOLATION

The benefits of seismic isolation for bridges may be summarized as follows:

- Reduction in the realistic forces to which a bridge will be subjected by a factor of between 5 and 10 (based on curves (1) and (2) of figure 3.5 and a shift in fundamental period, due to isolation, say from 0.4 s to 2 s.)

- Elimination of the ductility demand and hence damage to the piers during strong earthquakes.
DIFFERENCE BETWEEN DEMAND AND CAPACITY RESULTS IN DAMAGE - PLASTIC HINGE FORMATION

1. FORCE ON NON-ISOLATED BRIDGE IF SUFFICIENTLY STRONG (Demand Curve)
2. FORCE ON ISOLATED BRIDGE (Inelastic Spectrum)
3. PROBABLE STRENGTH OF STRUCTURE DESIGNED TO AASHTO STANDARD SPECIFICATION (Capacity Curve)
4. AASHTO DESIGN FORCE (Standard Specification)

RANGE OF FLEXIBILITY FOR ISOLATED BRIDGES \( \frac{\text{Increasing Flexibility}}{(Period)} \)

Fig. 3.5 Earthquake Forces
• Control of the distribution of seismic forces of the structure elements with appropriate sizing of the isolation bearings. In conventional bridge constructed with fixed bearings at some piers, horizontal forces concentrate at the time of an earthquake on those piers due to their high stiffness.

• Reduction in foundation design forces by a factor greater than 2.5 compared to conventional design (based on the fact that conventional design requires higher design forces for the foundations than for piers).

3.6 DIFFERENT TYPES OF ISOLATION SYSTEMS USED IN BRIDGES

A considerable amount of interest currently exists in the improvement of seismic isolation systems to provide greater earthquake resistance. In New Zealand, Italy, Japan and more recently, the United States, many innovative ideas have been implemented in this field. These ideas use the principles of restraint, isolation and energy dissipation to modify structural behavior during earthquakes. In each case these bearing systems also provide for the normal functions of bridge bearings. A few examples of the available isolation systems are discussed below.

3.6.1 Lead-Rubber Bearing

A lead-rubber bearing is an elastomeric bearing with a cylindrical lead plug inserted vertically along the center line of the unit (Fig.3.6). Elastomeric bearings are manufactured by inserting steel plates in rubber. Leading demands on a bearing are such that the bearing should be stiff with respect to vertical loads but flexible with respect to lateral loads. Its physical construction exploits the fact that rubber is the softest solid engineering material and has a Poisson's ratio close to 0.5, making it relatively incompressible. Under compression, the high bulk modulus causes the rubber to barrel out at the sides so that the volume remains constant. Thus, the more the volume can be restrained, the higher the vertical stiffness. The parallel steel plates, called shims, are used to restrain bulging and enhance the compression modulus of the bearing. The spacing of the shims is also important.

During cyclic shear deformations of the bearing, the lead core is forced to deform in shear also. Plastic shear deformations in the lead dissipate significant amounts of energy and
thereby limit the shear displacement in the bearing. Because lead has low rate of work hardening, it can sustain many cycles of imposed deformation due to thermal and creep movements without fracture.

The lead plug is composed of commercially pure lead to ensure that its mechanical properties are predictable. Overlength lead plugs are machined to size and are pressed into a pre-formed hole in the bearing. The center hole for the lead plug and the dowel holes are formed by mandrels during manufacture of the bearing unit. The lead cylinder is required to act integrally with the surrounding elastomeric bearing, and a tight plug is essential for this purpose, to ensure good energy dissipation.
3.6.2 Viscous Damping Device

Viscous damping devices function both as an energy absorber and as a reinforcement device that prevents the superstructure from sliding off the bridge pier. Viscous dampers were first developed in Japan in the 1960's with the original object of dispersing the horizontal force that acts on multi-span continuous bridge during earthquake (Kawashima et al., 1991). These viscous dampers were made of highly viscous materials such as hydrocarbon polymer, butane polymer and polyolefin polymer. But these materials are inappropriate for use as viscous materials for isolation device since they do not flow at room temperature and their viscosity is affected by temperature changes. Therefore, viscous dampers intended to use as isolator is developed using silicone oil as viscous material, which has good flowability and comparatively smaller variation in viscosity due to temperature change.

As shown in Fig 3.7 (a), viscous damper consists of a steel movable post that is secured to the superstructure, a top cover that is fastened to the steel movable post and seals the viscous materials together with a rubber box that is embedded and clamped in the substructure. The external box is fully packed with silicone oil, and a pressure regulating tank installed at the bottom end of the steel movable post is designed to stabilize the external box while the pressure of the silicone oil is floating.

The characteristics of viscous damper are as follows:

1) The resistance force of the damper is basically dependent only upon the relative velocity between the superstructure and the substructure, and increase as the velocity increases.

2) The damper also possesses the functions of a reinforcement device.

3) There are no design restrictions for displacement, except for the dimensional restrictions of the substructures.

4) The change in properties of the viscous material is minimal; stable damping performance is assured for long-term service.

Theoretically, the damping force of viscous dampers is dependent on vibrational velocity only: it is not influenced by displacement, vibration frequency, or the number of cyclic loads. In practice however, many factors other than vibrational velocity, such as pressure changes in viscous material due to changes in displacement or vibration frequency, or temperature rise as a result of repeated loading, may have an influence on the damping force.
Fig. 3.7(a) Details of Viscous Damping Device used in Japan

Fig. 3.7(b) Typical Force Displacement Hysteresis Loop of Viscous Damping Device
3.6.3 High Damping Rubber Bearing

The high damping laminated rubber bearing is basically a maintenance-free and isotropic isolation device, which can show all functions required as a isolator device, namely vertical load-bearing, horizontal restoring force and energy damping by its simple structure.

Base isolation bearings must bear large loads for long periods of time while also being capable of sustaining large horizontal deformation during an earthquake. Under these conditions rubber bearings must satisfy requirements of spring characteristics and ability to deform. One property of rubber material is that it degrades depending on the environment. Also, creep occurs when rubber is placed under load for a long period of time. Another important thing is that a damping function is indispensable to a seismic isolation system and this has, up to now, required a separate mechanism with low damping rubber bearing. If this function could be added to rubber, it would be great advantage from the standpoint of construction.

After several years of research in Japan, a high damping rubber was developed which is now used in the manufacturing of the high damping rubber bearing (Bridgestone Corporation,1993). High damping rubber bearings have two functions, flexibility and damping as an intrinsic property of the high damping rubber itself.

As can be easily imagined, there are many antipodal subjects to be satisfied simultaneously for the realization of a high-damping rubber bearing, in particular high-damping with low creep and low temperature dependent properties in the rubber. High creep yields large local stress and strain inside rubber bearings and may in extreme be responsible for the tilting of a structure. High temperature and velocity dependent properties, on the otherhand, change the stiffness and damping of rubber bearings over the use temperature and frequency range.

The high damping rubber bearing has quite a high-damping capacity, with smooth hysteresis loop shown in Fig. 3.8(b) (Nishikawa et al, 1991)
Fig. 3.8 (a) A typical Model of HDRB

Fig. 3.8 (b) Force Displacement Hysteresis Loop of HDRB
3.6.4 Friction Pendulum Systems

Friction bearing systems have been long used in bridge construction to accommodate motions due to thermal expansion and foundation settlement. A variety of seismic isolation systems have been developed along similar lines; typically, a Teflon faced slider slides along a flat stainless steel plate. This system is inherently able to sustain large loads and it displacements and dissipate dependably large amounts of hysteretic energy, and utilizes technology long employed in bridge construction. However, post-earthquake residual displacements are potentially large resulting in disruption of operation while the bridge is re-centered. Supplemental stiffening elements are usually added to such systems in order to reduce residual displacements. The design of such restoring force systems is often troublesome and their implementation complex.

The Friction Pendulum System offers a simple solution to these problems (Zayas et al, 1989). Figure 3.9(a) schematically shows a cross section through a FPS connection. The sliding surface is spherical in shape, rather than flat and the bearing material is supported by an articulated slider that conforms to the concavity of the sliding surface. Various bearing materials are available that can provide effective dynamic friction coefficients ranging between roughly 0.02 to 0.01 and greater (Mahin A.S., 1991).

This simple device provides ideal bilinear restoring force characteristics as shown in Fig.3.9(b). When activated, friction provides a restoring force proportional to the supported weight (R=μW). The spherical shape of the sliding surface raises the structure in a pendulum motion resulting in an additional restoring force which increases as the structure is displaced away from its original position. This latter term is proportional to the weight of the supported structure and the lateral displacement, δ, and inversely proportional to the radius of curvature, r, of the spherical surface. Thus, the total restoring force is given by:

\[ R = \mu W + W \frac{\delta}{r} \]  

(3.1)

This relation results in a number of desirable response characteristics. For example, the activated period of a (rigid) structure supported on FPS connections will not depend on the weight of the structure, but rather be governed by the relations for a pendulum. The period of the FPS isolator is given by:

\[ T_i = 2\pi \sqrt{\frac{r}{g}} \]  

(3.2)
where $g$ is the gravitational constant. This makes the inelastic response of the system relatively insensitive to variations in the supported mass. The dependence of the restoring force on the supported weight also results in the center of mass coinciding with both the centers of strength and stiffness of the activated structure. Thus, the activated system has little tendency to twist about a vertical axis.

The restoring force characteristics are reliably predicted by Fig 3.9(c) moreover, they can be easily altered in design by simply changing $r$ or $\mu$ as desired.

Another advantage of this relatively new device relative to conventional bridge bearings is that there is no stability problem at large horizontal displacement in the bearing and the bearing materials utilized allow much higher bearing stresses. These stresses usually range from 15 to 25 ksi, resulting in bearing areas considerably smaller than required with elastomeric isolators or conventional pot or spherical bearings.

### 3.6.5 Combinations

Steel dampers made of stainless steel are used in combination with other bearings. Its basic function is to absorb the energy of the earthquake through plastic deformation of the steel material. For the isolation system, the laminated elastomeric bearing mainly bears dead load, while the steel damper bears part of the horizontal force: it does not shoulder vertical forces. Stainless steel is characteristically more resistant to corrosion and has higher strength and elongation. The other characteristics of steel damper may be summarized as follows (Kamiya et al, 1991):

- Effective damping ratios of the circular steel damper are high compared to other systems. The effective damping ratio tends to increase as displacement is increased.
  - The effective stiffness of a circular steel damper tends to decrease as displacement increases.
- Both the effective stiffness and effective damping ratio of an oval steel damper are smaller than those of circular steel damper.
- The hysteresis loop of a circular steel damper is asymmetrical with respect to positive and negative fluctuations, while that of an oval steel damper still indicates the behavior of symmetric curve about positive and negative values even when the displacement is the same.
Fig. 3.9(a) Schematic of a FPS Connection

\[ T_c = 2\pi \sqrt{\frac{r}{g}} \]

\[ M = \frac{W}{g} \]

Fig. 3.9(b) Pendulum and Friction Action Resulting in Bilinear Restoring Force

\[ R = \frac{(W/r)}{\delta} \]

\[ R = \mu W \]

Fig. 3.9 (c) Typical Hysteresis Loops for a FPS Connector
Fig. 3.10(a) Shape of Steel Dampers

Fig. 3.10(b) Force Displacement Hysteresis Loop of Typical Circular Steel Damper
Figure 3.10 shows shapes and typical force displacement hysteresis loop for a steel damper.

Flat sliders (described in Sec. 3.5.4) in combination with restoring force devices have been employed in bridges. While flat sliders introduce considerable damping through frictional losses, the restoring or recentering devices are required to minimize residual displacements of the deck after an earthquake.

3.7 METHODS OF ANALYSIS FOR ISOLATED BRIDGE

The basic premise of seismic isolation design provisions is twofold. First, the energy dissipation of the isolation system can be expressed in terms of equivalent viscous damping; and second, the stiffness of the isolation system can be expressed as an effective linear stiffness. These two basic assumptions permit both single and multimodal methods of analysis to be used for seismic isolation design.

The subsequent sections describe the analysis procedure that has been incorporated in the new 1991 AASHTO Guide Specifications Seismic Isolation Design (Mayes et al, 1992).

3.7.1 Statically Equivalent Seismic Force and Coefficient

For the design of conventional bridges the form of the elastic seismic coefficient ($C_s$), reported in section 2.8, is

$$C_s = \frac{1.24S}{T^{2/3}}$$

(3.3)

The soil coefficient $S$ ranges from 1.0 to 1.5 for different soil types. $A$ is the Acceleration Coefficient and depends on the location of the bridge in the seismic risk map and $T$ is the fundamental period of vibration of the bridge.

For seismic isolation design, the elastic seismic coefficient is directly related to the elastic ground response spectra. This is because the intent of seismic isolation design is to introduce flexibility and damping in specifically designed and tested elements with the goal of eliminating or significantly reducing the ductility demand on the substructures. Consequently, the conservatism of the seismic coefficient required for long period (long span, tall column)
conventional bridges is not necessary for short span, regular column height isolated bridges. The form of the seismic coefficient is therefore slightly different from that for a conventional design and for 5% damping, is given by

\[ C_s = \frac{A S_i}{T} \]

In this case, \( A \) is acceleration coefficient, \( S_i \) is the site coefficient for seismic isolation design and the \( 1/T \) factor accounts for the decrease in the ground response spectra ordinates as \( T \) increases. The specific \( S_i \) values for the isolation requirements reflect the fact that above period of 1.0 second, there is a 1.0 to 1.5 to 2.0 relationship for the spectral acceleration for Soil Types I, II and III, respectively.

If the effects of damping are included, the elastic seismic coefficient is given by

\[ C_s = \frac{A S_i}{T B} \]  \hspace{1cm} (3.5)

Where \( B \) is the damping term for the isolation system and for 5% damping, \( B=1.0 \).

The quantity \( C_s \) is a dimensionless design coefficient, which when multiplied by \( g \) produces the spectral acceleration. This spectral acceleration \( (S_A) \) is related to the spectral displacement \( (S_D) \) by the relationship

\[ S_A = \omega^2 S_D \]  \hspace{1cm} (3.6)

Where \( \omega \) is the circular natural frequency and is given by \( 2\pi/T \). Therefore, since \( S_A = C_s \cdot g \)

\[ S_A = \frac{A S_i}{T B} g \]  \hspace{1cm} (3.7)

\[ S_D = \frac{9.79 A S T}{B} \]  \hspace{1cm} (3.8)
From Eqs. 3.6 and 3.7, putting for \( g = 386 \text{ in/sec}^2 \), the expression for \( S_0 \) follows. Denoting \( S_0 \) as \( D \), which is the displacement in inches across the elastomeric bearings,

\[
D = \frac{10AS/T}{B} \quad (3.9)
\]

An alternate form for \( C_s \) is possible. The quantity \( C_s \) is defined by the relationship

\[
F = C_sW \quad (3.10)
\]

Where \( F \) is the earthquake design force and \( W \) is the weight of the structure. Therefore,

\[
C_s = \frac{F}{W} = \frac{\sum K_{eff}D}{W} \quad (3.11)
\]

Where \( \sum K_{eff} \) is the sum of the effective linear springs of all isolation bearings supporting the superstructure segment. The equivalence of this form to the previous form is evident by recalling that \( \sum K_{eff} = \omega^2 \frac{W}{g} \), from which

\[
C_s = \frac{\omega^2W}{g} \frac{D}{W} \quad (3.12)
\]

Thus in summary, equations (3.9) and (3.11) are used to determine the statically equivalent seismic force. The isolated period of vibration is given by

\[
(3.13)
\]
The base shear \( V \), (which is equal to the statically equivalent seismic force) is obtained by substituting equations 3.9 and 3.13 into equation 3.10.

\[
V = F = \sum K_{eff} D
\]  

(3.14)

3.7.2 Single Mode Spectral Analysis

The single mode method of analysis given in AASHTO (described in section 2.8) is also appropriate for seismic isolation design. In fact, use of the method is simplified with seismic isolation. Steps 1, 2 and 3 of the procedure are not necessary since the use of an isolation system will ensure a simple rigid body deformation pattern of the superstructure.

In Step 4 of the procedure the value of \( P_s(x) \), the intensity of the equivalent static seismic loading, is determined as

\[
P_s(x) = w(x) \ast C_s
\]  

(3.16)

Where \( C_s \) is calculated by Equation (3.5) and \( W(x) \) is the dead load per unit length of the bridge superstructure. In Step 5 of the procedure the loading \( P_s(x) \) is applied to the superstructure to determine the resulting member forces and displacements.

3.7.3 Multimode Spectral Analysis

The multimode method of analysis given in AASHTO (described in section 2.8) is also appropriate for the response spectrum analysis of an isolated structure with the following modifications:

- The Isolation bearings are modeled by use of their effective stiffness properties determined at the design displacement.
• The ground response spectrum is modified to incorporate the damping of the isolation system.

3.7.4 Time History Analysis

In time history method of analysis, the response is evaluated for a series of short time increments \( \Delta t \), generally taken equal length for computational convenience. The condition of dynamic equilibrium is established at the beginning and end of each interval, and the motion of the system during the time increment is evaluated approximately on the basis of an assumed response mechanism (generally ignoring lack of equilibrium which may develop during the interval). The nonlinear nature of the system is accounted for by calculating new properties appropriate to the current deformed state at the beginning of each time increment. The complete response is obtained by using the velocity and displacement computed at the end of one computational interval as the initial conditions for the next interval; thus the process may be continued step by step from the initiation of loading to any desired time, approximating the nonlinear behavior as a sequence of successively changing linear system.
CHAPTER 4

METHODOLOGY FOR NUMERICAL ANALYSIS

4.1 GENERAL

Although design methods for bridges subjected to seismic loading allows for usual equivalent static loads, in many cases, it is preferred to have a truly dynamic response analysis. The response spectrum method described in chapter 2, is a widely used and accepted approximate method for dynamic analysis. For better results and for important bridges, time-history analysis may be conducted. Specially when some elements of the bridge show nonlinear behavior, emphasis would be more on time-history analysis.

This research embarks upon extensive time-history analysis with the objective of developing a better understanding of the complex dynamic behavior of bridges. The response is further modified by the introduction of isolation devices which impart both flexibility and damping during design earthquakes. A nonlinear finite element program is used to study the seismic response of both isolated and non-isolated bridges. Brief description are provided in this chapter on the main features of this program.

The validity of the results obtained depends considerably on effective modeling of the system. This chapter describes the simplified modeling techniques of actual bridge which may be used effectively for extensive parametric studies. Descriptions are also provided on the finite elements that have been used.

The characteristics of real earthquake records used in this study are also presented in this chapter.

4.2 NONLINEAR DYNAMIC FINITE ELEMENT ANALYSIS USING ANSR

4.2.1 Introduction to Finite Element Method

The development of the finite element method as an analysis tool was essentially initiated with the advent of the electronic digital computer. In the numerical solution of a continuum problem it is basically necessary to establish and solve a system of algebraic equations. Using the finite element method on digital computer it is possible to establish and solve governing equations for complex systems in a very effective way. It is mainly for the generality of the structure or continuum that can be analyzed, for the relative ease of
establishing the governing equations, and for the good numerical properties of the system matrices involved that the finite element method has found wide appeal.

In this project non-linear finite element analysis is performed to obtain the seismic response of isolated and non-isolated bridge.

4.2.2 The Program ANSR

In the last few decades, a considerable amount of effort has been invested in developing computer codes for nonlinear structural analysis using finite element. Most programs have been developed in research projects for achieving special purposes. Some of these programs perform dynamic analysis. There are some general purpose program such as MARC-CDC (Control Data Corporation, 1971) and ANSYS (Swanson Analysis Systems, Inc., 1973) are commercially available and are being used extensively. The program NONSAP (Bathe and Wilson, 1974) is also used, but to a more limited extent.

The program ANSR, developed at the University of California, Berkeley (Mondkar and Powell, 1975), is particularly suitable for rational investigation of nonlinear structures subjected to both static and dynamic forces. The program used in this study is a later (1992) PC-Version of ANSR with some modifications in the element library. Features and limitations of ANSR are listed below:

**Structural Idealization**

1. The structure to be analyzed is idealized as an assemblage of discrete finite elements connected at nodes. The theory and solution procedure are based on the finite element formulation of the displacement method, with the nodal displacement as the field variables.

2. Each node may possess up to six displacement degrees of freedom.

3. Provision is made for degrees of freedom to be deleted or combined.

4. The structure mass is assumed to be lumped at the nodes so that the matrix is diagonal.

5. Viscous damping effects may be included, if desired. Damping effects proportional to mass, initial stiffness and/or tangent stiffness can be specified.
**Static and Dynamic Loadings**

(1) Loads are assumed to be applied only at the nodes. Static and/or dynamic loads may be specified; however, static loads, if any, must be applied prior to the dynamic loads.

(2) For static analysis, a number of static force patterns must be specified. Static loads are then applied in a series of load increment, each load increment being specified as a linear combination of the static force pattern.

(3) The dynamic loading may consist of earthquake ground accelerations, time dependent nodal loads, and prescribed initial values of the nodal velocities and acceleration.

(4) Earthquake excitations are defined by time histories of ground acceleration. Three different time histories may be specified, one for each of the X, Y and Z axes of the structure.

(5) Any number of time histories of dynamic force may be specified. As with the earthquake records, these time histories may be specified to be at equal or unequal time intervals.

(6) Values of initial translation and/or rotational velocity and acceleration may be specified at each node. Structure subjected to impulsive loads can be analyzed by prescribing appropriate initial velocities.

### 4.3 FINITE ELEMENTS

Of the wide variety of finite elements used, the program PC-ANSR has very limited number of element types. The elements that are used for dynamic model of bridge are described in detail below.

#### 4.3.1 Three Dimensional Elastic & Inelastic Truss Element

Truss elements may be arbitrarily oriented in space, but can transmit axial load only (Fig 4.1a). Large displacement effects may or may not be included. When this effect is specified, it is included in both static and dynamic analysis.

Two alternative modes of inelastic behavior may be specified, namely (1) yielding in both tension and compression (Fig 4.1 b) and (2) yielding in tension and buckling in compression
Strain hardening effects may be considered. The inelastic behavior is stress strain based, rather than axial force and axial deformation. For computations, a strain hardening stress-strain relationship is decomposed into two components, one linearly elastic and the other elastic-perfectly plastic. A truss bar for linearly elastic behavior may be obtained by specifying a very high value of the yield stress. The length, area, elastic modulus and post-yielding modulus of the inelastic element are represented by L, A, E and $E_h$, respectively (Fig 4.1).

Initial axial forces in the truss elements can be specified. For combined static and dynamic analysis, these initial forces will typically be the forces in the elements under static loading, as calculated by a separate analysis.

4.3.2 Two Dimensional Elastic Beam-Column Element

Beam Column elements may be arbitrarily oriented in the global XYZ plane (Riahi et al, 1978).

Each element must be assigned an axial stiffness plus major axis flexural stiffness. Torsional and minor axis flexural stiffness may also be specified if necessary. Elements of variable cross section can be considered by specifying appropriate flexural stiffness coefficients.

Flexural shear deformations and the effects of eccentric end connections can be taken into account.

Yielding may take place only in concentrated plastic hinges at the element ends. Hinge formation is affected by the axial force and major axis bending moment only. An element may be placed in a three-dimensional frame, but its yield mechanism is only two-dimensional, in the plane of major axis bending.

Strain hardening is approximated by assuming that the element consists of elastic and elasto-plastic components in parallel. With this type of strain hardening idealization, if the bending moment in the element is constant, and if the element is of uniform strength, the moment-rotation relationship for the element will have the same shape as its moment-curvature relationship (Fig 4.2a). If however, the bending moment or strength vary, then the curvatures and rotations are no longer proportional, and the moment-rotation and moment-curvature variations may be quite different (Fig 4.2c).
Fig 4.1 (a) Truss Element

Fig 4.1 (b) Yield in Tension and Compression (Inelastic Truss Element)

Fig 4.1 (c) Yield in Tension Buckling in Compression (Inelastic Truss Element)
The beam column has three primary modes of deformation, namely (a) axial extension (b) flexural rotations in the major plane at ends i and j and (c) deformation due to nodal displacement. These three modes of deformation are shown in Fig 4.3.

Yield interaction surfaces of three types may be specified, but in this study only that for reinforced column type is used for analysis with the assumption that the bridge is made of reinforced column. Different types of yield interaction surfaces that are available in the program and their corresponding shape code are shown in Fig.4.4.

The element is considered as the sum of an inelastic component and an elastic component in the major plane of bending, plus a further elastic component providing torsional and minor axis flexural stiffness. This third component is needed to avoid singular stiffness matrices in certain circumstances. The axial stiffness is constant and is given by

$$d_{s1} = \frac{EA}{L} \nu_1$$

In which $E$= elastic modulus, and $A$= effective cross sectional area. The primary elastic flexural stiffness is given by,

$$\begin{bmatrix} d_{s2} \\ d_{s3} \end{bmatrix} = \frac{EI}{L} \begin{bmatrix} k_{i} k_{j} \\ k_{j} k_{i} \end{bmatrix} \begin{bmatrix} \nu_2 \\ \nu_3 \end{bmatrix}$$

In which $I$= reference moment of inertia and $K_{ii}, K_{ij}, K_{ji}$ are coefficients which depend on the cross section variation. For a uniform element, which is the case in this project $I$= actual moment of inertia, $K_{ii}= K_{ij}=4$, and $K_{ji}=2$. 

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Fig 4.2 Moment Curvature and Moment Rotation Relationship of Beam Column Element
Fig 4.3 Deformations and Displacements of Beam Column Element
Fig 4.4 Yield Interaction Surfaces for Beam Column Element
4.4 SOLUTION PROCEDURES

(1) The program incorporates a solution strategy defined in terms of a number of control parameters. By assigning appropriate values to these parameters, a wide variety of solution schemes, including step-by-step, iterative and mixed schemes, may be constructed. This permits the program user considerable flexibility in selecting optimal schemes for particular types of nonlinear behavior.

(2) For static analysis, a different solution scheme may be employed for each load increment.

(3) The dynamic response is computed by step-wise time integration of the incremental equations of motion using Newmark's $\beta$-$\gamma$-$\delta$ operator. A variety of integration operators may be obtained by assigning appropriate values to the parameters $\beta$ and $\gamma$ operator. The most commonly used scheme is the 'constant average acceleration' scheme with $\beta=0.25$, $\gamma=0.5$ and $\delta=0.0$. Viscous damping effects may be introduced by specifying a positive value to the parameter $\delta$. In most cases, damping effects will be introduced more explicitly, in mass dependent or stiffness dependent form.

4.4.1 Equations of Motion

The discrete incremental equations of motion for an undamped system are:

$$M \cdot \ddot{q} + [k_{-E} + k_{-G}] \cdot q = 2P - (M \cdot \dot{q} + R)$$  \hspace{1cm} (4.1)

Where $q$ and $\dot{q}$ are the vectors of increments of nodal displacement and acceleration, respectively; and $\ddot{q}$ is the vector of nodal acceleration. $M, k_{-E}, k_{-G}$ are lumped mass matrix, linear stiffness matrix, geometric stiffness matrix respectively and are obtained from the element matrices using well known assembly procedure (Zienkiewicz, O.C., 1971). $2P$ and $R$ denotes nodal load due to applied forces and state of stress.
The equation of equilibrium for static analysis can be obtained from equation (4.1) by omitting the terms containing accelerations. Viscous effects may be included by modifying equation (4.1) as follows:

\[
M \cdot \ddot{q} + C \cdot \dot{q} + [K_{E} + K_{G}] \cdot q = \ddot{p} - (M \cdot \dddot{q} + C \cdot \ddot{q} + R)
\]

(4.2)

In which \( \ddot{q} \) and \( \dot{q} \) are the vectors of velocity increment and velocity respectively; and \( C \) is a damping matrix.

In general, the structural response will be computed by applying the load in small steps, and in some cases equilibrium iterations may have to be carried out to obtain results with a sufficient degree of accuracy.

### 4.4.2 Solution Technique

Most solution procedures for nonlinear analysis can be classified as either step-by-step or iterative. Both procedures have been widely used in static nonlinear analysis, and both applicable to dynamic nonlinear analysis in which the response is computed by step wise marching in time.

In step-by-step solution procedure the load is applied in several small steps and the structure is assumed to respond linearly within each step, the response being obtained without iteration.

Two types of iterative procedure are commonly used, namely Newton-Raphson iteration and Constant Stiffness Iteration. In Newton-Raphson iteration the structure tangent stiffness matrix is reformed at every iteration, and a disadvantage of this procedure is that a large amount of computational effort may be required to form and decompose the stiffness matrix. In constant stiffness iteration, the stiffness matrix is formed only once. Although this procedure has the advantage that the tangent stiffness matrix is not formed and inverted at every iteration, its disadvantage is that iteration will typically converge more slowly than Newton-Raphson iteration, and schemes to accelerate convergence may be desirable.
The computer program includes the "alpha constant" acceleration scheme (Nayak and Zienkiewicz, 1971). In this scheme the displacement increments during any iteration are scaled in an attempt to obtain the same result as if Newton-Raphson iteration were employed. For each iteration the scheme requires two steps of displacement computation, and two steps of residual load computation.

In this study, the constant stiffness iteration has been used.

4.4.3 Integration of Equation

For integration of equations of motion (4.2), the time domain is divided into a number of time steps, and it is required to compute the displacements, velocities and accelerations in configuration at time $\tau = t + \Delta t$ with the knowledge of the previous deformation history from time 0 to time $t$. An implicit, single step, two parameter ($\beta, \gamma$) family of integration operators has been described by Newmark (Newmark, N.M., 1959), in which it is assumed that the increments in velocity and acceleration are related to increment in displacement and the state of motion at time $t$. A number of operators can be obtained by specifying various values of the parameters $\beta$ and $\gamma$. The "constant average acceleration" operator with $\beta = 1/4$, $\gamma = 1/2$ has been shown to be unconditionally stable for linear analysis. Accuracy and stability for nonlinear analysis can be studied only by numerical experimentation. It is possible to introduce artificial viscous effects by specifying a damping parameter $\delta$. With Newmark's operator, an integration algorithm has been designed in which iterations are performed within a time step to satisfy equilibrium subject to a specified tolerance. In this study, $\beta = 1/4$, $\gamma = 1/2$, $\delta = 0$ have been used.
Fig 4.5 Constant Stiffness Iteration
4.5 BRIDGE MODELING

For dynamic analysis of bridges it is common to use a simple model that would portray the basic dynamic features of the bridge. In published literature (Ghobarah and Ali, 1988; Hosoda et al., 1991; Spyrokos, 1990; Shimada et al., 1991; Takeda et al., 1991; Turkington et al., 1989a and 1989b;) dynamic models comprising of beam-column elements, spring elements and lumped masses have been used.

During an earthquake a bridge may vibrate in both longitudinal and transverse direction. The longitudinal and transverse vibration of bridge are often treated separately. This study also treats the longitudinal and transverse motions separately.

A three span highway bridge is such as shown in Figure 4.6 is considered for dynamic modeling. The bridge is reinforced concrete box type that is continuous over three spans and is supported by two single column piers. The foundations of abutments and piers are rigid footings, and they are constructed on firm ground. Isolation device is located between the pier and deck. Fig. 4.7 and 4.8 shows the same bridge modeled by the simplified frame model in longitudinal and transverse direction of earthquake loading respectively. Both superstructure and substructures are modeled in beams and lumped masses. Substructures are supported on elastic shear and rocking springs of the ground. Isolation device represented by spring elements have bi-linear characteristics as shown in Fig. 4.9. It has large stiffness of $K_1$ up to yield point A. Then the stiffness decreases to $K_h$ at the place exceeding the yielding displacement (point A). After unloading, the stiffness $K_l$ is regained.
Fig 4.6 Typical Three Span Highway Bridge

Fig 4.7 Modeling of the bridge structure in longitudinal direction

Fig 4.8 Modeling of bridge structure in transverse direction
Fig 4.9 Isolator Force Displacement Relationship
4.6 EARTHQUAKE RECORDS USED IN THE STUDY

To account for the uncertainty in the characteristics of earthquake induced ground motions regarding site conditions, intensity, and frequency content, actual earthquake records are used as input time histories. Six different earthquake records were selected. The first ground motion considered is El Centro Earthquake NS component of May, 1940 with peak acceleration of 0.35g. This is a typical low-frequency California earthquake excitation, which has been used quite extensively in earthquake engineering studies. The San Fernando earthquake of February 1971 is another well known earthquake. The S74W component of the acceleration time history recorded adjacent to Pacoima dam abutment has a maximum peak horizontal acceleration of 1.075g. The Taft Lincoln School Tunnel record of July, 1951 earthquake at the Kern County is another California earthquake. Its N21E component has a maximum acceleration of 0.16g. The 1989 Santa Cruz Mountains (Loma Prieta) earthquake measured at Eureka Canyon road has a peak acceleration of 0.63g. Kobe and Hachinohe are the two Japanese earthquakes that have been used in this study. The 1995 Kobe is one of the more recent major earthquakes. Its NS component has a peak acceleration of 0.83g. The NS component of long period motion Hachinohe has peak acceleration of 0.23g.

Table 4.1 List of Earthquake Motion Used

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Record Description</th>
<th>Magnitude (Richter)</th>
<th>Peak Ground Acceleration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Taft</td>
<td>Kern County, July 1952</td>
<td>6.7</td>
<td>0.15g</td>
</tr>
<tr>
<td></td>
<td>Component N21E</td>
<td></td>
<td></td>
</tr>
<tr>
<td>El Centro</td>
<td>Imperial Valley, May 1940</td>
<td>7.2</td>
<td>0.35g</td>
</tr>
<tr>
<td></td>
<td>Component NS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loma Prieta</td>
<td>Loma Prieta, Oct. 1989</td>
<td>7.1</td>
<td>0.63g</td>
</tr>
<tr>
<td></td>
<td>Corralitos-Eureka Canyon Road</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Component 0°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kobe</td>
<td>Kobe, Japan, Jan. 1995</td>
<td>6.9</td>
<td>0.83g</td>
</tr>
<tr>
<td></td>
<td>Component NS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hachinohe</td>
<td>Tokachi-Oki Earthquake, Japan, May 16, 1968. Component NS</td>
<td>7.9</td>
<td>0.23g</td>
</tr>
<tr>
<td>Pacoima</td>
<td>San Fernando, Feb. 9, 1971</td>
<td>6.4</td>
<td>1.075g</td>
</tr>
</tbody>
</table>
Fig. 4.10 to 4.15 show time histories of ground motion and response spectra for 5% damping ratio for the six earthquake records considered in this study.

The earthquake motions applied to bridge structure are expressed as a percentage of the actual record and are classified as moderate or strong according to its peak acceleration. Examples of strong motions are El Centro 200% (0.70g), Loma Prieta 100% (0.63g), Kobe 100% (0.83g), Pacoima Dam 100% (1.075g) and Taft 300% (0.48g). Moderate motions include El Centro 100% (0.35g), Hachinohe 150% (0.34g), Loma Prieta 50% (0.31g) and Taft 100% (0.16g). Peak ground accelerations are given within brackets.
Fig 4.10 Time History of Ground Motion and Response Spectrum for 5% Damping Ratio of El Centro Earthquake
Fig 4.11 Time History of Ground Motion and Response Spectrum for 5% Damping Ratio of San Fernando (Pacoima Dam) Earthquake
Fig 4.12 Time History of Ground Motion and Response Spectrum for 5% Damping Ratio of Taft Earthquake
Fig 4.13 Time History of Ground Motion and Response Spectrum for 5% Damping Ratio of Loma Prieta Earthquake
Fig 4.14 Time History of Ground Motion and Response Spectrum for 5% Damping Ratio of Kobe Earthquake
Fig 4.15 Time History of Ground Motion and Response Spectrum for 5% Damping Ratio of Hachinohe Earthquake
CHAPTER 5

PARAMETRIC STUDY ON SEISMIC RESPONSE OF BRIDGES

5.1 GENERAL

The dynamic response of bridges during earthquakes is a complex phenomena, depending on several factors. Such factors not only include bridge parameters such as, stiffness and yielding properties, damping characteristics, but also to a great extent involve characteristics of the ground motion such as peak acceleration, peak velocity, frequency content and duration of shaking. Possible non-linear behavior of certain elements make it even more complicated phenomena. It is not possible to represent all these variables in a very simple manner. With this in mind, extensive numerical studies were conducted to identify key parameters and to have an assessment of their effects.

Without loosing the essence of the dynamic behavior, simple dynamic models have been used for conducting extensive parametric studies. Bridge vibration in both longitudinal and transverse modes is considered. Both non-isolated and isolated bridges have been studied, however the study concentrates more on isolated bridges. The use of isolation system, as discussed in earlier chapters, is intended to derive certain benefits such as reduction of seismic forces which has been investigated in detail. Possible non-linear behavior of the pier is also extensively considered in this work.

5.2 LONGITUDINAL VIBRATION RESPONSE FOR LINEAR PIER MODEL

Longitudinal vibration of bridges due to earthquake loading, assuming elastic behavior of the pier is studied in the following subsections.

5.2.1 Bridge Model

The study is conducted considering a four span bridge as shown in Fig. 5.1. The bridge is a reinforced concrete box type that is continuous over four spans and is supported by three single column piers. The structural system shown in Fig.5.1 is idealized in longitudinal direction by the analytical model shown in Fig.5.2. A frame consisting of beam-column elements with masses lumped at the nodes represents the bridge structure while non-linear truss elements placed in between the deck and identical piers $P_1, P_2, P_3$ (or identical
Fig 5.1 Typical Four Span Highway Bridge

Fig 5.2 Modeling of the Bridge Structure in Longitudinal direction

Fig 5.3 Single Pier (Stick) Model
abutments A1, A2) represent the isolation units. The truss elements (isolators), allowing relative horizontal movement between the deck and pier is assumed to have bi-linear characteristics (Fig.4.9). The base of the pier element is fixed, i.e., the foundation flexibility is ignored. Some flexibility is assigned to elements representing abutments instead of regarding it as perfectly rigid. This is also done by authors like Shimada et al (1991), Takeda et al. (1991). Fig. 5.3 shows a simple stick model in which a beam-column element with lumped mass ‘m_p’ and length ‘H_p’ represents an isolated typical central pier of the same bridge. The isolator (truss element) on the pier top is connected to a lumped deck mass ‘m_d’ located 0.75 m above pier mass ‘m_p’. Furthermore, rotation is restrained at the pier top which is a valid assumption considering the deck to be rigid in its own place. The simpler model is employed because the comparison or results (Table 5.1) shows good agreement between the two models. This formed a basis for conducting further studies using the much simpler stick model. Unless stated otherwise, the longitudinal vibration studies involved the single pier model.

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Numerical Model</th>
<th>Peak Shear Force (tf)</th>
<th>Peak Isolator Displacement (mm)</th>
<th>Peak Deck Displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>El Centro</td>
<td>Complete Bridge</td>
<td>365</td>
<td>57.9</td>
<td>62</td>
</tr>
<tr>
<td>100%</td>
<td>Single Pier (Stick)</td>
<td>361</td>
<td>57.6</td>
<td>62</td>
</tr>
<tr>
<td>Taft</td>
<td>Complete Bridge</td>
<td>272</td>
<td>30.4</td>
<td>33.1</td>
</tr>
<tr>
<td>100%</td>
<td>Single Pier (Stick)</td>
<td>269</td>
<td>30.3</td>
<td>32.9</td>
</tr>
<tr>
<td>Loma Prieta</td>
<td>Complete Bridge</td>
<td>409</td>
<td>98.7</td>
<td>105.1</td>
</tr>
<tr>
<td>100%</td>
<td>Single Pier (Stick)</td>
<td>413</td>
<td>101.9</td>
<td>107.6</td>
</tr>
<tr>
<td>Kobe</td>
<td>Complete Bridge</td>
<td>951</td>
<td>278.3</td>
<td>290.2</td>
</tr>
<tr>
<td>100%</td>
<td>Single Pier (Stick)</td>
<td>939</td>
<td>274.8</td>
<td>286.6</td>
</tr>
</tbody>
</table>

**Table 5.1 Seismic Bridge Response for Different Numerical Model**

5.2.2 Isolated and Non-isolated Bridge

To study the benefit of isolation system, the same bridge with or without isolators is subjected to ground motion. The non-isolated bridge is obtained by removing the non-linear truss element representing the isolator. The pier is assumed to have an yield force of 20% of
the weight it carries. Results are presented for pier forces within the elastic limit. The same earthquake is applied to both bridges and it is found that different strengths (percentage) of the same earthquake record results in similar seismic forces in the pier. As for example, only 25% of El Centro record generate a force of 431tf in the non-isolated bridge, whereas, 150% of the same record results in similar force in the isolated bridge. Results are also presented for two more earthquakes.

The use of the isolator, on the other hand, increases deck displacement relative to the ground. For El Centro the peak pier top deck displacement is 95.7 mm in isolated bridge compared to 7.1 mm for the non-isolated bridge. The bridge, therefore, needs to be designed to accommodate the increased deck movement.

Table 5.2 Comparison of Isolated and Non-Isolated Bridge in Longitudinal Vibration

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Isolated Bridge</th>
<th>Non Isolated Bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak Pier Force (tf)</td>
<td>Deck Displ. (mm)</td>
</tr>
<tr>
<td>El Centro 150%</td>
<td>431</td>
<td>85.7</td>
</tr>
<tr>
<td>Loma Prieta 100%</td>
<td>435</td>
<td>113.3</td>
</tr>
<tr>
<td>Hachinohe 120%</td>
<td>415</td>
<td>63.3</td>
</tr>
</tbody>
</table>

5.2.3 Parametric Study

An extensive parametric study is conducted to study the role of different bridge and isolator parameters, as well as different ground motions on the response of isolated bridges. Table 5.3 shows the bridge and isolator parameters considered and their default values which are based on published literature. The bi-linear properties used in this study corresponds to those of Lead Rubber bearing and High Damping Rubber Bearings. Other isolation systems are not considered. Unless otherwise specified, the default values are used. The effect of these parameters are investigated on two important design issues: pier shear force and isolator displacement. The peak pier shear force normalized with respect to the weight \((W_d + W_p)\), may be visualized as an equivalent lateral force coefficient for seismic design.
Table 5.3 Isolator and Bridge Parameters and their Range of Values Studied.

<table>
<thead>
<tr>
<th>Member</th>
<th>Parameter</th>
<th>Default Value</th>
<th>Range Studied</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck</td>
<td>Weight, $W_d$</td>
<td>2000 tf</td>
<td>400-6000tf</td>
</tr>
<tr>
<td>Pier</td>
<td>Weight, $W_p$</td>
<td>200 tf</td>
<td>200-600 tf</td>
</tr>
<tr>
<td></td>
<td>$E_p$ (concrete)</td>
<td>2700000 tf/m²</td>
<td>2700000 tf/m²</td>
</tr>
<tr>
<td></td>
<td>Area, $A_p$</td>
<td>7.5 m²</td>
<td>7.5 m²</td>
</tr>
<tr>
<td></td>
<td>Height of Pier, $H_p$</td>
<td>15 m</td>
<td>10-25 m</td>
</tr>
<tr>
<td></td>
<td>Moment of Inertia, $I_p$</td>
<td>7.5 m⁴</td>
<td>2.5-10 m⁴</td>
</tr>
<tr>
<td></td>
<td>Damping</td>
<td>5%</td>
<td>5%</td>
</tr>
<tr>
<td>Isolator</td>
<td>$F_y$/Weight</td>
<td>0.1</td>
<td>0.05-0.2</td>
</tr>
<tr>
<td></td>
<td>Initial Stiffness, $K_i$</td>
<td>18000 tf/m</td>
<td>10800-54000 tf/m</td>
</tr>
<tr>
<td></td>
<td>Post Yielding Stiffness, $K_h$</td>
<td>2700 tf/m</td>
<td>900-4500 tf/m</td>
</tr>
</tbody>
</table>

5.2.3.1 Effect of Mass Ratio

The mass distribution along the deck does not appear to have much significant effect on the pier force, as mentioned earlier in section 5.2.1. Henceforth, the single pier model has been used for further studies. However, the ratio of deck mass ($m_d$) to pier mass ($m_p$), hereafter, called the mass ratio is envisaged to be of prime importance. Figure 5.4 shows the relationship between normalized pier force and pier mass, where the mass ratio is remained constant while both deck mass and pier mass are varied. There is a slight decreasing trend of normalized pier force with the increased mass, however, the effect may be considered small for all earthquakes considered. Thereafter, further study was performed with different values of mass ratio.

The mass ratio ($m_d/m_p$) is varied from 5 to 20 where the deck mass $m_d$ is varied only with the pier mass set at 200 tf. The normalized pier force is significantly larger at small values of mass ratio and decreases as the deck mass is increased. This effect is more pronounced for the stronger earthquakes (Fig.5.5).

The mass ratio does not have any definite pattern of influence on isolator displacement for strong earthquakes (Fig.5.6b). However, in cases of moderate earthquakes, there is a tendency of increased isolator displacement with increased mass ratio (Fig. 5.6a).

Fig 5.7 to 5.11 show the time history of Pier force and Isolator system response for $m_d/m_p$ ratio of 10 for different earthquakes studied.
Fig 5.4 Effect of different values of pier mass when mass ratio is same on normalized pier force for (a) Loma Prieta 100% (b) Taft 100% (c) El Centro 100%
(a) Moderate earthquakes

(b) Strong earthquakes

Fig. 5.5 Effect of mass ratio on normalized pier force
(a) Moderate earthquakes

(b) Strong earthquakes

Fig 5.6 Effect of mass ratio on isolator displacement
Fig 5.7 (a) Time History of Pier Force (b) Time History of Isolator Force and (c) Isolator Force Displacement Curve for Taft 100%
Fig 5.8 (a) Time History of Pier Force (b) Time History of Isolator Force and (c) Isolator Force Displacement Curve for Pacoima Dam 100%
Fig 5.9 (a) Time History of Pier Force (b) Time History of Isolator Force and (c) Isolator Force Displacement Curve for Loma Prieta 100%
Fig 5.10 (a) Time History of Pier Force (b) Time History of Isolator Force and (c) Isolator Force Displacement Curve for Kobe 100%
Fig 5.11 (a) Time History of Pier Force (b) Time History of Isolator Force and (c) Isolator Force Displacement Curve for El Centro 100%
5.2.3.2 Effect of Pier Period

Different combinations of \(W_p\), \(W_d\), \(H_p\) and \(I_p\) have been used to obtain pier periods \((T_p)\) in the range of 0.202 sec. to 1.46 sec. as shown in Table 5.4, where,

\[
T_p = 2\pi \sqrt{\frac{m_d + m_p}{K_{ip}}} 
\]

\(K_{ip}\) represents the stiffness of pier.

**Table 5.4 Combinations of Different Properties for Different Pier Period**

<table>
<thead>
<tr>
<th>(W_d) (Weight of Deck), tf</th>
<th>(W_p) (Weight of Pier), tf</th>
<th>(I_p) (m^4)</th>
<th>(H_t), of Pier (m)</th>
<th>(K_{ip}) (tf/m)</th>
<th>(T_p) (secs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3000</td>
<td>300</td>
<td>10</td>
<td>10</td>
<td>324000</td>
<td>0.202</td>
</tr>
<tr>
<td>2000</td>
<td>200</td>
<td>5</td>
<td>10</td>
<td>162000</td>
<td>0.234</td>
</tr>
<tr>
<td>2000</td>
<td>200</td>
<td>2.5</td>
<td>10</td>
<td>81000</td>
<td>0.331</td>
</tr>
<tr>
<td>3000</td>
<td>350</td>
<td>7.5</td>
<td>15</td>
<td>72000</td>
<td>0.433</td>
</tr>
<tr>
<td>5000</td>
<td>500</td>
<td>10</td>
<td>20</td>
<td>40500</td>
<td>0.739</td>
</tr>
<tr>
<td>4000</td>
<td>400</td>
<td>5</td>
<td>20</td>
<td>20250</td>
<td>0.935</td>
</tr>
<tr>
<td>6000</td>
<td>600</td>
<td>10</td>
<td>25</td>
<td>20736</td>
<td>1.132</td>
</tr>
<tr>
<td>6000</td>
<td>550</td>
<td>7.5</td>
<td>25</td>
<td>15552</td>
<td>1.302</td>
</tr>
<tr>
<td>5000</td>
<td>500</td>
<td>5</td>
<td>25</td>
<td>10368</td>
<td>1.461</td>
</tr>
</tbody>
</table>

The normalized pier forces (Fig. 5.12) for Taft 100%, El Centro 100%, Loma Prieta 100% and Kobe 100% earthquakes are around 0.12, 0.16, 0.2 and 0.5 respectively, which is slightly affected by the pier flexibility. There seems to be a pattern of reduced pier force with increase in pier period. This agrees with the general intuition that increased flexibility in pier would reduce pier forces in the event of an earthquake.

Furthermore, pier period slightly affects the isolator displacement (Fig 5.13). Hachinohe earthquake which is a long period motion appear to have a significant effect on isolator displacement at larger periods.
5.2.3.3 Influence of isolator yield force

The yield force $F_Y$ for each isolator is varied as 0.05, 0.1, 0.15 and 0.2 times the weight on the isolator. For moderate El Centro 100%, Hachinohe 150%, Loma Prieta 50% and Taft 100% the lateral force coefficient shows the trend of increase (El Centro 0.12 to 0.25) with the increase in $F_Y$ (Fig. 5.14). On the other hand, for strong earthquakes like Kobe, the lateral coefficient varies as 0.48, 0.39, 0.41 corresponding to $F_Y$/Weight value of 0.05, 0.15, 0.20 respectively.

The effect of increasing $F_Y$ is two-fold. Smaller values of $F_Y$ implies earlier isolator yielding which is beneficial from flexibility point of view. On the other hand, larger $F_Y$ means larger height of the isolator hysteresis loop leading to more energy dissipation.

The isolator yield force has a considerable effect on isolator displacement for stronger earthquakes like Pacoima and Kobe. In which case isolator displacement decreases with the increase in $F_Y$. For Kobe, the values of isolator displacement are 374, 279 and 246 mm corresponding to $F_Y$/Weight values of 0.05, 0.10 and 0.20 respectively (Fig. 5.15). For moderate earthquakes, change in isolator displacement is less pronounced and isolator yield force vs. isolator displacement curves are relatively flat. Except for long period Hachinohe (150%), where the isolator displacement slightly decreases with increased $F_Y$. 
(a) Moderate Earthquakes

(b) Strong Earthquakes

Fig 5.12 Effect of Pier Period on Normalized Pier Force
Fig 5.13 Effect of Pier Period on Isolator Displacement
**Fig 5.14 Effect of Isolator yield force on Normalized Pier force**
Fig. 5.15 Influence of isolator Yield force on Isolator Displacement
5.2.3.4 Influence of Initial Stiffness of Isolator

Increase in initial isolator stiffness $K_i$ from 10800 tf/m to 54000 tf/m results in significant decrease (up to 50%) in isolator displacement for both moderate and strong earthquake. For 100% of El Centro the decrease is from 72 to 37 mm while in case of strong Kobe earthquake the decrease is from 337 to 209 mm (Fig.5.17).

Initial isolator stiffness doesn't show any definite pattern of influence on normalized pier force. Only in case of long period Hachinohe (150%) and strong Kobe (100%) earthquakes that the normalized pier force is found to decrease with corresponding increase in initial stiffness of isolator.

5.2.3.5 Influence of Post Yielding Stiffness of Isolator

Post yielding stiffness of isolator $K_h$ of each isolator is varied from 900 to 4500 tf/m while the initial stiffness is maintained constant at 18000 tf/m. Increase in $K_h$ has the general trend of resulting increased lateral force coefficient. For moderate earthquakes the increase in $K_h$ causes small and gradual change in lateral force coefficient. But for strong earthquakes, the rate of increase is abrupt. For Kobe, the values changes from 0.19 to 0.7 for changing $K_h$ from 900 to 4500 tf/m. On the other hand, $K_h$ does not appear to have any influence on isolator displacement for moderate earthquakes. But for stronger earthquakes like Kobe and Pacoima isolator displacement shows rapid increase with the increase in $K_h$. For Kobe, isolator displacement varies from 205mm to 521mm when $K_h$ is changed from 900 tf/m to 4500 tf/m, an increase of about 150% (Fig 5.19).
Fig 5.16 Influence of Isolator Initial Stiffness on Normalized Pier Force
Fig 5.17 Influence of Initial Isolator Stiffness on Isolator Displacement
Fig 5.18 Influence of Post Yielding Stiffness of Isolator on Normalized Pier Force
Fig 5.19 Influence of Post Yielding Stiffness of Isolator on Isolator Displacement
5.2.4 Effect of Vertical Motion

The response of bridge to vertical ground motion that is associated with horizontal ground shaking in an earthquake is under study of several researchers. In this study, the isolated pier model is subjected to simultaneous horizontal and vertical motion for Loma Prieta, El Centro, Kobe and Taft earthquakes. The inclusion of vertical motion (Table 5.5) does not have any significant effect on the peak pier shear force. Zayas et al (1987) observed similar trend in experimental studies on base-isolated structures. This may be due to the fact that in each cycle of horizontal shaking, there are many cycles of vertical motions resulting in negligible average vertical motion effect. Inclusion of vertical acceleration, on the other hand, increases pier axial force significantly. The maximum pier axial load due to vertical motion is reported in Table 5.5.

Table 5.5 Effect of vertical motion on peak response

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Horizontal Only</th>
<th>Horizontal + Vertical</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pier Force (tf)</td>
<td>Isolator Pier Force (tf)</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Axial</td>
</tr>
<tr>
<td>Loma Prieta</td>
<td>416.6</td>
<td>2200</td>
</tr>
<tr>
<td>100%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>El Centro</td>
<td>361.5</td>
<td>2200</td>
</tr>
<tr>
<td>100%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kobe 100%</td>
<td>939.2</td>
<td>2200</td>
</tr>
<tr>
<td>Taft 100%</td>
<td>269.5</td>
<td>2200</td>
</tr>
</tbody>
</table>

5.3 LONGITUDINAL VIBRATION RESPONSE FOR NONLINEAR PIER MODEL

In moderate earthquakes conventional non-isolated bridges are likely to sustain some yielding in the piers. For strong earthquakes yielding may be to an extent of causing major damage. In the case of isolated bridges, for moderate earthquakes, the chance of yielding is minimal. For strong earthquakes, even for isolated bridges, some yielding may take place. It is therefore, considered important to study the possibility of yielding in the bridge pier. A simple non linear spring element is used to represent the non-linear behavior of the pier (Fig. 5.20). Where $K_p$ and $K_{pi}$ represent the initial and post yielding stiffness of pier.
The strength of pier is represented by \( \varepsilon \) and is given by the ratio:

\[
\varepsilon = \frac{F_{yp}}{W_d + W_p}
\]  

(5.2)

Where, \( W_p + W_d \) represents the total weight acting on pier and \( F_{yp} \) represents the pier yield force. The amount of yielding is simplistically indicated by the displacement ductility demand, \( \mu \), where,

\[
\mu = \frac{\Delta_u}{\Delta_y}
\]  

(5.3)

Where, \( \Delta_u = \) Peak displacement

\( \Delta_y = \) Yield displacement

*Fig. 5.20 Nonlinear Pier Model*
5.3.1 Influence of Nonlinear Behavior of Pier on Peak Response

Table 5.6 shows the maximum normalized pier force for the non-linear and linear pier models in the case of isolated bridge. From the aforementioned results, it is apparent that the lateral load coefficient during a strong earthquake like Kobe and Pacoima is reduced due to the effect exerted by the nonlinearity of the columns. The degree of lateral load reduction is approximately 40% for strong Kobe earthquake. Furthermore, the lateral load coefficient show little or no change for moderate earthquakes like Taft or El Centro. The reason is that the force at the pier remains within elastic limit for weaker seismic motions.

**Table 5.6 Comparison of Peak response for linear and non-linear column**

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Normalized Pier Force</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Linear Column</td>
</tr>
<tr>
<td>Taft 100%</td>
<td>0.12</td>
</tr>
<tr>
<td>El Centro 100%</td>
<td>0.15</td>
</tr>
<tr>
<td>Loma Prieta 100%</td>
<td>0.20</td>
</tr>
<tr>
<td>Pacioma Dam 100%</td>
<td>0.33</td>
</tr>
<tr>
<td>Kobe 100%</td>
<td>0.42</td>
</tr>
</tbody>
</table>
5.3.2 Effect of Isolation on Ductility Demand

The effectiveness of incorporating isolation system on pier ductility demand is demonstrated by comparing responses of isolated bridge to that of comparable non-isolated bridge. The ratio of pier strength to load carried by the pier $\varepsilon$ has assigned values of 0.1, 0.2 and 0.3. The case $\varepsilon = 0.1$ corresponds to a pier of lower yield strength. The results obtained in this case are of interest in the seismic retrofit of bridges with piers of lower yield strength.

The results presented in Table 5.7 demonstrate a marked reduction in the displacement ductility demand of the pier of isolated bridge. In case of the severely under-designed pier ($\varepsilon = 0.1$) the ductility demand is excessive and clearly indicates the possibility of collapse or severe damage. In contrast, the pier of the isolated bridge experiences limited ductility demand and nearly elastic behavior. Furthermore, the isolated bridge with pier strength of $\varepsilon > 0.2$ remains elastic except for the very strong Kobe earthquake where some damage of the pier is unavoidable.

Table 5.7 Comparison of ductility demand for isolated and non-isolated bridges.

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>$\varepsilon$</th>
<th>Isolated Bridge</th>
<th>Non-Isolated Bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Normalized Pier Force</td>
<td>Ductility Demand, $\mu$</td>
</tr>
<tr>
<td>El Centro</td>
<td>0.1</td>
<td>0.134</td>
<td>7.87</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>0.165</td>
<td>0.82</td>
</tr>
<tr>
<td></td>
<td>0.3</td>
<td>0.165</td>
<td>0.55</td>
</tr>
<tr>
<td>Taft</td>
<td>0.1</td>
<td>0.104</td>
<td>1.96</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>0.122</td>
<td>0.61</td>
</tr>
<tr>
<td></td>
<td>0.3</td>
<td>0.122</td>
<td>0.40</td>
</tr>
<tr>
<td>Loma Prieta</td>
<td>0.1</td>
<td>0.154</td>
<td>11.97</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>0.198</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td>0.3</td>
<td>0.198</td>
<td>0.66</td>
</tr>
<tr>
<td>Kobe</td>
<td>0.1</td>
<td>0.235</td>
<td>28.16</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>0.267</td>
<td>7.78</td>
</tr>
<tr>
<td></td>
<td>0.3</td>
<td>0.365</td>
<td>5.38</td>
</tr>
</tbody>
</table>
5.3.3 Parametric Study on Nonlinear Behavior of Pier

The influence of parameters like mass ratio, pier period, pier strength, initial isolator stiffness, isolator yield force and post yielding stiffness of isolator on ductility demand, normalized pier force and isolator displacement considering non-linear behavior of column is shown in from figures 5.22 to 5.27. The default values and the range of parameter studied shown in Table 5.3 is also applicable in this case with the addition that the strength of pier is taken as 20% of the weight on it as default. The ratio of post yielding stiffness and initial stiffness of pier has a set value of 0.05.

The increase in pier strength inevitably decreases the ductility demand which is shown in figure 5.22(a). Figure 5.22(b) shows the relationship between lateral force coefficient and pier strength. The pier force increases with the increase in pier strength. For Pacoima Dam earthquake increase in normalized pier force is from 0.22 to 0.37 when the pier strength is increased from 15% of the weight it carries to 30% of its weight. Furthermore, pier strength has no influence on isolator displacement for moderate earthquakes. But for strong earthquakes like Kobe, isolator displacement steadily increases from 170mm to 265 mm when the pier strength is increased from 15% to 30% of the weight it carries. The trend is, again, somewhat similar for strong Pacoima earthquake.

Fig 5.23(a) shows the relationship between mass ratio and ductility demand of piers. Ductility demand shows rapid decrease with the increase in mass ratio. For Pacoima Dam earthquake ductility demand decreases from 11 to a value of 2.4 when mass ratio is increased from 5 to 20. Mass ratio has similar decreasing effect on normalized pier force which is shown in Fig.5.20(b). The decrease is upto 50% for strong Kobe earthquake.

Figure 5.24(a) shows the relationship between the ductility demand and pier period. Ductility demand shows rapid decrease with the increase in pier period. For Kobe the decrease in ductility demand is from 9 to 1.2 when the pier period is increased from 0.202 to 1.46. The reason behind that is that increase in the pier period results in more flexible pier which is more efficient in absorbing energy per cycle of loading. For the same reason, normalized pier force decreases with the increase in pier period (Fig.5.24b). Unlike ductility demand and normalized pier force, pier period does not show any definitive influence on isolator displacement (Fig.5.24c) which remains more or less same for all earthquakes studied.
except for El Centro, where isolator displacement increases from 192mm to 315mm when pier period is increased from 0.202 to 1.46 sec.

Increase in isolator yield force cause gradual increase in ductility demand for moderate earthquakes like Loma Prieta. On the other hand, for stronger earthquakes like Kobe, ductility demand varies as 13, 7.7, 8.4 and 11.8 corresponding to $F_y$ values of 0.05, 0.1, 0.15 and 0.2 times the weight on isolator (Fig. 5.25a). This implies that ductility demand is higher when $F_y$ is lower that 10% of the weight on it and again rises when $F_y$ is more than 15% of the weight. A similar trend is noticed for strong Pacoima earthquake. This gives the impression that optimum isolator yield force for strong earthquakes should be set between 10 to 15 percent of the weight on it. Furthermore, normalized pier force increases with the increase in isolator yield force for moderate earthquakes. But for strong earthquakes, like Kobe, normalized pier force varies as 0.34, 0.27, 0.28 and 0.31 for $F_y$ values of 0.05, 0.1, 0.15 and 0.2 times the weight on isolator (Fig. 5.25b).

Initial stiffness of isolator does not show any significant influence on ductility demand, normalized pier force or isolator displacement (Fig. 5.26) except for the case of strong Kobe earthquake in which case ductility demand and normalized pier force show gradual decrease.

Increase in the value of post yielding stiffness $K_h$, shows gradual increase in both ductility demand and lateral force coefficient. The rate on change is increased with the increase in earthquake magnitude. On the other hand, post yielding stiffness does not influence isolator displacement as shown by the relatively flat isolator displacement vs. post yielding stiffness curves (Fig. 5.27c)
Fig. 5.22 Influence of Pier Strength on (a) Ductility Demand (b) Normalized Pier Force (c) Isolator Displacement
Fig. 5.23 Influence of Mass Ratio on (a) Ductility Demand (b) Normalized Pier Force (c) Isolator Displacement
Fig. 5.24 Influence of Pier Period on (a) Ductility Demand (b) Normalized Pier Force (c) Isolator Displacement
Fig. 5.25 Influence of Isolator Yield Force on (a) Ductility Demand (b) Normalized Pier Force (c) Isolator Displacement
Fig. 5.26 Influence of Initial Stiffness of Isolator on (a) Ductility Demand (b) Normalized Pier Force (c) Isolator Displacement
Fig. 5.27 Influence of Post Yielding Stiffness of Isolator on (a) Ductility Demand (b) Normalized Pier Force (c) Isolator Displacement
5.4 TRANSVERSE VIBRATION RESPONSE FOR LINEAR PIER MODEL

Transverse vibration is different from longitudinal vibration in certain aspects. First, the deck in plane stiffness that provided rotational restraint at pier location in the case of longitudinal vibration is not present in transverse vibration. Second, the pier stiffness is usually much higher in the transverse direction. Third, the deck mass is subjected to vibration in the transverse direction spanning across consecutive piers. Such vibration of deck mass may induce additional inertia forces. Fourth, displacement restraint in the transverse direction may be provided at the bridge abutments which may have some effect for relatively short span bridges. In this study, the first three points are considered.

Fig. 5.28 shows the model of the bridge described in section 5.2.1 in transverse direction.

Fig 5.28 Transverse model of bridge shown in Fig 5.1
5.4.1 Isolated and Non-Isolated Bridge

As in the case of longitudinal vibration, the same bridge with or without isolator is subjected to ground motion in transverse direction. Again, the pier is assumed to have a yield force of 20% of the weight it carries and results are presented for pier force within elastic limit. The same earthquake is applied to both bridges and it was found that different strengths (percentage) of same earthquake record results in similar seismic forces (Table 5.8) in piers. The deck displacement, however, increases for the case of isolated bridges on both pier top and mid-span.

As for example, only 25% of El Centro record generates a force of 456tf in the non-isolated bridge, whereas 150% of the same record results in force of similar magnitude in the isolated bridge. Deck displacement, however, increases from 6.3 mm in case of non-isolated bridge to 108.5 mm for the case of when the bridge is isolated.

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Pier Force (tf)</th>
<th>Isolator Displ. (mm)</th>
<th>Deck Displ. (mm) w.r.t ground</th>
<th>Earthquake</th>
<th>Pier Force (tf)</th>
<th>Deck Displ. (mm) w.r.t ground</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pier Top</td>
<td></td>
<td></td>
<td>Pier Top</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Midspan</td>
<td></td>
<td></td>
<td>Midspan</td>
</tr>
<tr>
<td>El Centro</td>
<td>456</td>
<td>102.2</td>
<td>105.8</td>
<td>460.68</td>
<td>6.3</td>
<td>13.4</td>
</tr>
<tr>
<td>150%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loma Prieta</td>
<td>426</td>
<td>108.8</td>
<td>113</td>
<td>432</td>
<td>6.3</td>
<td>12.8</td>
</tr>
<tr>
<td>100%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pacoima Dam</td>
<td>365</td>
<td>92.7</td>
<td>101.3</td>
<td>349</td>
<td>4.7</td>
<td>9.5</td>
</tr>
<tr>
<td>40%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 5.8 Comparison of Isolated and Non Isolated Bridges in Transverse Vibration
5.4.2 Parametric Study

Unless otherwise specified, the following parameters (based on published literature) are used for analysis of the complete bridge in transverse direction:

Table 5.9 Range of values for parameters in Transverse Direction

<table>
<thead>
<tr>
<th>Member</th>
<th>Parameter</th>
<th>Default Value</th>
<th>Range Studied</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck</td>
<td>Weight, $W_d$</td>
<td>2000 tf</td>
<td>400-7000 tf</td>
</tr>
<tr>
<td></td>
<td>Area, $A_d$</td>
<td>12.5 m$^2$</td>
<td>5-15 m$^2$</td>
</tr>
<tr>
<td></td>
<td>Inertia, $I_d$ (Longitudinal)</td>
<td>20 m$^4$</td>
<td>2-50 m$^4$</td>
</tr>
<tr>
<td></td>
<td>$I_d$ (Transverse)</td>
<td>100 m$^4$</td>
<td>10-200 m$^4$</td>
</tr>
<tr>
<td></td>
<td>Damping</td>
<td>5%</td>
<td>5%</td>
</tr>
<tr>
<td></td>
<td>$E_d$</td>
<td>2700000 tfl m$^2$</td>
<td>2700000 tfl m$^2$</td>
</tr>
<tr>
<td>Pier</td>
<td>Weight, $W_p$</td>
<td>200 tf</td>
<td>200-600 tf</td>
</tr>
<tr>
<td></td>
<td>Area, $A_p$</td>
<td>7.5 m$^2$</td>
<td>5-15 m$^2$</td>
</tr>
<tr>
<td></td>
<td>Inertia, $I_p$</td>
<td>40 m$^4$</td>
<td>10-50 m$^4$</td>
</tr>
<tr>
<td></td>
<td>Damping</td>
<td>5%</td>
<td>5%</td>
</tr>
<tr>
<td></td>
<td>$E_p$</td>
<td>2700000 tfl m$^2$</td>
<td>2700000 tfl m$^2$</td>
</tr>
<tr>
<td></td>
<td>Damping</td>
<td>5%</td>
<td>5%</td>
</tr>
<tr>
<td>Isolator</td>
<td>Fyi / Weight</td>
<td>0.1</td>
<td>0.05-0.2</td>
</tr>
<tr>
<td></td>
<td>$K_l$</td>
<td>18000 tfl/m</td>
<td>10800-54000 tfl/m</td>
</tr>
<tr>
<td></td>
<td>$K_h$</td>
<td>2700 tfl/m</td>
<td>900-4500 tfl/m</td>
</tr>
</tbody>
</table>

5.4.2.1 Effect of Mass Ratio

The mass ratio ($m_d/m_p$) is varied from 5 to 20 where the deck mass is varied only. As in the case of longitudinal vibration, the lateral force coefficient shows gradual decrease with the increase in mass ratio, which is the case for both moderate and strong earthquakes. But, mass ratio does not show any significant influence on deck displacement for moderate earthquakes. But for the case of strong earthquake like Kobe, increasing mass ratio results in increased deck displacement. Fig. 5.29(a) shows that, for Kobe, deck displacement increases from 273mm to 389mm when mass ratio is increased from 5 to 20; an increase of about 45%.
5.4.2.2 Effect of Pier Period

Pier period in transverse direction is varied from 0.18 sec to 1.307 sec by varying the properties that influence pier period i.e. \( W_d, W_p \) and \( I_p \), as shown in Table 5.10.

Table 5.10 Combinations of Different Properties for Different Pier Period in Transverse Direction

<table>
<thead>
<tr>
<th>( W_d ) (Weight of Deck), tf</th>
<th>( W_p ) (Weight of Pier), tf</th>
<th>( I_p ) (m(^4)) (Transverse)</th>
<th>( H_t ) of Pier (m)</th>
<th>( K_{ip} ) (tf/m)</th>
<th>( T_p ) (secs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3000</td>
<td>300</td>
<td>50</td>
<td>10</td>
<td>405000</td>
<td>0.181</td>
</tr>
<tr>
<td>2000</td>
<td>200</td>
<td>25</td>
<td>10</td>
<td>202500</td>
<td>0.209</td>
</tr>
<tr>
<td>2000</td>
<td>200</td>
<td>12.5</td>
<td>10</td>
<td>101250</td>
<td>0.296</td>
</tr>
<tr>
<td>3000</td>
<td>350</td>
<td>37.5</td>
<td>15</td>
<td>90000</td>
<td>0.387</td>
</tr>
<tr>
<td>5000</td>
<td>500</td>
<td>50</td>
<td>20</td>
<td>50625</td>
<td>0.661</td>
</tr>
<tr>
<td>4000</td>
<td>400</td>
<td>25</td>
<td>20</td>
<td>25312</td>
<td>0.836</td>
</tr>
<tr>
<td>6000</td>
<td>600</td>
<td>50</td>
<td>25</td>
<td>25920</td>
<td>1.012</td>
</tr>
<tr>
<td>6000</td>
<td>550</td>
<td>37.5</td>
<td>25</td>
<td>19440</td>
<td>1.164</td>
</tr>
<tr>
<td>5000</td>
<td>500</td>
<td>25</td>
<td>25</td>
<td>12960</td>
<td>1.307</td>
</tr>
</tbody>
</table>

As in the case of longitudinal vibration the normalized pier force is found to decrease slightly with the corresponding increase in pier period. Furthermore, variation of mid-span deck displacement seems to be independent of pier period for moderate earthquakes. But for strong Kobe earthquake the mid-span deck displacement shows about 30% increase when pier period is raised from 0.18 sec to 1.307 sec.

5.4.2.3 Effect of Isolator Yield Force

Increasing isolator yield force from 0.05 to 0.2 times the weight on it results in, as in the case of longitudinal vibration, a steady increase in lateral force coefficient for moderate earthquakes like Loma Prieta and El Centro. For strong earthquakes like Kobe the lateral force coefficient varies as 0.50, 0.43, 0.38 and 0.41 corresponding to \( F_{yi} \) Weight value of 0.05, 0.10, 0.15 and 0.20 respectively. Change in isolator yield force does not show any significant influence on mid-span deck displacement for moderate earthquakes. But for
strong earthquakes the displacement gradually decreases with the increase in isolator yield force. For the case of Kobe earthquake the decrease is about 40% when isolator $F_y/\text{Weight}$ value is increased from 0.05 to 0.20.

5.4.2.4 Influence of Initial Stiffness of Isolator

Figure 5.32 (a) and (b) shows the effect of initial isolator stiffness on pier force and deck displacement. For moderate earthquakes like El Centro and Loma Prieta, change in initial isolator stiffness does not appear to influence the lateral force coefficient and transverse deck displacement. But for strong earthquakes larger isolator initial stiffness results in smaller lateral force coefficient and lesser deck displacement.

5.4.2.5 Influence of Post Yielding Stiffness of Isolator

Post yielding stiffness of isolator, $K_h$ is varied from 900tf/m to 4500 tf/m, while initial stiffness is maintained constant at 6000tf/m. The effect of $K_h$ on pier force and mid-span deck displacement is relatively small for moderate earthquakes except in case of long period motion of Hachinohe that the deck displacement shows about 100% increase. But for strong earthquakes like Kobe the lateral force coefficient increases abruptly from 0.20 to 0.84 and mid-span deck displacement increases from 199 mm to 401 mm corresponding to $K_h$ value of 900tf/m and 4500 tf/m respectively (Fig. 5.33).
Fig 5.29 Influence of Mass Ratio on Mid-Span Deck Displacement and Transverse Pier Force
Fig 5.30 Influence of Pier Period on Mid-Span Deck Displacement and Transverse Pier Force
Fig 5.31 Influence of Isolator Yield Force on Mid-Span Deck Displacement and Transverse Pier Force
Fig 5.32 Influence of Initial Isolator Stiffness on Mid-Span Deck Displacement and Transverse Pier Force
Fig 5.33 Influence of Post Yielding Stiffness of Isolator on Mid-Span Deck Displacement and Transverse Pier Force
CHAPTER 6
SEISMIC RESPONSE OF JAMUNA MULTIPURPOSE BRIDGE

6.1 GENERAL

The recently commissioned 4.8 km long Jamuna Multipurpose Bridge (which has been named Bangabandhu Bridge) over the mighty Jamuna river has established the long-cherished road link between the East and West of Bangladesh. This bridge connects Bhuapur in Tangail (East Bank) with Sirajganj in Pabna (West Bank). The bridge site location map is shown in Fig. 6.1. The bridge is expected to promote better inter-regional trade, economic and social development and play a very significant role in the socio-economic development of the country. The total project cost of US $696 million was jointly financed by the Government of Bangladesh, Government of Japan (OECF), World Bank (IDA) and Asian Development Bank. The cost of the Bridge structure including approach viaducts is about US $247.4 million, whereas, the major cost component of the project belongs to river training works and reclamation works comprising US $276 million. The bridge is located in a seismically active region and has been designed to resist dynamic forces due to earthquakes with peak ground acceleration as high as 0.2g. JMB is the first bridge in this country where seismic pintles have been used. The pintles act as an isolation device for protection against earthquakes. The objectives of the present study are to establish a simplified model of the bridge that can be effectively used for dynamic analysis, evaluate the effectiveness of the pintles in reducing pier forces in the event of an earthquake and to examine the effect of scour depth on pier response.
Fig. 6.1 Bangabandhu (Jamuna Multipurpose) Bridge Site Location Map
6.2 BRIEF DESCRIPTION OF JMB

The Jamuna Multipurpose Bridge (JMB) consists of four lane road, a single track meter gauge railway, a footpath, a 230 KV electric power line, a high pressure natural gas pipeline and telecommunication cables. As shown in Fig.6.1 the bridge is slightly curved in plan and spans over water as well as large sand islands (chars) in the middle of the river. The braided nature of the river results in shifting of the channels as well as the 'chars'. The main bridge is about 4.8 km long, prestressed concrete box-girder type, and consists of 47 nearly equal spans of 99.375m plus 2 smaller end spans of 64.6875m. The main bridge is supported by twenty-one 3-pile piers and twenty-nine 2-pile piers. There is a 128m long road approach viaduct at each end of the main bridge. There are six hinges (expansion joints) that separate the main bridge structure into seven modules (two end modules, four 7-span module and a 6-span module in the middle).

<table>
<thead>
<tr>
<th>Table 6.1 Salient features of the bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Length of bridge</strong></td>
</tr>
<tr>
<td><strong>Length of viaducts of each side</strong></td>
</tr>
<tr>
<td><strong>Width of bridge</strong></td>
</tr>
<tr>
<td><strong>Number of spans</strong></td>
</tr>
<tr>
<td><strong>Length of each span</strong></td>
</tr>
<tr>
<td><strong>Length of end span</strong></td>
</tr>
<tr>
<td><strong>Number of lanes</strong></td>
</tr>
<tr>
<td><strong>3 Pile Pier (2500mm OD)</strong></td>
</tr>
<tr>
<td><strong>2 Pile Pier (3150mm OD)</strong></td>
</tr>
<tr>
<td><strong>Number of Total Piers</strong></td>
</tr>
<tr>
<td><strong>Number of Total Piles</strong></td>
</tr>
<tr>
<td><strong>Tubular steel Pile Thickness</strong></td>
</tr>
<tr>
<td><strong>Average Length of Pile</strong></td>
</tr>
<tr>
<td><strong>Box girder segment length</strong></td>
</tr>
<tr>
<td><strong>Absolute rake of Pile (Batter Pile)</strong></td>
</tr>
<tr>
<td><strong>Pier Stem height range</strong></td>
</tr>
</tbody>
</table>

For seismic protection of JMB, isolation system consisting of pin dissipating elements and shock transmitter units have been placed in between girders and the piers.
From seismic response viewpoint four elements of the bridge viz. Deck, Pier, Pile Group and Isolation unit are important and therefore further descriptions are provided regarding their configurations and properties.

6.2.1 Pile Configuration

The substructure of each module consists of three 3-pile piers and three or four 2-pile piers for the six and seven span modules respectively. The foundations consist of driven tubular steel piles, filled with concrete. Pile diameters are 3.15m for the 2-pile piers and 2.50m for the 3-pile piers, and toe levels vary from -70.0m PWD (Public Works Datum) to -82.0m PWD, with a head level of +11m PWD. The thickness of the steel tube varies along the length of the pile. For 2-pile system the thickness varies as 60mm (from pile cut-off upto -6.0m PWD), 55mm (from -6.0m PWD to -26.0m PWD) and 50mm (from -26.0 PWD upto pile toe). And for 3-pile system the thickness varies as 50mm (from pile cut-off upto -6.0m PWD), 45mm (from -6.0m PWD upto -26.0m PWD) and 40mm (from -26.0 PWD upto pile toe). Pile caps are of precast reinforced concrete shell with in-situ reinforced concrete infill construction. They have a base level of +11.0m PWD, and so the piles are embedded some 7m within the caps. The pile caps carry pier stems which in turn support the bearings. Fig. 6.2 shows the general arrangement of piles.

6.2.2 Pier Stem

The height of pier stem varies from 2.72m to 12.05m and is constructed of reinforced concrete. Figs. 6.3 and 6.4 show the cross-section and elevation of the pier stem respectively. The hollow section of pier stem is filled with concrete up to 3m of its height. The cross-sectional properties of hollow and solid sections are given in Table 6.2.

<table>
<thead>
<tr>
<th>Table 6.2 Cross sectional Properties of Pier Stem</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Section Type</strong></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Hollow</td>
</tr>
<tr>
<td>Solid</td>
</tr>
</tbody>
</table>

Fig. 6.5 shows the P-M diagram of the pier stem in transverse direction (T.Y. Lin International, 1994).
Fig 6.3 Cross Section of Pier Stem
Fig. 6.4 Elevation of Pier Stem
Fig. 6.5 P-M Diagram of Pier Stem
6.2.3 Deck Configuration

The deck is of prestressed post-tensioned concrete segmental construction, with a varying depth single box section (Fig. 6.6). Spans cantilevering out from the piers are joined by an in-situ closure at mid-span. The width of the box-section is the same for all sections which is 18.5m but the depth varies from 6.5m at the pier top to 3.25m at midspan. Accordingly, area and moment of inertia both in longitudinal and transverse direction vary along the span.

6.2.4 Isolation System

Pin dissipating device (pintles) has been used as base-isolator for protection against earthquakes in JMB. Two types of dissipating device are used:

- pin elements dissipating device for fixed location (at 3-pile piers)
- pin elements and shock transmitter for the mobile locations (at 2-pile piers)

Fig. 6.7 shows the schematic diagram of the device which comprises of the following elements: (FIP Industriale, 1995)

- A central body with pin dissipating elements (1)
- An upper (2) and a lower (3) plate, between which the pins are affixed. The top plate has an annular plate (4) to transmit the horizontal seismic loads from the superstructure to the pier.
- An external frame (5) with a tapered internal ring (6) affixed to the superstructure by means of bolts (9)
- An upper anchoring frame (7) with sockets (10)
- A ring (8) with an outer surface that is also tapered to match the taper of the element's inner surface.

The dissipating device for the mobile locations comprises the same components as described for the fixed devices plus a shock transmitter unit made of:

- A one piece hydraulic cylinder
Fig. 6.6 Typical Deck Cross-sections (Dimensions in mm)
Two bolted flanges that close the ends of the cylinder, and

A double headed piston rod that creates two chamber within the cylinder.

The basic operation of the isolation device can be summarized as:

1. All horizontal loads other than those of sudden onset are accommodated by the elastic deformation of the pin elements in the fixed type devices at 3-pile piers.

2. Slowly applied movements, such as those resulting from thermal displacement of the bridge deck, are accommodated by the shock transmitter in the mobile type devices at 2-pile piers.

3. Sudden onset loads, such as earthquake are accommodated by the pin elements in both the fixed and mobile devices. The sharing of the sudden onset loads is achieved because the shock transmitter in the mobile locations locks up and transmits the loads to the pin elements.

6.3 NUMERICAL MODELING

The 7-span module 2 (second module from the west end) was selected in this study for dynamic analysis because of its close proximity with the Bogra fault (25-50km), one of the four major faults with earthquake potentials that lies within Bangladesh and neighboring India and Burma. Fig. 6.8 shows plan of the complete bridge and elevation of 7-span module whereas Fig. 6.9 shows the foundation plan of the selected (module 2) module. The module has seven piers (pier no P8 through P14) with four 2-pile piers (P8, P9, P13 and P14) and three 3-pile piers (P10, P11 and P12). In numerical modeling of the complete module, the following simplifying assumptions have been made:

- The curvature of the bridge in the longitudinal direction is ignored.
- Masses are lumped at nodes.
- The concrete infill within the steel tube piles is neglected in computing foundation stiffness.
- The pile system is replaced by a single equivalent beam-column element.
6.3.1 Design Earthquake

Design earthquake for dynamic analysis of JMB is obtained from the Taft Lincoln School Tunnel record of July, 1951 at the Kern County, California. The 140% of the N21E component with the changed time interval of ground motion records from 0.02 sec. to 0.045 sec. is used as design earthquake. The response spectra for 5% damping ratio for this earthquake record shows close matching with the design response spectra of JMB proposed by Professor Bolt (Bolt, 1987). Fig. 6.10 shows the acceleration time-history and response spectra for this record together with the design response spectra of JMB.

6.3.2 Foundation Stiffness

The foundation of JMB is composed of either two or three pile systems. In numerical modeling of the bridge, these pile systems are represented by beam-column elements having equivalent stiffness.

The effect of variable scour depth is taken into consideration in determining foundation stiffness. For this, three different depth of pile fixity are considered. The equivalent stiffness of a pile group is obtained by applying unit load at the top of the pile group and measuring the corresponding deflection (Fig. 6.11) using the program ANSR. Table 6.3 shows the foundation stiffness for two and three pile systems with different depths of pile fixity.
**Fig. 6.10** Acceleration time history of the design ground motion for JMB and design response spectra for 5% damping ratio
Fig. 6.11 Computation of Foundation Stiffness for (a) 2-pile system transverse direction (b) 2-pile system longitudinal direction (c) 3-pile system longitudinal direction (d) 3-pile system transverse direction

Table 6.3 Foundation Stiffness at different depth of fixity

<table>
<thead>
<tr>
<th>Depth of Fixity</th>
<th>Longitudinal Stiffness (tf/m)</th>
<th>Transverse Stiffness (tf/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>2-pile System</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-24m PWD</td>
<td>3438</td>
<td>21866</td>
</tr>
<tr>
<td>-34m PWD</td>
<td>1058</td>
<td>13241</td>
</tr>
<tr>
<td>-44m PWD</td>
<td>453</td>
<td>9394</td>
</tr>
<tr>
<td><strong>3-pile System</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-24m PWD</td>
<td>16514</td>
<td>16517</td>
</tr>
<tr>
<td>-34m PWD</td>
<td>10211</td>
<td>10218</td>
</tr>
<tr>
<td>-44m PWD</td>
<td>7402</td>
<td>7234</td>
</tr>
</tbody>
</table>
6.3.3 Modeling of Isolation Device

Isolation unit used in JMB consists of 42 pins. Fig 6.12 shows the load-displacement curve referred to 4 pins that were tested for determining the mechanical properties of the system (FIP Industriale, 1996). The area under the closed curve is the energy dissipated by 4 pins during elasto-plastic cycle. The results obtained from 4 pins is extended to complete seismic device containing 42 pins by multiplying the values of force obtained from the graph by 10.5 (i.e. 42/4). These results are reported in Table 6.4.

Table 6.4 Force displacement relationship of pin device

<table>
<thead>
<tr>
<th>Pins</th>
<th>Force elastic (KN)</th>
<th>Elastic displacement (mm)</th>
<th>Force plastic (KN)</th>
<th>Plastic displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>317.5 KN</td>
<td>27</td>
<td>397.5</td>
<td>200</td>
</tr>
<tr>
<td>42</td>
<td>3333.75</td>
<td>27</td>
<td>4173.75</td>
<td>200</td>
</tr>
</tbody>
</table>

The results from Table 6.4 and Fig. 6.12 are used to establish the conclusive load deflection diagram (Fig. 6.13) of the pin dissipating device which is used for numerical modeling of the isolation unit.

Fig. 6.13 Load-Deflection Diagram of the Complete pin Device
From Fig. 6.13 the key properties of isolator are:

- Isolator Yield Force, $F_y = 339.83$ tf
- Initial Slope of Isolator, $K_i = 12586$ tf/m
- Post-Yielding Slope of Isolator, $K_h = 494.91$ tf/m

6.3.4 Modeling of the Bridge Structure

The structural system of the 7-span module shown in Fig. 6.8 is idealized in longitudinal and transverse direction by the analytical models shown in Fig. 6.14. A frame consisting of beam column elements with masses lumped at nodes represents the complete module. The non-linear truss elements placed between the deck and piers P8, P9, P10, P11, P12, P13 and P14 represent the isolation units. The truss element has the bi-linear force-displacement relationship as shown in Fig. 6.13. The pile system is represented by a single beam-column elements of equivalent stiffness. However, stiffness of this element is varied in analysis to take the effect of variable scour depth as described in section 6.3.2. Pier stem is represented by two elements. The bottom element, 3m in height, represents the solid bottom section of the pier. While the element on top of it represents the hollow portion of the pier stem. The height of the element representing the hollow portion is, however, varied according to the height of the pier.

Total mass of each element representing the pier is distributed equally between two adjacent nodes. The lumped mass at the bottom of all piers also contains the heavy mass (280 tf) representing the pile cap. The deck mass of each span is lumped at three nodes: one at mid-span and one each at two adjacent pier tops. In distributing the deck mass among these nodes, the effect of variable cross section of the deck is taken into consideration.

Fig. 6.15 shows a single pier stick model of the module. The elements and masses representing the pier is same as for any pier in the model of the complete module. However, the deck mass in this case is the total mass of deck in a span. To examine the single pier model as a basis for analysis both the complete module (Fig. 6.14) and a selected single pier model (P9 and P13) are subjected to the same earthquake. The result of analysis are reported in Table 6.5. The results show close agreement between two models. Therefore, a part of the analysis that follows was conducted on single pier model.
Table 6.5 Bridge Response for Different Numerical Models

<table>
<thead>
<tr>
<th>Pier</th>
<th>Single Pier Model</th>
<th>Complete Module</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Normalized Pier Force</td>
<td>Normalized Pier Force</td>
</tr>
<tr>
<td></td>
<td>Isolator Displacement (mm)</td>
<td>Isolator Displacement (mm)</td>
</tr>
<tr>
<td>P13 (3-pile pier), Long. dir.</td>
<td>0.171</td>
<td>141</td>
</tr>
<tr>
<td>P9 (2-pile pier), Trans. dir.</td>
<td>0.179</td>
<td>198</td>
</tr>
</tbody>
</table>

6.4 RESULTS OF ANALYSIS AND DISCUSSION

The numerical model for 7-span module 2 obtained in section 6.3.4 is subjected to design earthquake discussed in section 6.3.2. The results of this analysis and discussions on this results are presented in the subsequent sections.

6.4.1 Transverse Response

The subsequent section deals with transverse vibration of JMB. The bridge response is envisaged to be influenced by the depth of scouring. Therefore, three different level of pile fixity is considered, viz. -44m PWD, -34m PWD and -24m PWD. The default value of pile fixity is taken as to be -34m PWD.

6.4.1.1 Effectiveness of Seismic Pintles (Isolator)

To study the benefit of seismic pintles that has been used as isolation device in Bangabandhu (Jamuna Multipurpose) Bridge, the same pier with or without the pintles is subjected to ground motion. The non-isolated pier is obtained by removing the nonlinear truss element representing the pin device. The same design earthquake is applied to both isolated and non-isolated pier and it is found that different strengths (percentage) of the same earthquake results in similar seismic forces in the pier. For example, the lateral force coefficient in the isolated 2-pile pier P8 for 100% of the design earthquake is 0.181, while the same in the non isolated pier for 32% design earthquake is 0.179. This means that the isolated pier can withstand earthquake shaking about 3 times stronger than the non-isolated pier. The use of pintles has, however, resulted in increased deck displacement (Table 6.6).
Fig 6.14 Bangabandhu (Jamuna) Bridge Modeling (a) longitudinal direction (b) transverse direction

Fig 6.15 Single Pier Stick Model
### Table 6.6 Comparison of Isolated and Non-Isolated Pier in Transverse Direction

<table>
<thead>
<tr>
<th>Pier</th>
<th>Isolated</th>
<th>Non-isolated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Design Earthquake Strength</td>
<td>Normalized Pier Force</td>
</tr>
<tr>
<td>2-pile pier P8</td>
<td>100% 0.181 0.708</td>
<td>32% 0.179 0.530</td>
</tr>
<tr>
<td>3-pile pier P12</td>
<td>100% 0.155 0.611</td>
<td>35% 0.156 0.472</td>
</tr>
</tbody>
</table>

#### 6.4.1.2 Influence of Scour Depth

To examine the effect of scour depth, as stated earlier, three different depth of pile fixity is considered. The response of pier and pile group for these three depth i.e. -44m PWD, -34m PWD and -24m PWD are presented in from Table 6.7 through Table 6.9.

### Table 6.7 Maximum Pier and Pile toe Moment for Pile Fixity at -44m PWD

<table>
<thead>
<tr>
<th>Axial Force(tf)</th>
<th>P8</th>
<th>P9</th>
<th>P10</th>
<th>P11</th>
<th>P12</th>
<th>P13</th>
<th>P14</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. Pier Moment (tf-m)</td>
<td>3876</td>
<td>3883</td>
<td>3293</td>
<td>3691</td>
<td>3889</td>
<td>4167</td>
<td>4256</td>
</tr>
<tr>
<td>Max. overturning moment at level of pile fixity</td>
<td>22257</td>
<td>22396</td>
<td>19111</td>
<td>19012</td>
<td>18934</td>
<td>21298</td>
<td>21588</td>
</tr>
</tbody>
</table>

### Table 6.8 Maximum Pier and Pile toe Moment for Pile Fixity at -34m PWD

<table>
<thead>
<tr>
<th>Axial Force(tf)</th>
<th>P8</th>
<th>P9</th>
<th>P10</th>
<th>P11</th>
<th>P12</th>
<th>P13</th>
<th>P14</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. Pier Moment (tf-m)</td>
<td>3741</td>
<td>3779</td>
<td>2989</td>
<td>3183</td>
<td>3402</td>
<td>3977</td>
<td>4081</td>
</tr>
<tr>
<td>Max. overturning moment at level of pile fixity</td>
<td>15421</td>
<td>15506</td>
<td>13241</td>
<td>13071</td>
<td>13367</td>
<td>14643</td>
<td>15241</td>
</tr>
</tbody>
</table>
Table 6.9 Maximum Pier and Pile toe Moment for Pile Fixity at -24m PWD

<table>
<thead>
<tr>
<th></th>
<th>P8</th>
<th>P9</th>
<th>P10</th>
<th>P11</th>
<th>P12</th>
<th>P13</th>
<th>P14</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Force(tf)</td>
<td>3166</td>
<td>3174</td>
<td>3182</td>
<td>3190</td>
<td>3198</td>
<td>3207</td>
<td>3215</td>
</tr>
<tr>
<td>Max. Pier Moment (tf-m)</td>
<td>3524</td>
<td>3537</td>
<td>2786</td>
<td>3117</td>
<td>3269</td>
<td>3901</td>
<td>3984</td>
</tr>
<tr>
<td>Max. overturning moment at level of pile fixity</td>
<td>10046</td>
<td>10102</td>
<td>8626</td>
<td>8515</td>
<td>8708</td>
<td>9539</td>
<td>9929</td>
</tr>
</tbody>
</table>

The maximum moment and corresponding axial force in each of the pier for three different pile fixity depths are compared with the pier interaction diagram (Fig. 6.5). All the piers fall well within the elastic zone of the diagram with pier P9 showing the highest moment for the case when pile is fixed at -44m PWD. Increase in the depth of pile fixity slightly increases pier moment. On the other hand, increase in depth of pile fixity increase the total overturning moment at point of pile fixity abruptly. For example, the 3-pile pier P12 has an overturning moment of 8708 tf-m when the piles are fixed at -24m PWD. Whereas, when the piles are assumed to be fixed at -44PWD, the total pile overturning moment increases up to 18934 tf-m.

Fig. 6.16 shows the influence of foundation stiffness on peak pier top displacement with respect to ground. The peak pier top displacement is found to increase rapidly with the increasing depth of foundation.

6.4.3 Effect of Earthquake Strength

To examine the effect of increase in input earthquake strength on normalized pier force, 3-pile pier P12 and 2-pile pier P9 is subjected to 125%, 150%, 175% and 200% of the design earthquakes. The resulting increase in peak normalized pier force is graphically shown in Fig. 6.17. For 100% increase in input ground motion the pier lateral force for both 2-pile and 3-pile piers are found to increase by an amount of around 23%.

Fig. 6.18 show typical isolator response curves for isolator placed over 3-pile pier P12.
**Fig. 6.16 Influence of Foundation Flexibility of Peak Pier Top Displacement**

**Fig. 6.17 Influence of Earthquake Strength on Normalized Pier Force**
Fig. 6.18 Typical Isolator Response Curves
6.4.2 Longitudinal Response

The same pier with or without the pintles is subjected to ground motion to study the benefit of seismic pintles in reducing pier forces in case of longitudinal vibration. The results are reported in Table 6.10. From the aforementioned results, it is apparent that the isolated 3-pile piers can withstand earthquake shaking up to around 3 times stronger than the non-isolated pier. But for 2-pile piers, the effectiveness of isolation is somewhat reduced due to high foundation flexibility. In this case, the isolated pier can withstand about 2.5 times stronger he ground motion that non-isolated pier can withstand.

Table 6.10 Comparison of Isolated and Non-Isolated Pier in Longitudinal Direction

<table>
<thead>
<tr>
<th>Pier</th>
<th>Isolated</th>
<th>Non-isolated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Design Earthquake Strength</td>
<td>Normalized Pier Force</td>
</tr>
<tr>
<td>2-pile pier</td>
<td>100%</td>
<td>0.079</td>
</tr>
<tr>
<td>P8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3-pile pier</td>
<td>100%</td>
<td>0.171</td>
</tr>
<tr>
<td>P12</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 6.19 shows the influence of pile fixity depth on peak pier top displacement whereas, the influence of different strength of earthquake on normalized pier force is presented in Fig. 6.20. The 2-pile pier is apparently less sensitive to the increase in earthquake strength as shown by the normalized pier force vs. earthquake strength curves. On the other hand, 3-pile pier P12 shows an increase of about 48% in normalized pier force when input earthquake is increased by 100%.
6.19 Influence of Foundation Flexibility on Peak Pier Top Displacement

Fig. 6.20 Influence of Earthquake Strength on Normalized Pier Force
CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE STUDY

7.1 CONCLUSIONS

Seismic response of base-isolated bridges is studied using a nonlinear finite element method. A detailed investigation of the influence of various bridge and isolator parameters on the bridge response, is performed, for a variety of moderate and strong ground motions. A wide range of parameters have been considered, however while studying the effect of a certain parameter, the following default values have been used for other parameters: Deck properties: weight = 2000t/ft/span, moment of inertia=20m$^4$ (longitudinal), 100 m$^4$ (transverse), cross-sectional area =12.5 m$^2$, damping =5%; Pier properties: lumped weight at pier top =200t, cross-sectional area =7.5 m$^2$, moment of inertia =7.5 m$^4$ (longitudinal), 40 m$^4$ (transverse), height=15m, damping=5%, for ductility studies, yield strength/weight=0.2, post-yielding stiffness to initial stiffness ratio=0.05; Isolator properties: yield force/weight=0.1, initial stiffness=18000t/ft/m, post-yielding stiffness =2700 t/ft/m. Concrete bridge with elastic modulus of 2700000 t/ft/m$^2$ was considered. Furthermore, as a case study, the dynamic response of Bangabandhu (Jamuna Multipurpose) Bridge is evaluated for the design earthquake.

After an extensive and systematic study, the following conclusions may be drawn:

- Comparison with results for complete model of multispan bridge shows that the simplified single pier model may be effectively used for conducting extensive parametric investigations on the pier and isolator response.

- Base-isolation achieves reduction of shear forces by a factor of 3 to 10 and reduction of ductility demand by a factor of 4 to 12 for piers.

- Seismic Isolation is most effective at large values of mass ratio ($m_d/m_p>10$). The pier force increases by 50% to 100% as the mass ratio is reduced from 15 to 5.

- Pier period significantly decreases the pier force and ductility demand for piers in case of strong earthquakes. For Pacoima 100%, the decrease in ductility demand (for $\varepsilon=0.2$) is from 2.8 to 1.2 when pier period is raised from 0.202 to 1.416 sec. For moderate earthquakes, decrease in ductility demand for increase in pier period is insignificant.
• Increase in isolator yield force results in gradual increase of pier force with apparently no effect on isolator displacement for moderate ground motions. The changes in pier force, as the normalized isolator yield force is varied between 0.05 to 0.20, ranges from 25% to 100%. For strong motions, the isolator yield force corresponding to 10% to 15% of the weight on it results in the lowest ductility demand in piers.

• The effect of post yielding stiffness \((K_h)\) of isolator is generally relatively small (within 20% variation) for moderate earthquakes. But for strong earthquakes like Kobe, in case of longitudinal vibration, the lateral force coefficient increases from 0.19 to 0.83 and ductility requirement varies from 1.2 to 13 corresponding to \(K_h\) value of 900 tf/m and 4500 tf/m respectively. Isolator displacement also increase significantly. Post yielding stiffness of isolator should be designed sufficiently low for strong earthquakes.

• The long period motion Hachinohe shows different response characteristics of the bridge compared to other motions of similar magnitude. This may be considered as a special case.

• Vertical ground motion occurring simultaneously with horizontal ground motion doesn't appear to have any significant effect on pier shear force. But the pier axial force is changed (increased & decreased) significantly.

• The response in transverse direction is different from the longitudinal response due to difference in structural configuration. However, the effect of various parameters shows influence that is similar in trend as in the case of longitudinal response.

• The dynamic response study of Bangabandhu (Jamuna Multipurpose) Bridge shows that seismic pintles has increased the earthquake resistance capacity of piers by 200% to 300% depending upon pile configuration and direction of seismic motion.

• Scour depth does not appear to influence the response of pier stem of JMB. However, the moment at pile base is significantly increased with increased scour depth.

7.2 RECOMMENDATIONS FOR FUTURE STUDY

The following recommendations for future study can be made from the present research:

• This study considers in-phase motion at the base of all the piers. The effect of out of phase motion at different piers for long span bridges may be evaluated.
• Longitudinal and transverse vibration have been treated separately in this study. Simultaneous application of horizontal motions in two directions along with vertical motion considering bi-axial bending of bridge pier may be studied.

• More detailed analysis of deck vibration may be performed.

• Nonlinear behavior of pier was studied in this research using a simple bilinear pier model. The behavior of the pier can be made more realistic by using Clough’s degrading stiffness relationship (Park and Paulay, 1975) or representing it by trilinear model.

• Seismic response of curved continuous isolated bridges may be evaluated.

• The effect of soil-structure interaction, which is ignored in this study, may be considered.
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