Response of Curved Flyover Bridges under Seismic Loading

by

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R.Mutsuddy

August, 2009 **(Rupak Mutsuddy)**

Dedicated to my mother Sharmila Mutsuddy, my father Samiran Mutsuddy, my wife Nilantika Barua and my grandfather Debabrata Chowdhury. Their unfailing inspirations made it materialize

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LIST OF ABBREVIATIONS

μ	Ductility Demand
Δ_u	Maximum lateral deflection at the end of plastic deformation
Δ_y	Lateral Deflection at the yield load
Δ_b	Isolator displacement
ω	Natural frequency
A	Acceleration coefficient
AASHTO	American Association of State Highway and Transportation Officials
BNBC	Bangladesh National Building Code
C_s	Seismic response coefficient
CHDBC	Canadian Highway Bridge Design Code
g	Acceleration due to gravity
HDRB	High Damping Rubber Bearings
K	Stiffness of the pier
K_i	Isolator Elastic Stiffness
K_h	Isolator Post Yield Stiffness
K_e	Isolator Effective Stifness
LRB	Lead Rubber Bearing
$P_{DL,\ pier}$	Supported Dead Load of Pier
PGA	Peak Ground Acceleration
PRSI	Partially Restrained Seismic Isolation
Q	Isolator Charateristic Strength
T	Period of bridge
S	Dimensionless coefficient for soil profile characteristics
STU	Shock Transmission Unit
V_{col}	Column Shear
W	Weight of structure

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ABSTRACT

Flyover bridges have become an essential part of the transport system of modern cities. They are constructed to ease the traffic congestion in mega cities and improve the mobility of the people. Dhaka City, already a mega city, is one of the most densely populated cities of the world with severe traffic congestion problems. There are at present two flyover bridges. The Govt. is planning to construct several additional flyover type structures in Dhaka City to improve its overall transportation system.

Flyover bridges have some unique features including curved geometry which make it different from conventional bridges. According to Bangladesh National Building Code Dhaka city falls in the moderate seismic risk zone. Flyover structures built in these areas need to be studied for seismic loading conditions.

In this study, response of flyover bridge structures subjected to earthquake motion is evaluated using non-linear finite element method. Bridges with different angle of curvature has been considered. Time history analysis is carried out as it is the widely used method for assessing non-linear seismic response of structures. Both conventional non-isolated and isolated bridges are analyzed. Bridge structure including isolation bearings have been modelled with beam-column elements and nonlinear spring elements. In the study, non linearity is restricted to the isolation bearing while rest of the structure is assumed to remain elastic at all times.

Parametric studies are carried out to identify variations in different bridge responses like pier and deck displacement, pier and deck forces due to change in bridge curvature. The responses of curved bridges are compared with corresponding straight bridge. The result shows that transverse responses (displacement, acceleration, transverse shear) increase with increase in curvature and longitudinal responses (displacement, acceleration and longitudinal shear) increase with increase in curvature. Effectiveness of isolation is also evaluated by comparing results of non-isolated and isolated bridge of identical geometric and material properties. Isolation significantly reduces o the pier forces and responses (displacement and acceleration).

As a case study, the first flyover bridge of Dhaka City, Mohakhali Flyover Bridge has been selected for seismic response analysis. Widely used earthquake record has been used after scaling down its PGA (Peak Ground Acceleration) value to 0.15g. It is observed that the flyover bridge doesn't form any plastic hinge due to prescribed earthquake excitation and behave elastically. Again, its seismic responses are compared with an equivalent straight bridge and the influence of curvature is carefully compared.

Chapter 1

INTRODUCTION

1.1 General

In a developing country like Bangladesh rapid urbanization encourage rural people to migrate from the villages to the major cities like Dhaka, Chittagong, Rajshahi and Sylhet. As a result the population density is increasing very rapidly. The capital city Dhaka is the densest city with a population of near 12 millions. But it doesn't have enough infrastructures to provide adequate transport facilities for increasing population. So infrastructural development like construction of flyover bridges, overpasses, elevated roadways etc. have become a necessity.

Flyover bridges are becoming an essential part of the modern transport system. In Dhaka city there are only two flyovers in service now (at Mohakhali and Khilgaon) and another flyover near Kuril, is proposed, but it would require construction of more flyovers to meet the future transport demand of the Dhaka city. Flyovers often have significant curved geometry in the horizontal plane; in addition they have curved ramps that may have a curvature angle as large as 180^{o}. Generally these flyovers are subjected to heavy traffic during rush hours and speedy traffic at other times. The structural stability and serviceability of these flyovers are of great importance during any natural disaster because they are the life lines to maintain transportation equilibrium in the city.

Again, among all natural hazards, earthquake is the one that need special attention. Earthquake is one of the most feared natural disasters causing enormous destruction of properties and loss and injury of many human lives. Earthquakes occur due to sudden release of accumulated stress within the earth crust, which may lead to ground shaking, ground failures and tsunamis. Ground shaking resulting from earthquake causes structural damage or collapse when the structure is incapable of resisting the additional stress due to this shaking.

Several earthquake of large magnitude (Richter scale 7.0 or higher) with epicenters within Bangladesh and India (close to Indo-Bangladesh border) have occurred (Ali and Choudhury, 1994). Table 1.1 provides the list of earthquake that affected Bangladesh. There are also some faults within Bangladesh, India and Myanmar which may become sources of earthquakes that could affect Bangladesh. 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. Considering these information Bangladesh National Building Code, (BNBC, 1993) places the major cities of Bangladesh like Dhaka, Chittagong, Sylhet in moderate to high seismic zone. So the assessment of responses of the flyover bridges due to earthquake is becoming an important issue.

Table 1.1 List of Major Earthquake affecting Bangladesh (After Ali & Choudhury, 1994)

Date	Name	Magnitude (Richter)	Epicentral Distance from Dhaka (km)
10 January, 1869	Cachar Earthquake	7.5	250
14 July, 1885	Bengal Earthquake	7.0	170
12 June, 1897	Great Indian Earthquake	8.7	230
8 July, 1918	Srimongol Earthquake	7.6	150
3 July, 1930	Dhubri Earthquake	7.1	250

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

Location	Operating Basis Magnitude (Richter)	Maximum Credible Magnitude (Richter)	Depth of focus (km)
Assam fault zone	8.0	8.7	0-70
Tripura fault zone	7.0	8.0	0-70
Sub-Dauki fault zone	7.3	7.5	0-70
Bogra fault zone	7.0	7.5	0-70

1.2 Research Objectives

The principal objectives of the research are

- To develop finite element models to carryout time-history analysis of multi-span continuous curved and straight flyover bridges.
- To examine the effect of curvature on the seismic response of curved flyover bridges using dynamic analysis procedures.
- To study the effect of isolator bearings in reducing the seismic loading for flyover bridges.
- To study the effect of various parameters on flyover response due to earthquake loading.
- To study the seismic response of an existing flyover (Mohakhali) in Dhaka City as a case study.

1.3 Methodology

Three Dimensional Finite Element models of curved and straight flyover bridges is developed. Bridges are modeled using beam-column elements. Linear and non-linear spring elements are used to simulate the bearings of the bridges. The computer program SAP2000 is used in this research. Records of earthquake with different characteristics are used in this study. Time history analysis is applied as it is the most preferred method for seismic response analysis. By comparing the results for equivalent straight flyover bridges with the corresponding curved ones, effects of curvature are assessed. Curved flyover bridges with and without seismic isolators are analyzed to assess the benefit of using isolator bearings.

1.4 Organization of Thesis

The thesis consists of seven chapters. The first chapter deals with the introduction, objectives of the research and methodology. Chapter two reviews the related literatures about bridge dynamics, bridge design philosophy, design methods of bridges, seismic responses of bridges and seismic behavior of flyover bridges. It also deals with seismic protective system, where seismic isolation is emphasized. It also presents the AASTHO design methods. Chapter three describes the flyover bridge model used in the study, finite element tools used and structural idealizations adopted in the study. The results of parametric study on non-isolated bridges are presented in chapter four. In chapter five, parametric study on isolated bridges is presented along with the evaluation of effectiveness of isolation in flyover bridges. In chapter six, seismic response of Mohakhali flyover is evaluated. The conclusions of the study and some recommendations for future research scope are presented in chapter seven.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

In this chapter the various methods for structural analysis and response of bridges under seismic loading conditions have been discussed. This review is divided into several parts which include the basic principles of dynamic behavior of bridges, the procedure for determining the magnitude of design loads, the component design forces and associated detailed design requirements specially the design philosophy, the design requirements of the AASHTO Guide Specifications (AASHTO, 2002). Different methods of structural analysis of bridges, response of straight and curved bridges, seismic behavior and response of straight flyover bridges are also discussed here. This review also includes description of different seismic protective systems used in bridges with particular emphasis on rubber isolation system.

2.2 Bridge Types and Analysis Methods

Bridges have some unique features compared to other civil engineering structures. Bridges are long in longitudinal direction and its structural properties and soil condition are not the same along the axial axis. Again it consists of many structural types like deck bridges, arch bridges, cable stayed bridges, suspension bridges, box-girder bridges etc. Moreover it has several shapes like straight, skewed, curved, separation into several segments and combination of those types with varying heights (short bridges and high bridges) (Kawashima 2006). But compared to other bridges, flyover bridges have some special features like acute curvature and short pier height. Flyover bridges are mostly box girder and deck girder type (Fig 2.1 and Fig 2.2). The two existing flyovers of Dhaka City have two different deck systems, Mohakhali Flyover Bridge is of box-girder type and Khilgaon flyover is of deck-girder type. In recent times the use of box-girder bridges in interchanges of modern highway systems has become increasingly popular for economic and aesthetic reasons (Sennah and Kennedy, 2001). In this study only box girder type bridges are used for parametric study of bridge seismic responses.

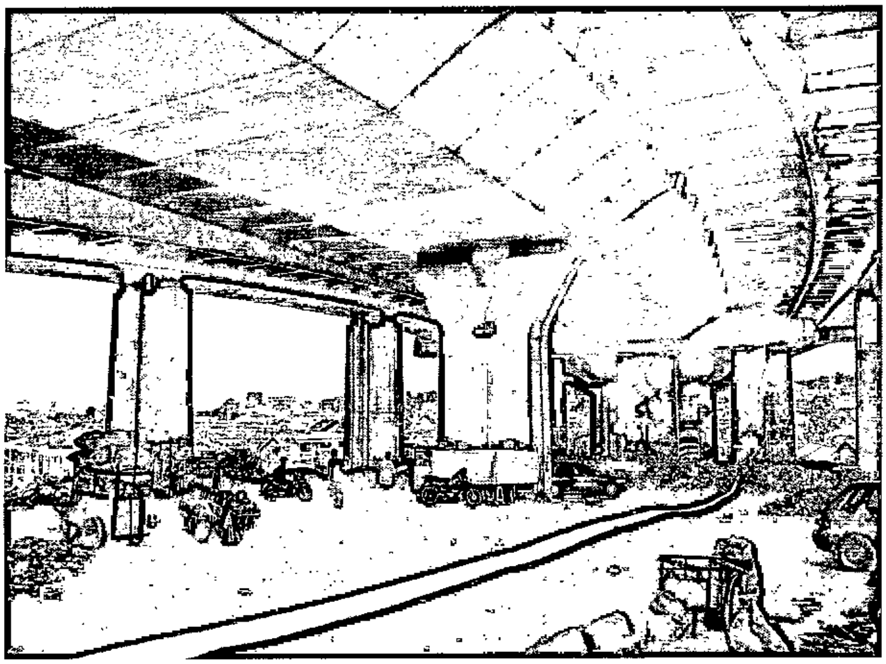


Fig 2.1 Flyover Bridge with Box Girder Section

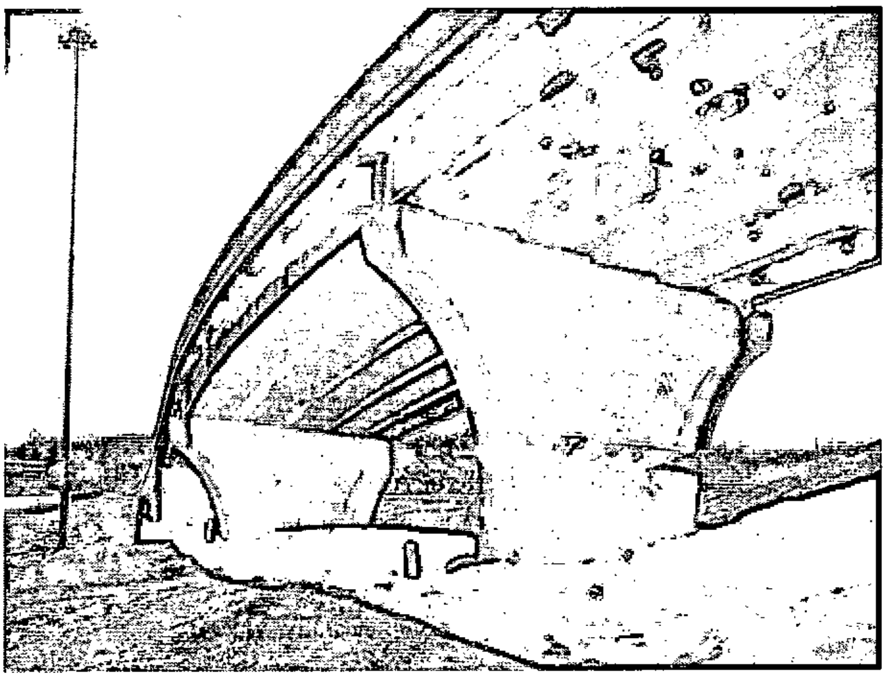


Fig 2.2 Flyover Bridge with Deck-Girder Section

2.2.1 Box-Girder Bridges

The bridges may be entirely constructed of reinforced concrete, prestressed concrete, steel, or composite concrete deck on steel I- or box girders. Concrete box girders are usually cast in situ or precast in segments erected on falsework or launching frame and then prestressed. The decks could be of steel, reinforced concrete, or prestressed concrete (Sennah and Kennedy, 2002).

As shown in Fig. 2.3, a box-girder cross section may take the form of single cell (one box), multiple spine (separate boxes), or multicell with a common bottom flange (contiguous cells or cellular shape).

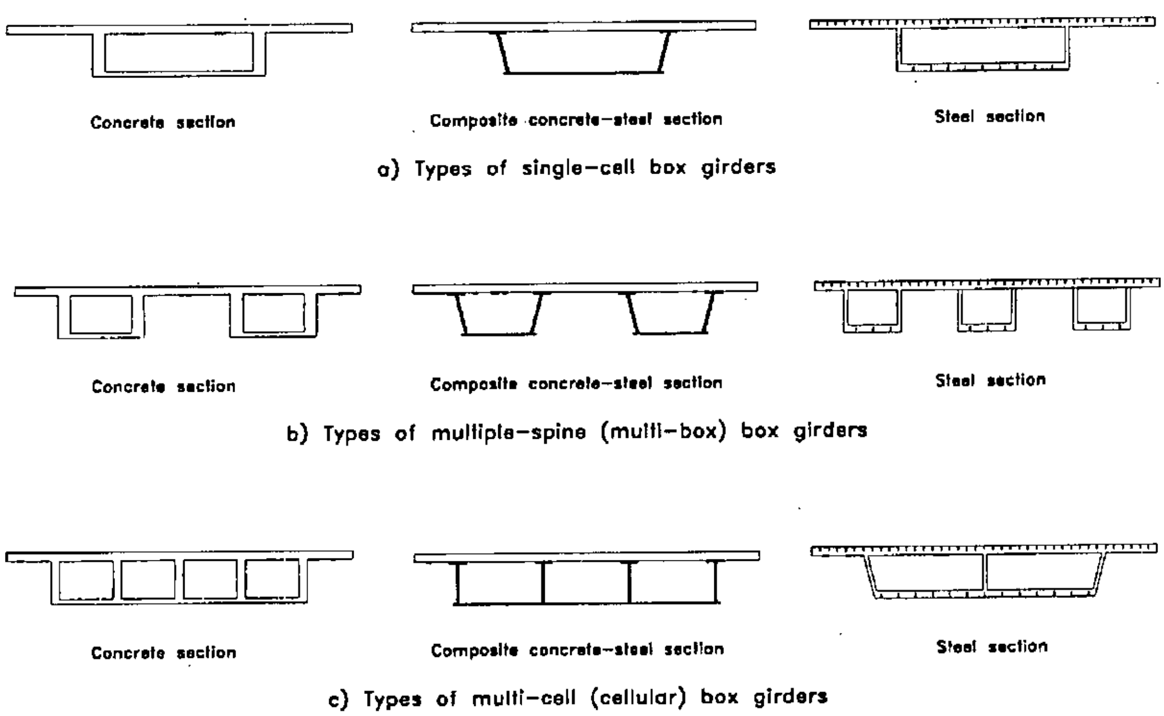


Fig 2.3 Different Box Girder Sections

Single or multi cell box cross sections often appear in single or multispan medium- and long-span bridges. The usual types of bridges are not economical for long spans because of the rapid increase in the ratio of dead load to total design load as the span lengths increase. The box girder concrete bridge has developed as a solution to this problem.

2.2.2 Different Analysis Methods for Box-Girder Bridges

In the design of bridges, analysis is usually simplified by means of assumptions that establish the relationship between the behavior of single element in the integrated structure. The combined response of these single elements is assumed to represent the response of the whole structure. The accuracy of such solutions depends on the validity of the assumptions made. The Canadian Highway Bridge Design Code (CHBDC 2000) as well as the American Association of State Highway Transportation Officials (AASHTO 2002) has recommended several methods of analysis for only straight box-girder bridges. Several authors have applied these methods along with the thin-walled beam theory to the analysis of straight and curved box-girder bridges.

These methods include:

- Orthotropic Plate theory,
- Finite-difference technique,
- Grillage analogy,
- Folded plate,
- Finite strip,
- Finite element technique,
- Thin-Walled Beam Theory.

In the following section, some of the methods are discussed briefly.

a) Orthotropic Plate Theory Method

In the equivalent orthotropic plate theory method, the stiffness of the flanges and girders are lumped into an orthotropic plate of equivalent stiffness, and the stiffness of diaphragms is distributed over the girder length. This method is suggested mainly for multigirder straight and curved bridges. However, the Canadian Highway Bridge Design Code (CHBDC 2000) has recommended the use of this method for the analysis of straight box-girder bridges of multispine cross section but not for multicell cross section. Bakht et al. (1981) presented the various methods of calculating the equivalent plate parameters, which are necessary for 2D analysis of straight cellular and voided slab bridges. Cheung et al. (1982) used the orthotropic plate method to calculate the longitudinal moments and transverse shear in multispine box-girder bridges.

b) Grillage-Analogy Method

Canadian Highway Bridge Design Code (CHBDC 2000) limits the applicability of this method to voided slab and box-girder bridges in which the number of cells or boxes is greater than two. In this method, the multicellular superstructure was idealized as a grid assembly by Hambly and Pennells (1975). Similar idealization was applied to curved multispine box-girder bridges by Kissane and Beal (1975). Cheung et al. (1982) dealt with the calculation of the longitudinal bending moment and transverse shear in multispine box-girder bridges using the grillage-analogy method.

c) Folded-Plate Method

The folded plate method utilizes the plane-stress elasticity theory and the classical two-way plate bending theory to determine the membrane stresses and slab moments in each folded plate member. The method has been applied to cellular structures by Meyer and Scordelis (1971), Al-Rifaie and Evans (1979), and Evans (1984). However, it was evident that the method is complicated and time-consuming.

d) Finite-Element Method

During the past two decades, the finite-element method of analysis has rapidly become a very popular technique for the computer solution of complex problems in engineering.

In this method a structure is represented as an assemblage of discrete elements interconnected at a finite number of nodal points. Chapman et al. (1971) conducted a finite element analysis on steel and concrete box-girder bridges Sisodiya et al. (1970) approximated the curvilinear boundaries of finite elements used to model the curved box-girder bridges by a series of straight boundaries using parallelogram elements. Chu and Pinjarkar (1971) developed a finite element formulation of curved box-girder bridges, consisting of horizontal sector plates and vertical cylindrical shell elements. Fam and Turkstra (1975) described a finite-element scheme for static and free-vibration analysis of box girders with orthogonal boundaries and arbitrary combinations of straight and horizontally curved sections. Ishac and Smith (1985) presented simple design approximations for determining the transverse moments in single-span single-cell concrete box-girder bridges. Chang and Zheng (1987) used a finite-element technique to analyze the shear lag and negative shear lag effects in cantilever box girders. Elbadry and Ibrahim (1996) determined the time-dependent temperature variations within the cross section and along the length of curved concrete single-cell box-girder bridges using a 3D finite-element model used in heat transfer. A similar study was presented by Gilliland and Dilger (1998).

2.3 Basic Dynamics of Bridges

The basic aim of any engineering design is to ensure that the resistance of the structure is greater than the effect of loads applied to it. Loads may be generalized as static load, dynamic loads, thermal loads etc. Dynamic loads are different from static loads due to their time varying nature. For bridge design various kinds of dynamic loads like wind, earthquake as well as dynamic effect of vehicle loads are considered. Load which is applied very slowly causes response which is virtually identical to the response due to static loading and the cyclic loading which is applied very rapidly has negligible effect on a structure (Biggs, 1964). Response due to dynamic loading is amplified when the frequency of applied load is close to frequency of the vibration of bridge (Clough and Penzien, 1993). In the following sections the details of bridge dynamics will be discussed.

2.3.1 Damping

In general, the process by which free vibration steadily diminishes in amplitude is called damping. All structural system exhibits damping to varying degrees by various mechanisms, such as friction at steel connections, opening and closing of micro cracks in concrete and friction between the structure itself and non-structural elements. Damping effect on dynamic response is generally beneficial. Structural damping is assumed to be viscous by nature. If damping coefficient which relates force to velocity is sufficiently large, it is possible to suppress the oscillatory motion completely, which is called 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 of 2 to 10 percent. This damping is very important in controlling peak displacements and forces of the bridge, especially near resonance and bringing the bridge to rest at the end of an earthquake.

2.3.2 Period of Vibration

The term natural frequency is used to mean the frequency at which a bridge will vibrate freely, without any dynamic excitation. Free vibration response is established by initially deflecting the structure, releasing it to vibrate without interference, and determining the time taken by the bridge to complete a given number of cycles. The number of cycles per unit time is the measure of natural frequency. The reciprocal of frequency is the time the bridge takes to complete one cycle of vibration. This time is called the period of vibration and it is used more common than frequency.

2.3.3 Response Spectra

Due to seismic excitation, structures are subjected to time-varying forces which produce time-varying displacements and stresses within these structures. Designers are interested in the maximum values of displacement and stress, the variation with time of these quantities is of less importance. Hence, only the peak value is needed to be known to design a bridge with adequate safety and this information is made available in the form of response spectra.

It is possible to generate curves which give peak displacements for any structures subjected to a given earthquake. The structure is idealized as single degree of freedom (SDOF) system and the peak response of the SDOF system is evaluated. 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 period or frequency and damping ratio. Figure 2.4 shows a typical response spectrum for El Centro (1940) Earthquake

2.3.4 Ductility Demand

In recent times bridges and buildings are economically designed to withstand seismic forces and it is achieved by permitting flexural yielding of the supporting piers. Flexural yielding in the piers implies deformation beyond the yield capacity of the pier. And this extent of deformation beyond yield is referred to as the ductility demand of the pier.

Figure 2.5(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 2.5 (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 therefore is 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

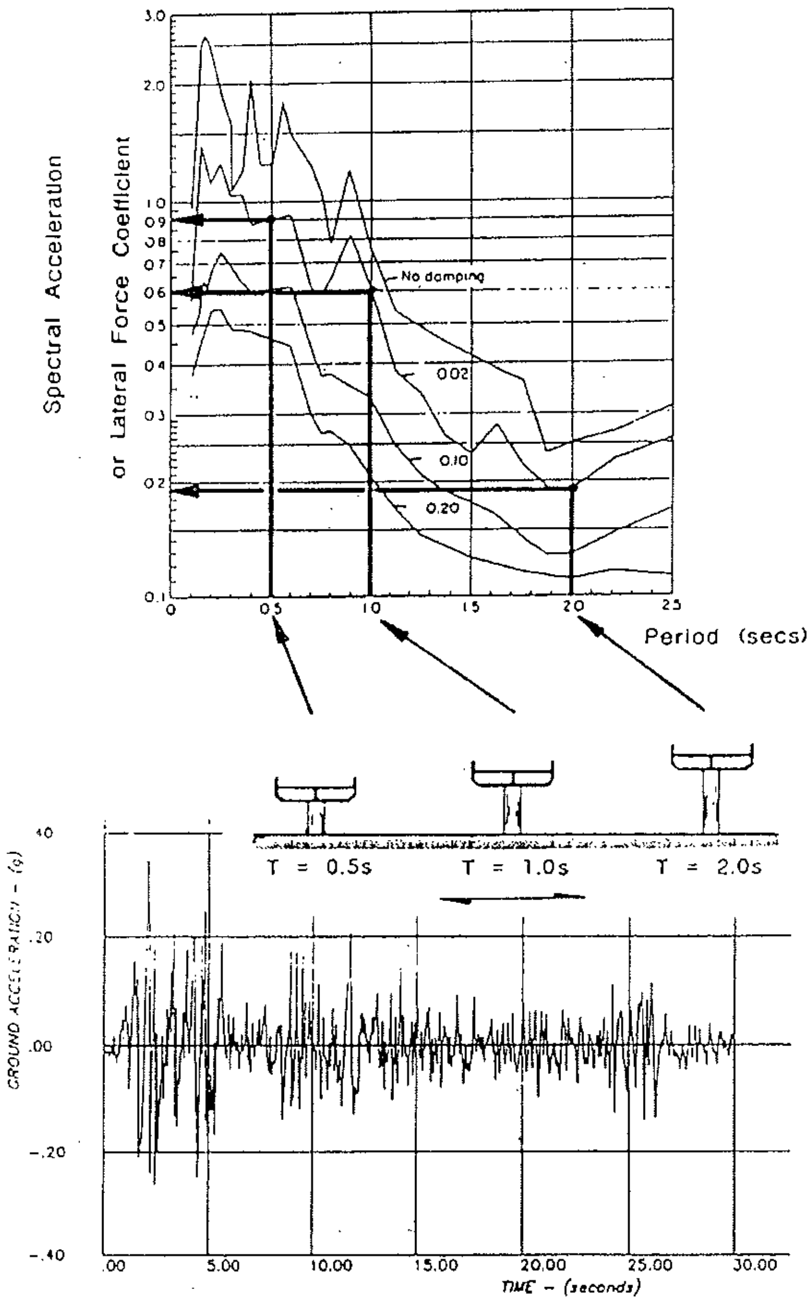


Fig 2.4 Acceleration Spectra for the El Centro (1940) Earthquake

of a pure elastic zone. This is because less energy is fed back into the system on return cycle.

If ductility is expressed in displacement terms, a ductility factor μ may be defined by

$$\mu = \frac{\Delta}{\Delta y} \tag{2.1}$$

Where Δ is the maximum lateral deflection at the end of plastic deformation and

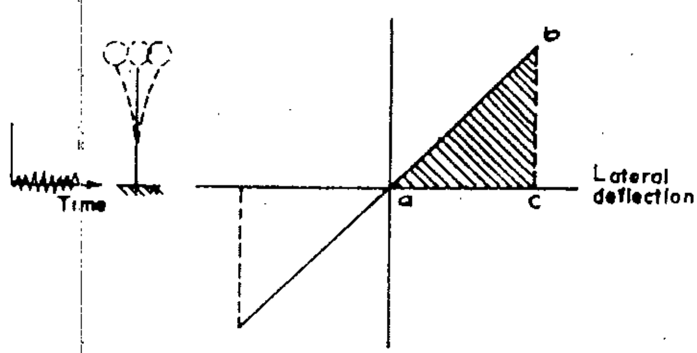
Δy is the lateral deflection when yield in the column is first reached.

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 Δu that can be sustained by the bridge columns without collapse.

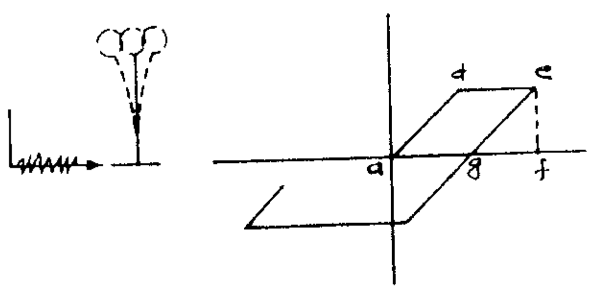
2.3.5 Basis for Bridge Seismic Design Philosophy

Usually, for dead loads and frequently occurring live loads, engineering design is based on elastic principles so that the capacity of the structure is sufficient to resist all possible loads with a specified margin of safety. But in case of earthquake loads this principle wouldn't be realistic for most bridges because of its probability of occurrence and its magnitude. Considering these, 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 withstand these loads without damage only.
- For more severe earthquake, which may occur once in the life time of a bridge, some structural damage is allowed but controlled to prevent collapse and maintain public safety and functionality.



(a) Elastic Response



(b) Elasto-Plastic Response

Fig 2.5 Idealized Response of a Single Column Pier

This design principle can be illustrated by means of a simple bridge example shown is Fig 2.6 and two response spectra given in Fig 2.7 (Buckle et al., 1987). The lower level spectrum in Fig 2.7 is representative of low to moderate earthquakes whereas the higher level spectrum is representative of a more severe event at the same site of 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.

But, it is uneconomical to design bridge to remain elastic under such a high lateral load. So we need to reduce the design shear force and allow some structural damages. But this reduction depends on the ability of the substructure to withstand this damage without collapse. For the illustrated single pier bridge 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 2.7. The elastic spectrum for a low-to-moderate earthquake for the same site 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.

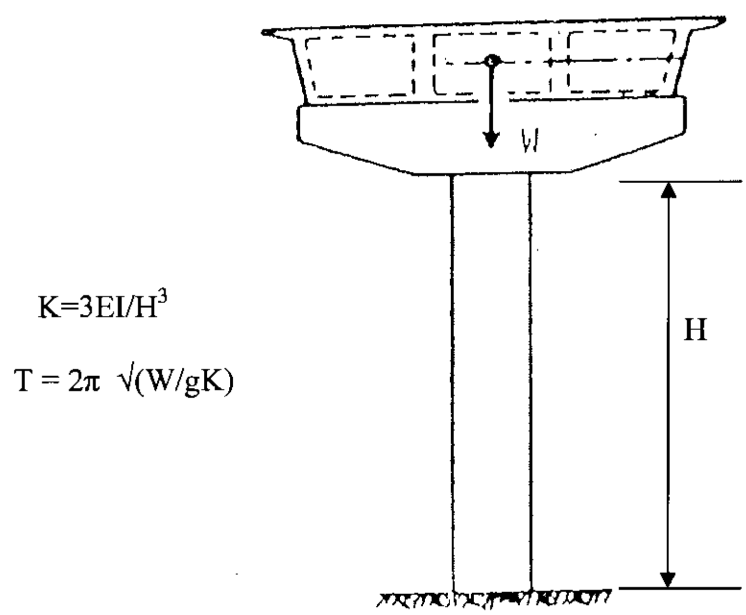


Fig 2.6 Simplified Model for Calculation of Period of Vibration

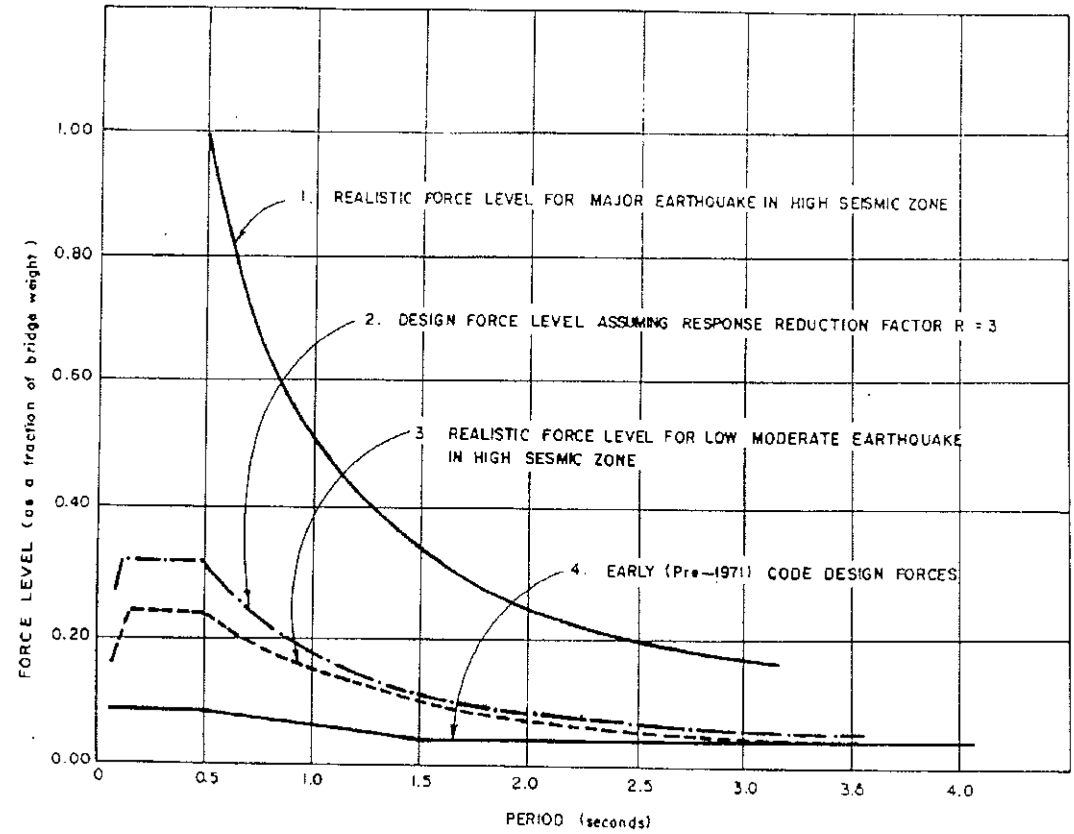


Fig 2.7 Comparison of Realistic and Design Earthquake Forces

2.3.5.1 Economic Consideration

In recent times the design philosophy of bridges is primarily influenced by economic consideration. A bridge can be designed such that it will suffer only minor damage in a major earthquake if the upper level curve in Fig 2.7 is used to design all the components elastically. However, the cost increases considerably due to this type of design. Thus, in development of a design philosophy, clearly stated objectives are required if a compromise between cost and safety is required.

Both acceptable and unacceptable types of damage were defined in the development of the AASHTO Guide specification. Detailed design and analysis requirements were then developed to achieve these performance criteria as follows.

(a) Acceptable Damage

Flexural yielding is the only form of damage that is allowed in piers. A steel reinforced concrete column can be subjected to many cycles of flexural yielding without risk of collapse if that column is well designed and detailed. In that pier any resulting damage will be visible and repairable.

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 and/or back wall impact.

(b) Unacceptable Damage

Loss of Girder support: 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: Generally, there are two types of reinforced concrete column failures that can lead to a catastrophic collapse, shear failures and pullout of the longitudinal reinforcement. To minimize the shear failure the capacity design approach is adopted in the AASHTO Guide Specification. 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.

Connection Failures: As connections are very important component of structural system to maintain overall integrity of the bridges, special attention is given to the displacement that occurs at moveable supports. For the fixed connections, conservative design forces are specified.

Liquefaction Failure: Past earthquake history shows that liquefaction of saturated granular foundation soil has been a major source of bridge failures. Investigations indicated that liquefaction of foundation soils contributed much to the damage, with loss of 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.

2.4 Seismic Protective Systems

Due to earthquake excitation, substantial amounts of energy are imparted into structures, which may cause the structures to deform excessively or even collapse. In order to remain in service after earthquake, they must have the capability to dissipate this input energy through either their inherent damping mechanism or inelastic deformation. This issue of energy dissipation becomes even more acute for bridge structures because most bridges, especially long-span bridges, possess low inherent damping, usually less than 5% of critical.

When these structures are subjected to strong earthquake motions, excessive deformations can occur by relying on only inherent damping and inelastic deformation. For bridges designed mainly for gravity and service loads, excessive deformation leads to severe damage or even collapse. Existing bridge seismic design standards and specifications are based on the philosophy of accepting minor or even major damage but no structural collapse. Lessons learnt from recent earthquake damage to bridge structures have resulted in the revision of these design standards and a change of design philosophy. For example, the latest bridge design criteria for California (ATC -32) recommend the use of a two-level performance criterion which requires that a bridge be designed for both safety evaluation and functional evaluation. A safety evaluation earthquake event is defined as an event having a very low probability of occurring during the service life of the bridge. For this design earthquake, a bridge is expected to suffer significant damage but the bridge will not collapse. A functional evaluation earthquake event is defined as an event having a reasonable probability of occurring once or more during the design life of the bridge. Damages suffered under this event should be immediately repairable. These design criteria have placed heavier emphasis on controlling the behavior of bridge structural response to earthquake ground motions.

For many years, efforts have been made by the structural engineering community to search for innovative ways to control how earthquake input energy is absorbed by a structure and hence controlling its response to earthquake ground motions. Among various techniques Seismic Isolation System is getting more and more acceptability among the structural engineers now a days due to its cost-effectiveness and innovative solution.

2.4.1 Different Types of Seismic Protective System:

The functions of an isolating/dissipating system are generally one or a combination of the following: (i) supporting gravity loads and providing for (ii) lateral flexibility (period shift), (iii) restoring force and (iv) energy dissipation (either of hysteretic, in the case of displacement activated dampers, or viscous nature, in the case of velocity activated dampers); According to their performance, the anti-seismic devices can be grouped as: rigid connection devices (e.g. shear links, lock-up devices), linear devices, non linear devices, viscous dampers, isolators (e.g. sliders, rubber bearings) etc.

Common types of seismic protective devices are:

- Elastomeric bearings: Natural Laminated, Lead and High Damping Rubber Bearings (HDRB)
- Friction Dampers
- Metallic Dampers (sometimes combined with bearings to form sliders): yielding steel systems, lead extrusion devices;
- Viscous and Viscoelastic Dampers: Taylor Devices
- Self-centering Dampers: Shape Memory Alloys, Energy Dissipation Restraints, SHAPIA Devices
- Self-centering Dampers: Shape Memory Alloys, Energy Dissipation Restraints (sometimes combined with Hysteretic Dampers)

2.4.2 Basic Elements of Isolation System:

There are three basic elements in any practical isolation system:

- A flexible support so that the period of vibration is lengthened sufficiently to reduce the force response.
- A damper or energy dissipator so that the relative deflections across the flexible support can be limited to a practical design level.
- Rigidity at low (service) load levels such as wind and braking forces.

(a) Flexibility: The process of a structure responding to earthquake ground motions is actually a process involving resonance buildup to some extent. The extent of response is closely related to the amount of energy and frequency content in the earthquake loading. Hence preventing resonance is a measure of controlling the response of structure and this is achieved by isolation system, as it lengthens the period of the structure. The idealized force response with increasing period (flexibility) is shown schematically in the acceleration response curve of Fig. 2.8. Reductions in base shear occur as the period of vibration is lengthened. But the additional flexibility needed to lengthen the period will give rise to relative displacements across the flexible support. Fig. 2.9 shows an idealized displacement response curve from which displacements are seen to increase with increasing period (flexibility).

(b) Energy Dissipation: The response of the structure can also be controlled by providing a supplementary energy dissipation mechanism. Seismic Isolation System provides this sort of energy dissipation by introducing substantial additional damping into the structure at the isolation level. One of the most effective means of providing a substantial level of damping (greater than 20% equivalent viscous damping) is hysteretic energy Dissipation. Fig. 2.10 shows the smoothing effect of higher damping and Fig. 2.11 shows an idealized force-displacement loop where the enclosed area is a measure of the energy dissipated during one cycle of motion.

(c) Rigidity under low lateral loads

While lateral flexibility is highly desirable for high seismic loads, it is clearly undesirable to have a structural system that will vibrate perceptibly under frequently occurring loads, e.g., wind or braking loads. Mechanical energy dissipators may be used to provide rigidity at these service loads by virtue of their high initial elastic stiffness.

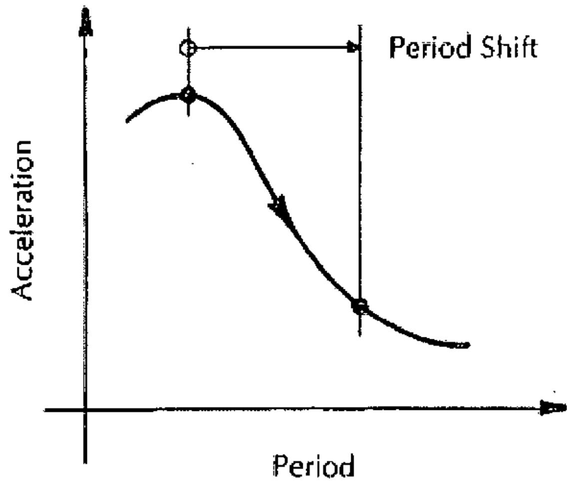


Fig 2.8 Idealized Acceleration Response Spectrum

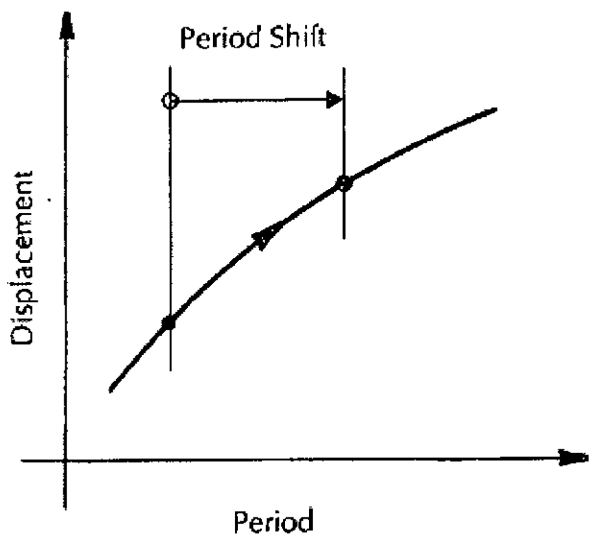


Fig 2.9 Idealized Displacement Response Spectrum

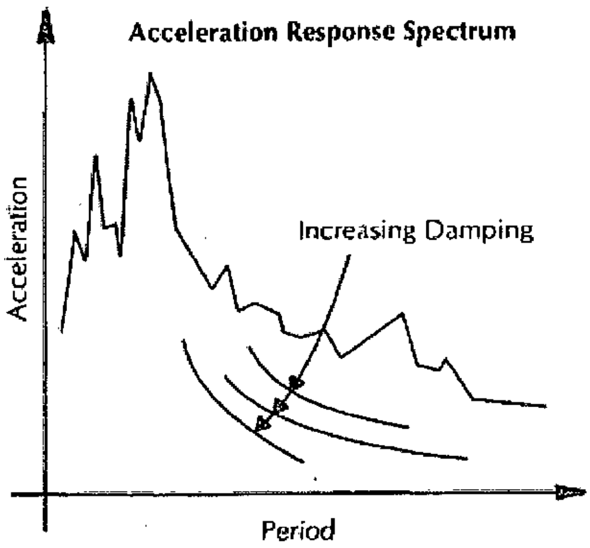


Fig. 2.10 Response Spectra for Increasing Damping

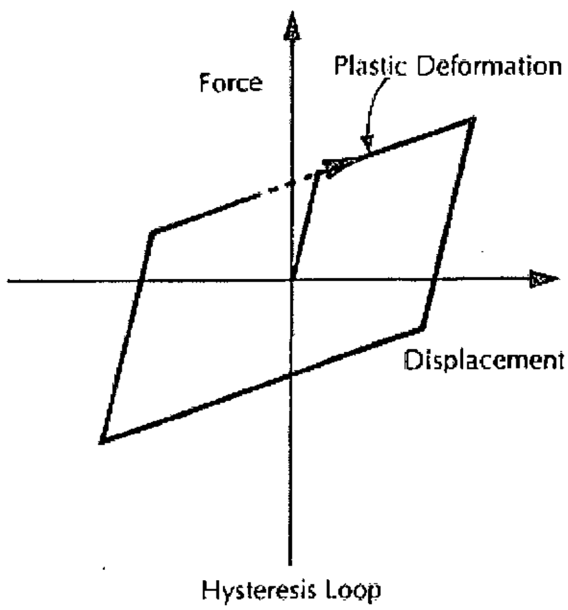


Fig. 2.11 Hysteretic Force Displacement Curve

2.4.3 Design Principles of Seismic Isolation:

The design principles for seismic isolation are shown in Fig. 2.12. The top curve on this figure shows the elastic forces that will be imposed on a conventional fixed-base structure (from the new AASHTO standard specifications) for a rock site if the structure has sufficient strength to resist this level of load. The lowest curve shows the forces for which the AASHTO standard specifications requires a multi-column bent bridge to be designed, and the second lowest curve shows the probable strength, assuming the structure is designed for the AASHTO forces. The probable strength is 1.5-2.0 times higher than the design strength because of the design load factors, actual material strengths (which are greater in practice than those assumed for design), conservatism in structural design, and other factors. The difference between the maximum elastic force and the probable yield strength is an approximate indication of the energy that must be absorbed by ductility in the structural elements. However, when the bridge is isolated, the maximum forces are reduced considerably due to period shift and energy dissipation. The forces on a seismically isolated structure are shown by the small dashed curve in Fig. 2.12. If a seismically isolated bridge is designed for the AASHTO forces in the period range of 1.5-3.0 sec as shown in Fig. 2.12, then the probable yield strength of the isolated bridge is approximately the same level as the maximum forces to which it will be subjected. Therefore, there will be little or no ductility demand on the structural system, and the lateral design forces are reduced by approximately 50%.

2.4.4 Benefits of Seismic Isolation:

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

1. Reduction in the design forces to which a bridge will be subjected by a factor 3 to 10 (based on curves 1 and 2 of Fig. 2.12 and a period shift due to isolation, from 0.4 to 2 sec).
2. Elimination of the ductility demand and thus damage to the piers due to earthquake is avoided
3. Control of the distribution of the seismic forces to the substructure elements with appropriate sizing of the elastomeric bearings.

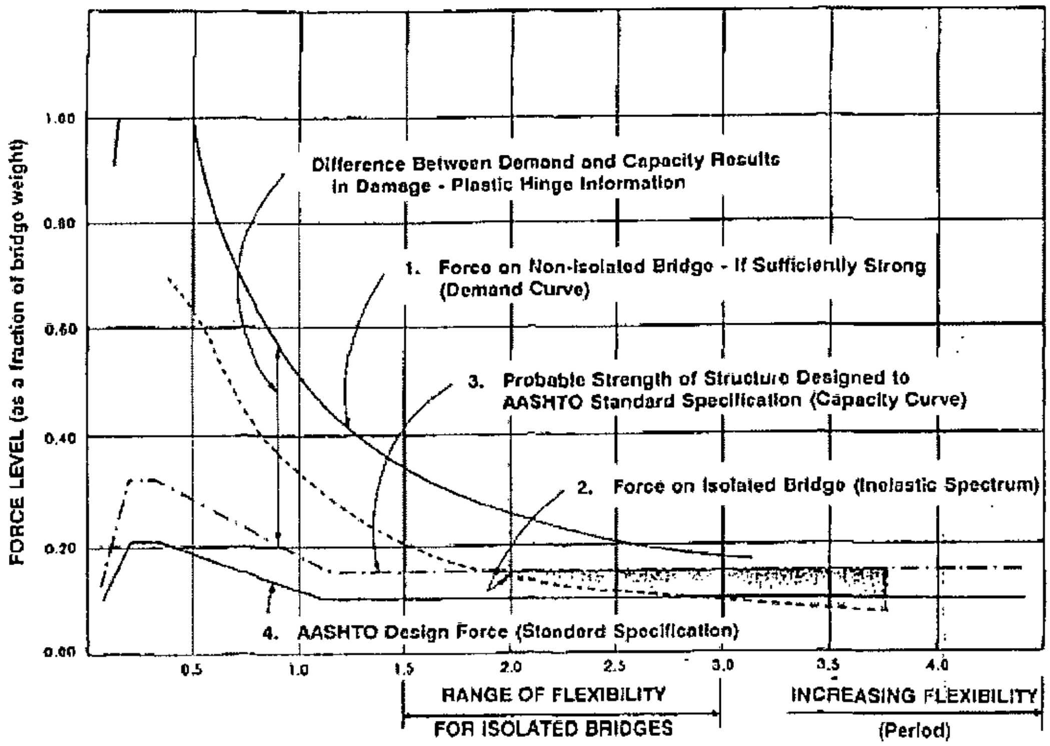


Fig. 2.12 Earthquake Forces (Mayes et al. (1992))

2.5 Response of Straight and Curved Bridges to Vehicular and Environmental Forces

The principal live load acting on bridges is the load imposed by moving vehicles. Bridges response due to vehicular load varies with their curvature. DeSantiago et al., (2005) performed analysis on a series of horizontally curved deck girder bridges with using a typical truckload and dead load. In each analysis, the behavior of bridges was investigated, and the major internal forces developed in members were determined. From the analysis they found that the vertical deflection of a curved bridge is about 80% higher than the deflection of a straight bridge. This is when the angle of curvature is 30° and a reasonable design for lateral bracing support system is used. Compare to straight bridges, curved bridge girders experience about 23.5% higher bending moments for an angle of curvature of 30°. The introduction of curvature in a bridge will also result in a sizeable torsional moment to develop in the girders.

Bridges are also subjected to seismic loads, wind loads and other dynamic loads. Bridge responses due to these dynamic loads depend much on the dynamic properties of bridges. Dynamic properties of straight and curved bridges vary. Cheung and Cheung (1972) determined the natural frequencies and mode shapes of undamped vibrations of curved and straight single-span beam-slab or box girder bridges using the finite strip method. Heins and Sahin (1979) evaluated the first natural frequencies of straight and curved continuous multiple-spine box girder bridges using the finite difference technique. Samaan et al. (2007) conducted free vibration tests on two continuous two-span bridge models. They have found that fundamental frequency decreases with increase in the bridge curvature considering other parameters to be constant. From this study it is found that the above mentioned effect of bridge curvature on fundamental frequency holds true for box girder type bridges also.

Hosada et al. (1991) carried out a series of earthquake response analysis of a particular curved isolated prestressed concrete girder bridge and a similar straight bridge using an analytical approach. It was found that the seismic responses at an individual isolation bearing and substructure in the tangential direction to the deck axis were larger than the corresponding response of the straight bridge.

Al-Hussaini et al. (1998) performed extensive parametric studies on straight bridge to show the influence of key bridge and isolator parameters like distribution of mass, isolator yield forces, post yield stiffness etc. on the seismic response of base isolated bridges. Jangid (2004) compares effectiveness of isolation by performing analysis on both isolated and non isolated straight multispan continuous deck bridges under bi-directional earthquake excitation. The bridge is isolated by LRB (Lead Rubber Bearing) and the interaction between the restoring forces of the bearings in two horizontal directions is duly considered in the response analysis.

2.6 Seismic Behavior of Flyover Bridges

Compared to the conventional bridges, flyover bridges have some distinctive characteristics. Usually conventional bridges have longer piers and carry loads of very long span. But in flyover bridges the span is relatively small and their piers are also short. Also the flyover bridges have curved loop within very short span. Hence their response is quite different from that of conventional bridges. Earthquake forces are found to target structural weaknesses and concentrate damages at connections and piers of bridges. In recent earthquakes it was noticed that superstructures as well as substructures of bridges of different spans were damaged considerably e.g. cracking, spalling of concrete, tilting and overturning of piers, resulting in partial/ total collapse of the whole bridge. Seismic problems of Flyover Bridges have been highlighted by Jain et al, (2006) as follows;

- Due to unequal height of piers and unequal spans, there is a possibility of out of phase motion of piers which may result in falling of spans.
- In flyover bridges, the piers have unequal heights in the ramp portions..
- The vertical vibrations of spans could be important.
- In case of skewed flyover bridges, large ground motions can introduce hinge seat movements.
- Such bridges generally have curved loops. The seismic response may be asymmetrical in this case that may result in falling of spans.
- Strong ground motions can result in dislocation of bearings and expansion joints especially in the case of curved bridges.

2.7 Performance of Flyover Bridges in Past Earthquake

Damage to bridge structures may occur in the superstructure, substructures, foundations or the abutments. Typical structural damages to flyover bridges are discussed in the following sections (Iwasaki et al, 1973). Most failures occur due to horizontal component of ground motion.

2.7.1 Superstructure

Superstructure damage include the loss of support for the girders which may be caused by a lack of continuity in the superstructure (Fig 2.13), inadequate support length for the girders, skew supports which may cause in-plane torsion of the superstructure or gross movements at the superstructure supports due to some form of soil failure under the piers or abutments.

2.7.2 Substructure

Substructure damage includes damages of columns (Fig 2.14), abutments and foundations (piles, footings) etc. Column damage can be caused by the flexural failure, shear failure and anchorage failure of longitudinal reinforcement. Substructure damage may also cause collapse of the superstructure by removal of support for the superstructure.

2.7.3 Foundations

Foundation damage due to seismic excitation occurs due to 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 and consequently experience severe cracking or complete failure.

2.7.4 Abutments

Usually abutments have high stiffness and attract large share of the seismic inertia forces developed in the superstructure. These forces can be very high and may cause brittle failure of abutments. Also the shear key failure in abutments occurs due to inadequate reinforced pedestal.

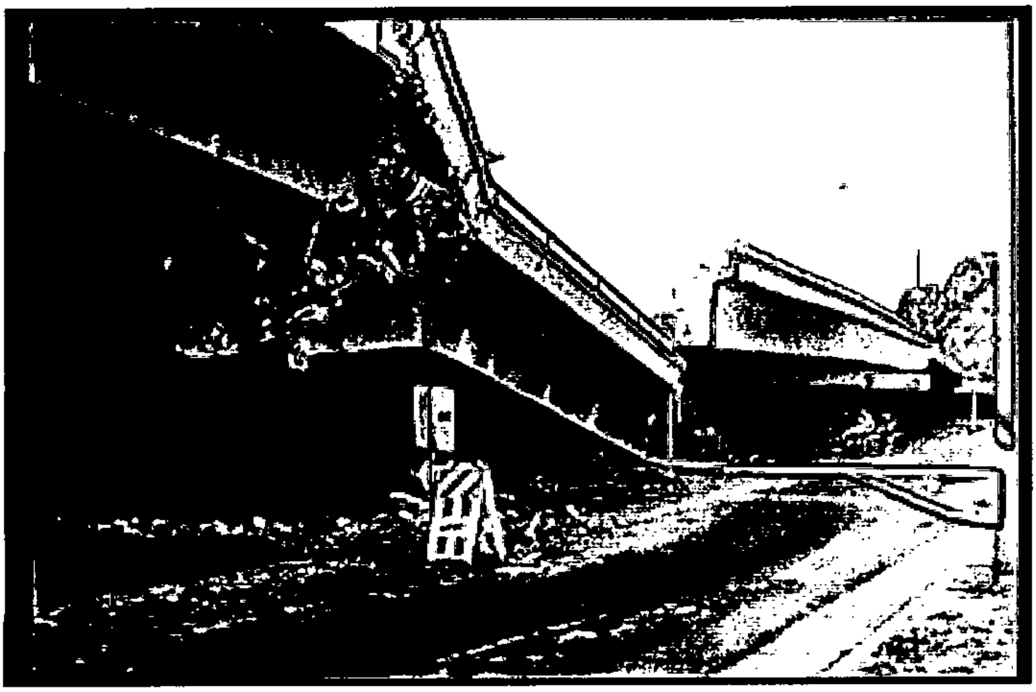


Fig 2.13 Superstructure Failure of a Flyover Bridge due to Earthquake (Kobe Earthquake)

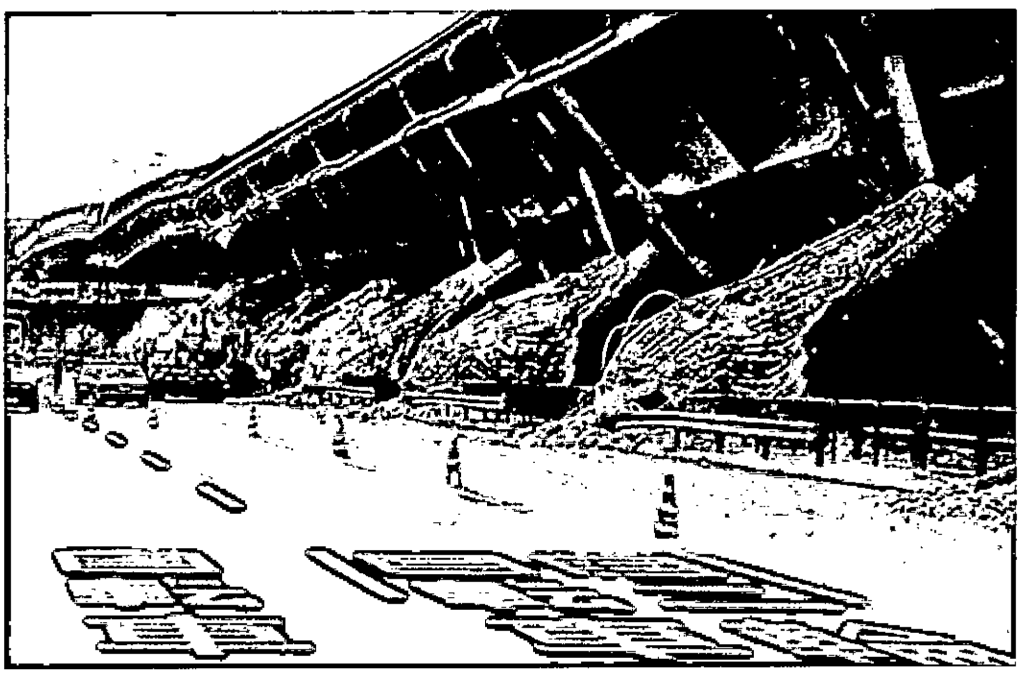


Fig 2.14 Pier Failure of a Flyover Bridge due to Earthquake (Kobe Earthquake)

2.8 Earthquake Loading for Design

In the AASHTO Guide Specification (2002), the design loads are expressed as a design coefficient or design response spectra which represents the expected force level for the site of interest. These force levels are derived such that they have a 10 to 20 percent probability of being exceeded every 50 years and are a function of the acceleration coefficient and the site soil conditions.

2.8.1 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 activity to which the bridge will be exposed. The seismic zoning map proposed by Bangladesh National Building Code (BNBC 1993) is shown in Fig 2.15

2.8.2 Design Earthquake Ground Motion

The determination of appropriate seismic design loads has been significantly simplified for code application. The design ground motion for a location is the ground motion that the engineers should consider when designing a structure to provide a satisfactory degree of protection for life safety and to prevent collapse.

2.8.3 Influence of Soil Condition on Ground Motion

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 of the earthquake producing the ground motions
- The distance of the earthquake from the proposed site and the geology of the travel path.

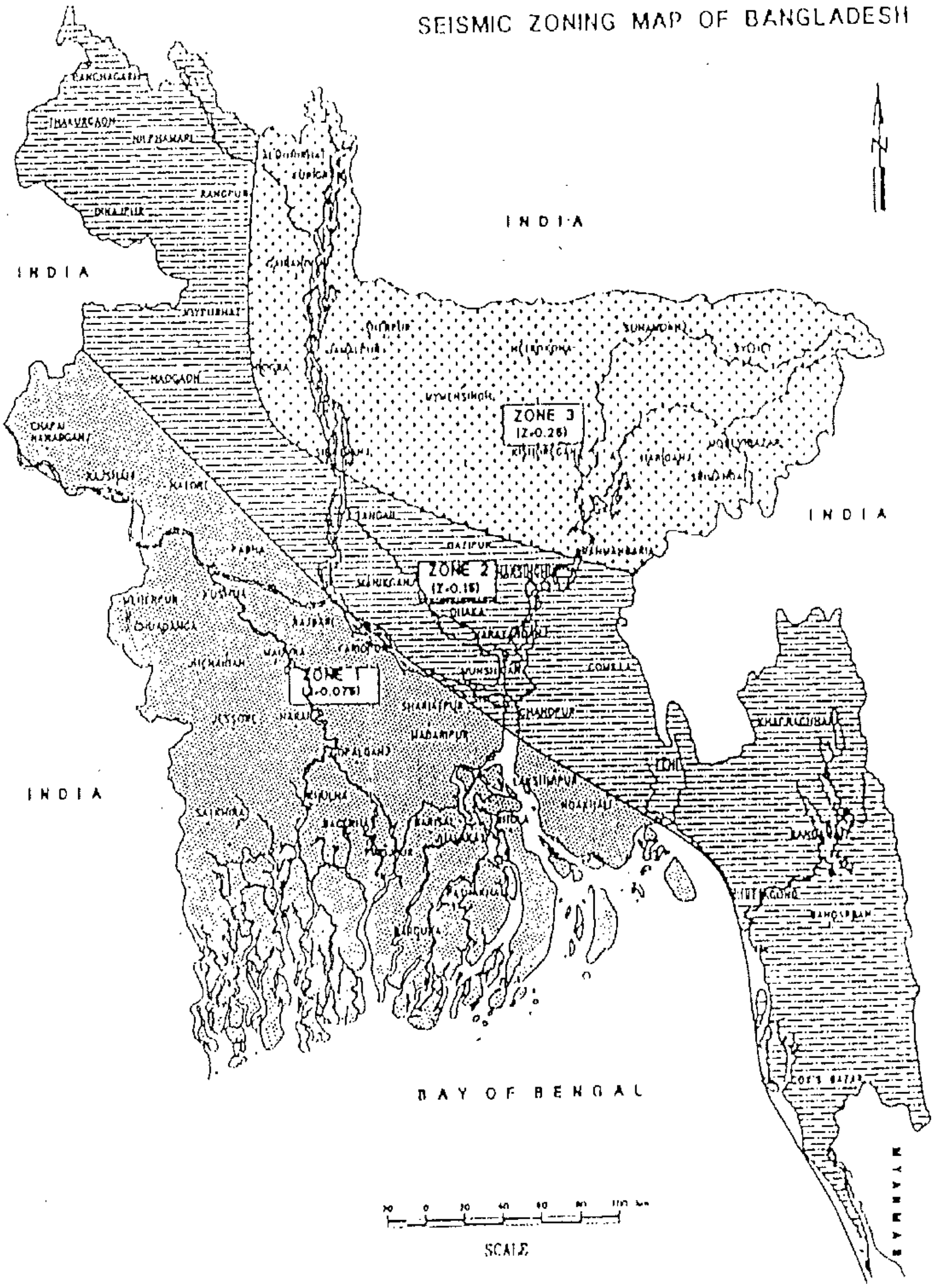


Fig 2.15 Seismic Zoning Map of Bangladesh

Considering the effects of soil condition over the ground motion different design criteria have been developed. Typically these criteria use four different soil conditions, which are also included in the AASHTO Guide Specification.

These soil profile types are as follows

Soil Profile Type I is a profile with either

- Rock of any characteristics, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 2500 ft/sec (760 m/sec), or by other appropriate means of classification)
- Stiff soil conditions where the soil depth is less than 200ft (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 200ft (60 meter) and the soil types overlying rock are stable deposits of sands, gravel, or stiff clays.

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

Soil Profile Type IV is profile with soft clays and silts greater than 40 feet in depth. These materials may be characterized by a shear wave velocity less than 500 feet/second and might include loose natural deposits or synthetic, nonengineered fill.

2.8.4 Method of Analysis:

The determination of appropriate seismic design loads has been significantly simplified for code application. The AASHTO seismic guide specification suggests the following four analysis procedures:

2.8.4.1 Procedure 1- Uniform Load Method

The equivalent static earthquake loading is given by the expression

$$p_e = C_s \frac{W}{L} \tag{2.2}$$

Where C_s is the dimensionless elastic response coefficient, W is the total weight of the bridge and L is total length of the bridge

C_s is determined by the following equation,

$$C_s = 1.2 \frac{AS}{T^{2/3}} \tag{2.3}$$

Here, A is an acceleration coefficient given by the seismic zoning map

S is dimensionless coefficient for the soil profile characteristics of the bridge site as given in the Table 2.1

T is the period of the bridge in seconds

Table 2.1 Site Coefficient

	Soil Profile Type			
	I	II	III	IV
S	1.0	1.2	1.5	2.0

2.8.4.2 Procedure 2 - Single Mode Spectral Analysis:

The single mode spectral analysis 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_o. The uniform loading p_o is applied over the length of the bridge. It has units of force/unit length and arbitrarily set equal to 1. The static displacements $v_s(x)$ has units of length.

In step 2, factors α, β and γ are calculated using the following equations

$$\alpha = \int v_s(x)dx \qquad (2.4)$$

$$\beta = \int w(x)v_s(x)dx \qquad (2.5)$$

$$\lambda = \int w(x)v_s(x)^2 dx \qquad (2.6)$$

Here, w(x) is the weight of the dead load of the bridge superstructure and tributary substructures (force/unit length). The factors α, β and γ have units of (length^2), (force-length), (force-length^2) respectively.

In step 3, period of the bridge, T is calculated using the expressions

$$T = \frac{2\pi}{\sqrt{\dfrac{\gamma}{p_0 g \alpha}}} \qquad (2.7)$$

Where g = acceleration due to gravity.

In step 4 the equivalent static earthquake loading $p_e(x)$ is calculated from the following expression:

$$p_e(x) = w(x)v_s(x)C_s\frac{\beta}{\gamma} \qquad (2.8)$$

Where C_s is calculated from the Eq. 2.3

In step 5, $p_e(x)$ is applied to the bridge structure to determine the resulting member force and displacements for design.

2.8.4.3 Procedure 3 – Multimode Spectral Analysis

The multimode spectral analysis method is applied 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. 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 supports in any one of the two horizontal directions will produce forces along both principal axes of the individual members because of the coupling effects.

2.8.4.4 Procedure 4 – Time History Method

Time history method of dynamic analysis may be used if the following requirements are satisfied:

- The time histories of input acceleration used to determine the earthquake loads shall be selected in consultation of the Owner or Owner's representative. Unless otherwise directed, five spectrum compatible time histories shall be used when site-specific time histories are not available. The spectrum used to generate these five time histories shall preferably be a site-specific spectrum. In the absence of such a spectrum the response coefficient given by equation 2.2 for the appropriate soil type, may be used to generate a spectrum.
- The sensitivity of the numerical solution to the size of the time step used for the analysis shall be determined. A sensitivity study also carried out to investigate the effects of variations in assumed material properties.
- If an in-elastic time history method of analysis is used the response factors permitted by AASHTO code shall be taken as 1.0 for all substructures and connections.

2.9 Conclusion

From the above literature review, it is found that many research works have been carried out on the behavior of straight bridges, skewed bridges and curved bridges under vehicle loading. Dynamics of various bridges along with the effect of dynamic loading also studied. In some studies bridges are analyzed individually and in others comparisons have been made between two types of bridges. Effect of angle of curvature on bridge response due to vehicular loading also studied. But effect of bridge curvature on various bridge responses due to seismic excitation has not been studied yet. For the reason the current study focuses on this particular topic.

Chapter 3

STRUCTURAL ANALYSIS MODEL

3.1 Introduction

Usually design methods for bridges subjected to seismic loading allows for equivalent static loads. But, it is recommended to have a dynamic response analysis when bridges are not 'regular' as shown mentioned in Table 3.1, specified by AASHTO Standard Specification of Highway Bridges (2002).

Table 3.1 Regular Bridge Requirements

Parameter	**Value**				
Number of Span	2	3	4	5	6
Maximum subtended angle	90^{o}	90^{o}	90^{o}	90^{o}	90^{o}
Maximum span length ration from span-to-span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span-to-span (excluding abutments)	---	4	4	3	2

The curved bridge considered in this study is 'regular', but time-history analysis is used with the objective of developing a better understanding of the complex dynamic behavior of bridges. A nonlinear finite element program is used to study the response of both non-isolated and isolated bridges. Brief descriptions are provided in this chapter on the main features of the program used for analysis.

The acceptability of the results obtained depends considerably on effective modeling of the bridge system. The structural idealizations for simplified modeling techniques of actual bridge are also described here.

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

3.2 The Flyover Bridge Structure

A flyover bridge is different from typical bridges because of its geometry, span length and pier height. It usually consists of several span of length of 30 to 60 meters, with very acute curvature. Also it has very short pier height compared to the conventional bridges. In this study a typical two span bridge is considered with span length of 50 meter each. The bridge is reinforced concrete box-girder type continuous over two spans. It is supported by three identical single column pier of height 8 meters each. For non-isolated bridges, normal rubber bearings are used and for isolated bridge rubber-isolator bearings are used.

3.3 Finite Element Modeling of the Bridge Structure

3.3.1 Finite Element Software

In this study the computer software SAP2000 (version Advanced 9.0.4) was used for finite element modelling and dynamic analysis of the isolated and non-isolated bridges. SAP2000 is a fully integrated program that allows model creation, modification, execution of analysis, design optimization, and results review from within a single interface. A Finite Element Dynamic analysis using SAP2000 consists of three distinct stages: preprocessing, simulation and post processing.

When an analysis is run in SAP2000, it automatically converts the object-based model into an element-based model that is used for analysis. This element-based model is called the analysis model, and it consists of traditional finite elements and joints (nodes). Results of the analysis are displayed on the object-based model.

SAP2000 provides options to control how meshing is performed, such as the degree of refinement, and how to handle the connections between intersecting objects.

3.3.2 Structural Idealization

The following assumptions are made for the earthquake analysis of non-isolated and isolated bridges under consideration,

(i) The bridge superstructure and piers are assumed to remain in the elastic state during the earthquake excitation.

(ii) The superstructure of the bridge is modeled as a lumped mass system divided into a number of small discrete segments. Each adjacent segment is connected by a node, and at each node six degrees of freedom are considered. The masses of each segment are assumed to be distributed between the two adjacent nodes in the form of point masses.

(iii) Simple beam-column elements are used to model the bridges.

(iv) The bridge piers are assumed to be rigidly fixed at the foundation level which means that soil-structure interaction is ignored.

(v) Two horizontal components of an earthquake ground motions are applied simultaneously in time-history analysis.

(vi) The stiffness contribution of nonstructural elements such as curbs, parapet walls, and the wearing coat is neglected.

(vii) The bridge is founded on firm soil or rock, and the earthquake excitation is perfectly correlated at all of the supports.

(viii) The Lead Rubber Bearing (LRB) used in isolated bridge is isotropic, implying the same dynamic properties in two orthogonal directions.

(ix) The force-deformation behavior of the L-RB is considered as bilinear, based on the nonlinear model proposed by Park et al. (1986), which had been widely used in the past (Nagarajaiah et al. 1991; Jangid and Datta 1994)

3.3.3 Materials and Sections

In the study, rectangular RCC pier and box-girder type deck has been used. A typical box-girder section and pier section are shown in Fig 3.1 and Fig 3.2

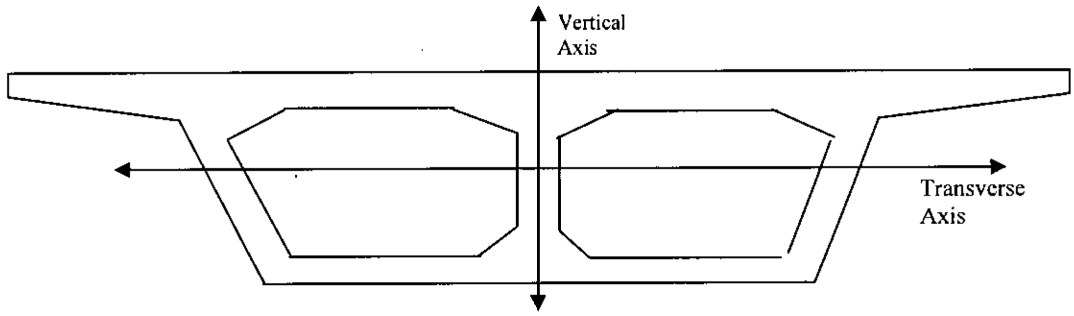


Fig 3.1 Typical Box Girder Section

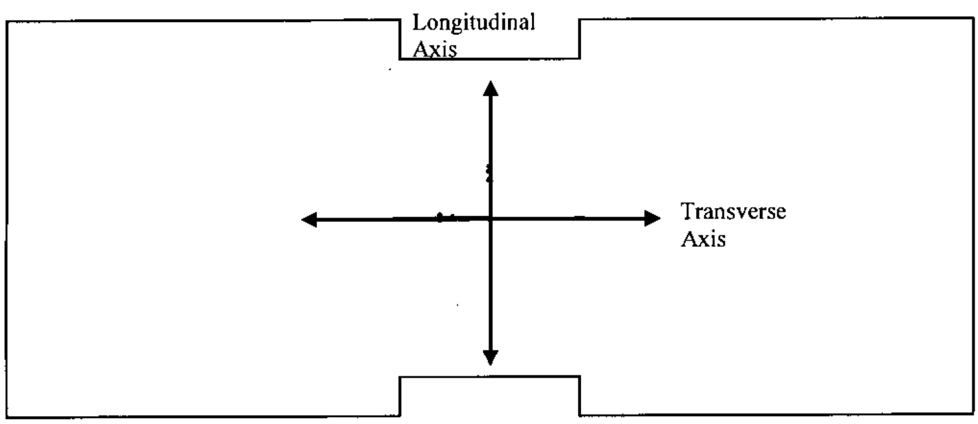


Fig 3.2 Typical Pier Section

The material properties of concrete is given in the Table 3.2,

Table 3.2 Concrete Properties

28 days compressive stress f'_c	27.58 MN/m^2
Weight per unit volume	23.56 KN/m^3
Modulus of Elasticity	24.82 GN/m^2
Poisson's Ratio	0.2
Coefficient of Thermal Expansion	9.9E-06 / ^0C

3.3.4 Bridge Modelling

For dynamic analysis of bridges it is common to use a simple model that would represent the basic dynamic characteristics of the bridge. In published literature (Gobarah and Ali, 1988; Hosada et al., 1991; Spyrokas, 1990; Shimada et al., 1991; Takeda et al., 1991) dynamic models comprising of beam-column elements, spring elements and lumped masses have been used.

During an earthquake a bridge is subjected to ground motion with components in two perpendicular directions. This study involves earthquake motion along transverse and longitudinal directions of the bridge.

A two span flyover bridge, as shown in Fig 3.3 is considered. In Fig 3.4 the bridge model is shown in line elements. The bridge is reinforced concrete box girder type that is continuous over three spans and is supported by three identical single column piers. The base of piers is assumed to be fixed. Bridge Superstructure is modeled as beam-column elements and lumped masses.

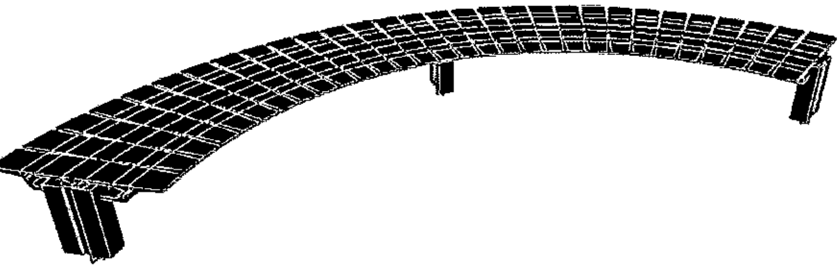


Fig 3.3 Typical Two Span Curved Flyover Bridge (Solid Model)

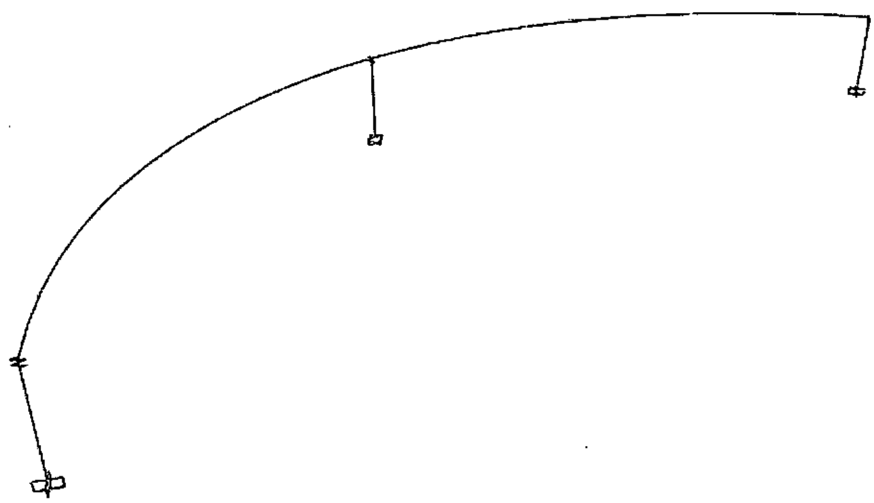


Fig 3.4 Model of Two Span Curved Flyover Bridge (Line Element)

3.3.5 Isolation System

Isolation device used in isolated bridges have bi-linear characteristics as shown in Fig 3.5. It has large stiffness of K_i up to yield point A. Then the stiffness decreases to K_h on exceeding the yielding displacement (point A). While unloading stiffness K_i is regained.

3.4 Seismic Loading

Dhaka City is located in zone 2 having a seismic zone coefficient (z) of 0.15. So, to account for the seismic responses of flyover bridges in Dhaka City, actual earthquake records are scaled to Peak Ground Acceleration (PGA) of 0.15g. Here four earthquake records are used. The first ground motion considered is El Centro Earthquake NS component of May, 1940 with peak acceleration of 0.35 g. This record has been used quite extensively in earthquake engineering studies. The list of earthquake motion used is shown in Table 3.5. Fig 3.6 to Fig 3.13 show time histories of orthogonal components of ground motion for the four earthquake records considered in the study

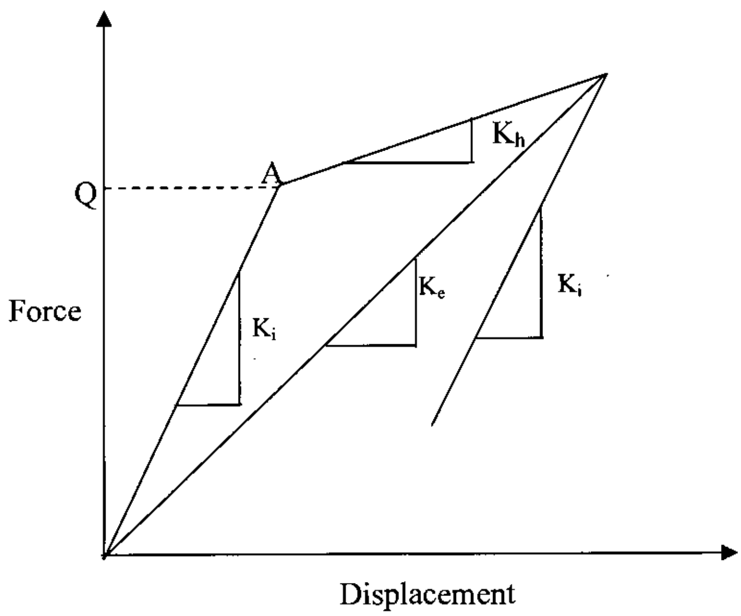


Fig 3.5 Isolator Force Displacement Relationship

Table 3.2 List of Earthquake Motion Used

Earthquake	Record Description	Magnitude (Richter)	Component	Peak Ground Acceleration
El Centro	Imperial Valley, May, 1940 Component NS	6.7	North South	0.35g
			East West	0.214g
Taft	Kern County, July, 1952	7.7	N21E	0.156g
			S69E	0.18g
Loma Prieta	Santa Cruz Mountains, October,1989	7.1	0^0	0.44g
			90^0	0.41g
Hachinohe	Tokachi-Oki Earthquake, Japan, May16, 1968	7.9	0^0	0.18g
			90^0	0.37g

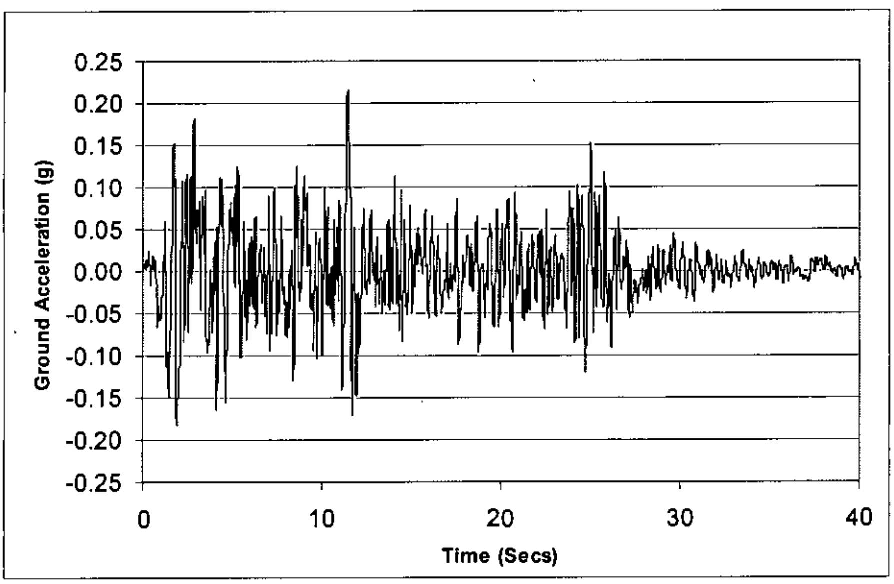


Fig 3.6 Time Histories of Ground Motion of El Centro Earthquake

(EW Component)

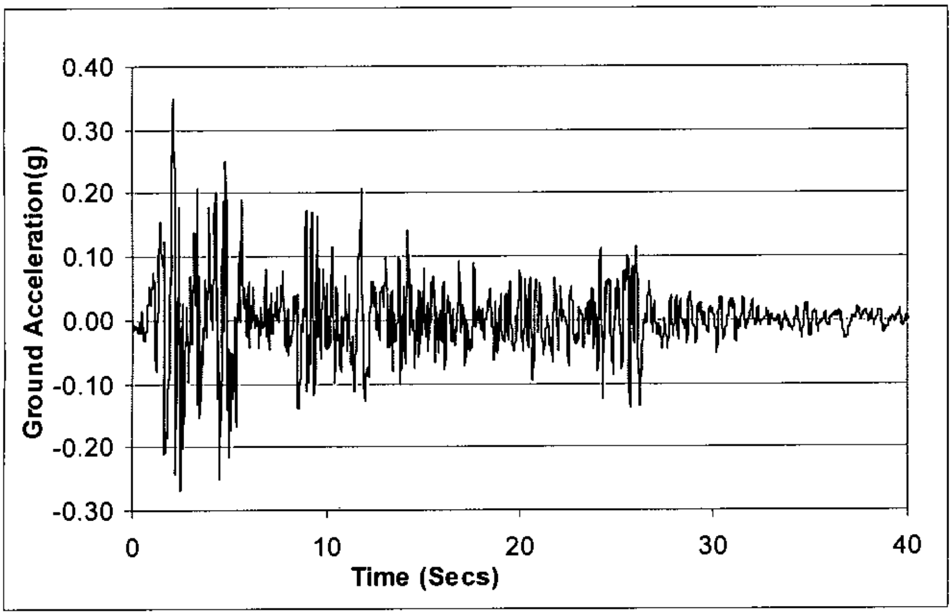


Fig 3.7 Time Histories of Ground Motion of El Centro Earthquake

(NS Component)

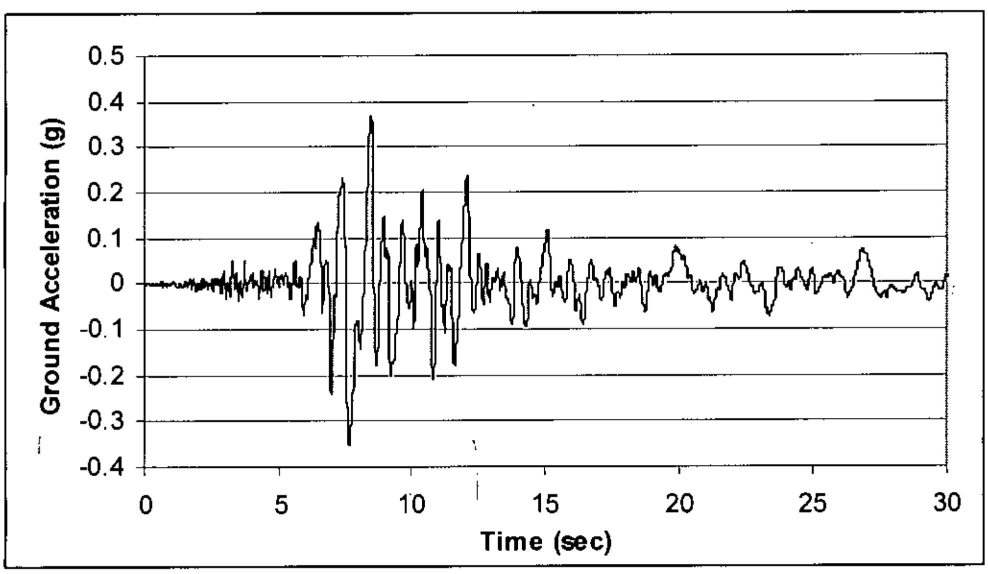


Fig 3.8 Time Histories of Ground Motion of Hachinohe Earthquake

(90° Component)

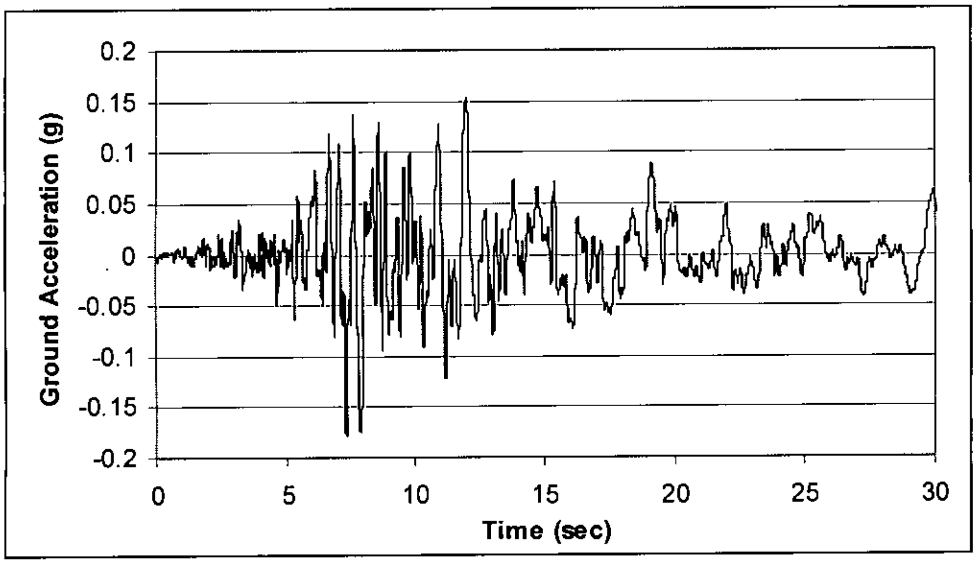


Fig 3.9 Time Histories of Ground Motion of Hachinohe Earthquake (0° Component)

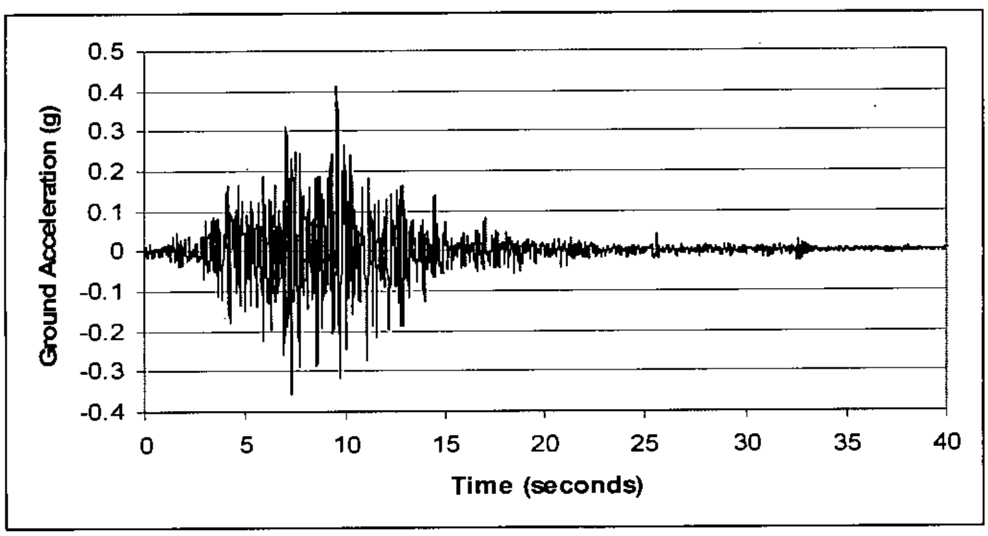


Fig 3.10 Time Histories of Ground Motion of Loma Prieta Earthquake (90° Component)

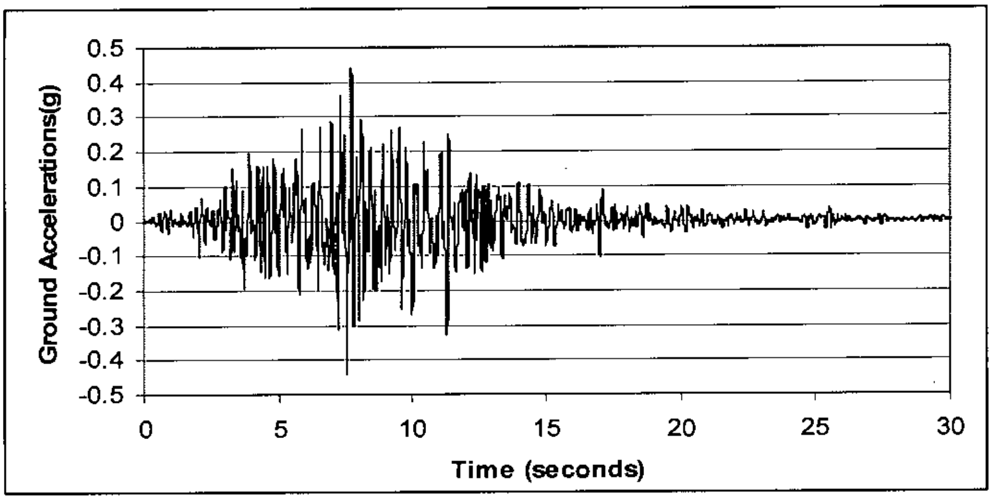


Fig 3.11 Time Histories of Ground Motion of Loma Prieta Earthquake (0° Component)

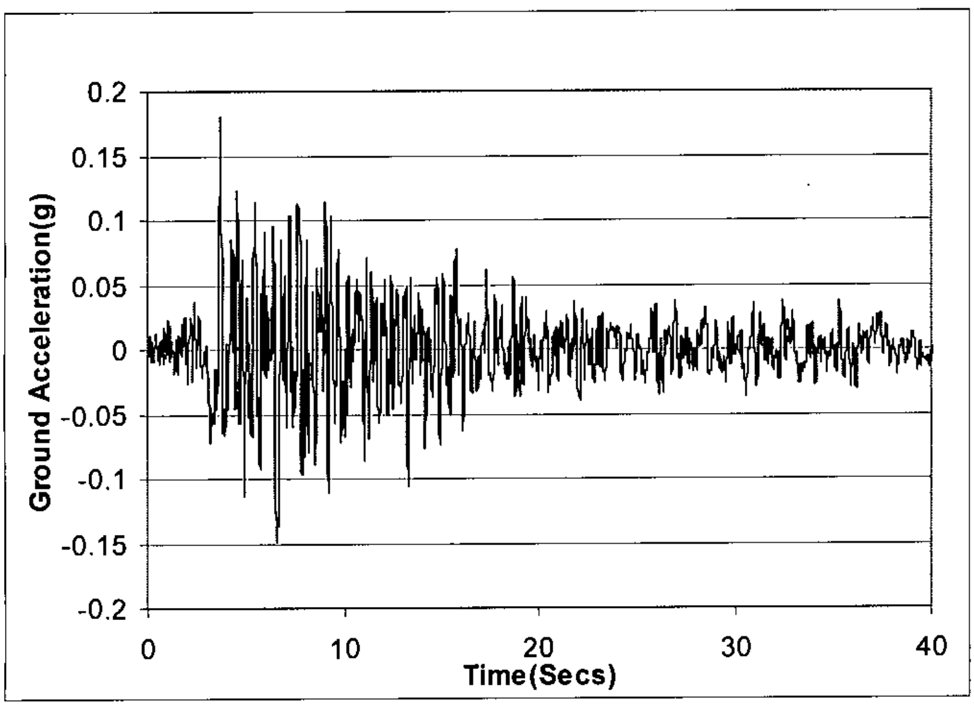


Fig 3.12 Time Histories of Ground Motion of Taft Earthquake (Component S69E)

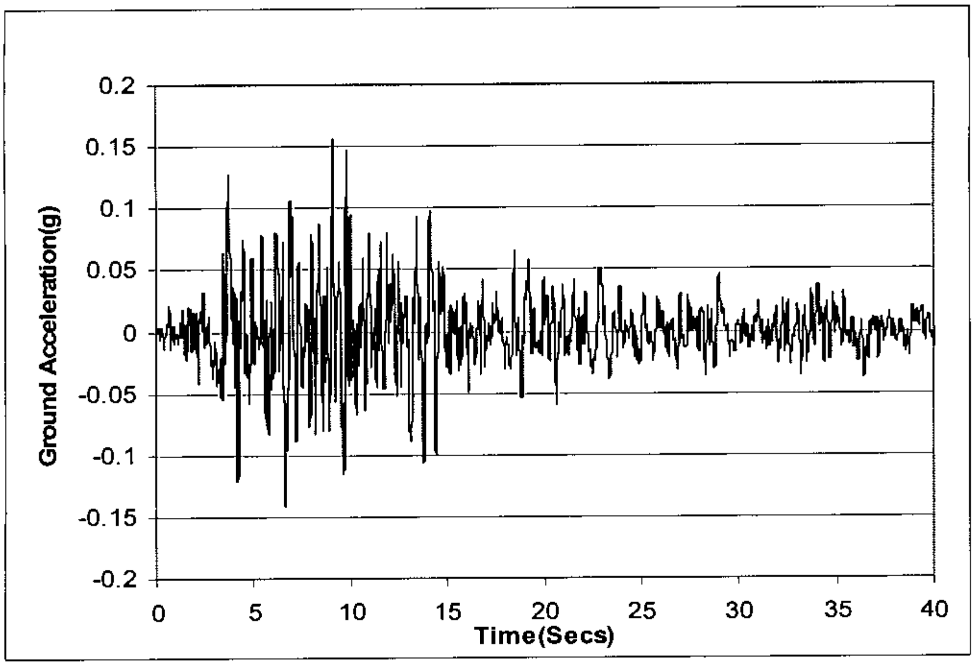


Fig 3.13 Time Histories of Ground Motion of Taft Earthquake (Component N21E)

3.5 Model Validation

To validate the bridge modeling technique and analysis procedure a reference model from published literature has been selected (Tsai, 2008). In that literature the effectiveness of the partially restrained seismic isolation is evaluated. Analytical expressions are derived to assess the effect of the partial restraint on the transverse dynamic characteristics of a seismically isolated regular bridge. A non-isolated, seismically isolated and a Partially Restrained Seismically Isolated (PRSI) bridge model are constructed for a highway bridge and nonlinear time history analyses of the bridge models under earthquake excitations are carried out to investigate their transverse seismic responses. In this study a similar bridge model is generated, time-history of a recognized earthquake has been selected and non-linear time history analysis has been performed for that bridge. The results of the analysis then compared with results in that published literature.

3.5.1 Reference Bridge Description

A bridge of total length of 103 m with continuous spans arranged as 28 m + 47 m + 28 m is considered. It has medium-rise column piers of 17 m in height. A pair of Lead Rubber Bearings (LRB) is installed at each pier top, and a pair of polytetrafluoroethylene (PTFE) coated natural rubber bearings at each abutment. A non-isolated model is constructed for the bridge using the SAP2000 commercial program. Pinned joints are used to simulate the fixed bearings at the pier tops and the abutments of the non-isolated bridge. Plastic hinges, which may occur under strong ground motions, are assigned to the pier bottoms for the non-isolated bridge. For the PRSI Bridge LRB is used. The elastic and post-yielding stiffness of the LRB are equal to 15 100 kN/m and 2323 kN/m, respectively. Its effective stiffness at the 11.6 cm design displacement is 4365 kN/m. The design characteristic strength is 236.4 kN. A 5% inherent damping ratio is assumed for the bridge. Fig 3.14 shows the box section and Fig 3.15 shows the pier section of the reference model.

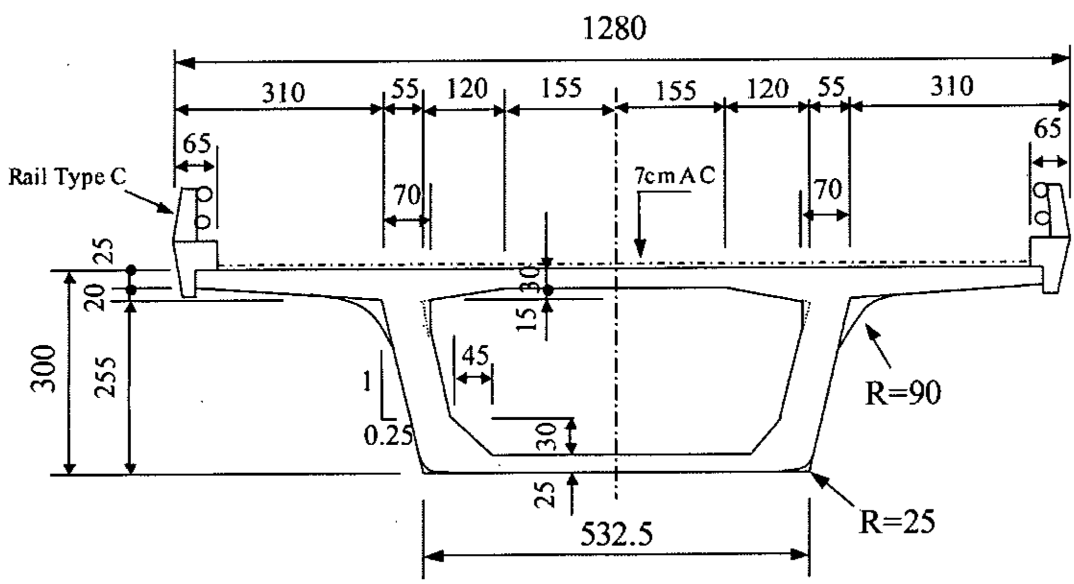


Fig 3.14 Box Section (All dimensions in centimeter) (After Tsai, 2008)

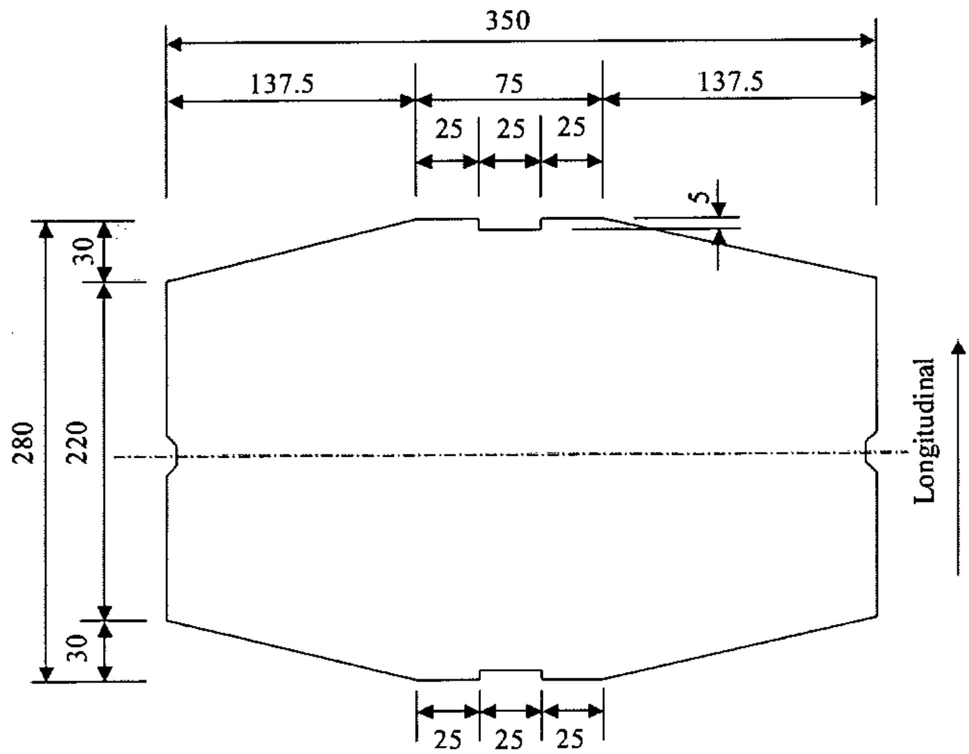


Fig 3.15 Pier Section (All dimensions in centimeter) (After Tsai, 2008)

Various ground motion records have been selected and scaled to six peak ground acceleration values ranging from 0.05g to 0.4g for the non-linear time-history analysis.

3.5.2 Bridge Model for Validation

In the Validation Bridge model, the bridge length and pier rise are same as of reference bridge model. The pier section and box-section is so defined that their sectional properties are almost the same as of the reference model. In Fig 3.16 box section used for the validation model is shown and in Table 3.6 the sectional properties of both reference model and validation model are presented. In non-isolated bridge, the bridge deck and pier connection is modeled by using very stiff rubber bearing. And in PRSI bridge model properties of LRB and other support conditions are provided in accordance with the reference. For the analysis of validation model, the Record-PUL104, (Event-1994/Northridge, Pacoima Dam) has been considered. This record is also scaled to five peak ground acceleration values ranging from 0.05g to 0.3g for the non-linear time-history analysis.

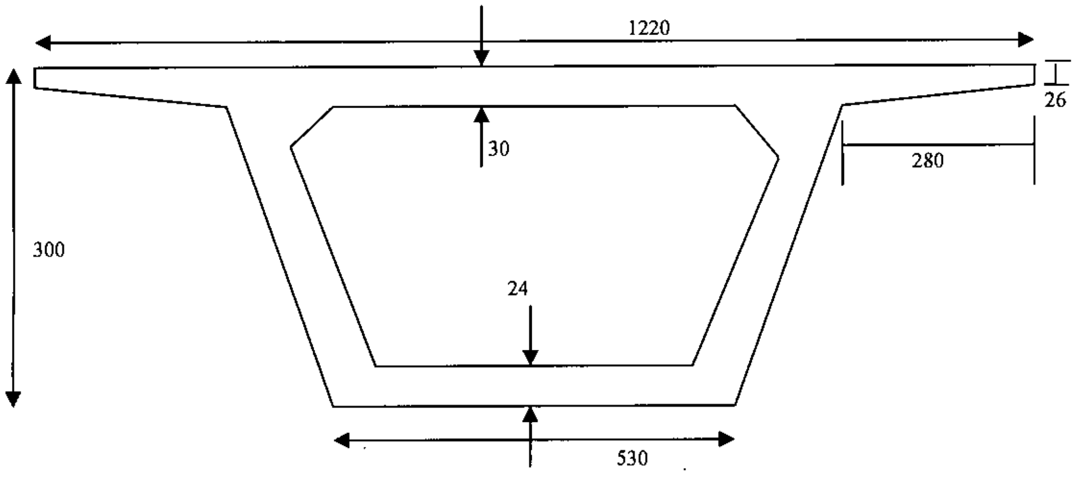


Fig 3.16: Validation Box Section (All Dimensions in cm)

Table 3.3 Sectional Properties of Reference and Validation Model.

		Reference Model (Tsai, 2008)	**Validation Model**
Bridge Pier	Cross Sectional Area (m^2)	7.68	8.96
	Moment of inertia w.r.t longitudinal axis (m^4)	8.93	8.49
	Moment of inertia w.r.t transverse axis (m^4)	4.55	5.01
Box Girder Section	Cross Sectional Area (m^2)	8.75	8.43
	Moment of inertia w.r.t transverse axis (m^4)	9.60	9.97
	Moment of inertia w.r.t longitudinal axis (m^4)	59.35	75.67

3.5.3 Comparison of Results

Time history analysis has been carried out for each type (non-isolated and PRSI) of bridges. After the analysis of non-isolated and PRSI bridges, the comparison of periods in both longitudinal and transverse direction with the reference value is shown in Table 3.7. In Fig 3.17 the comparison of the seismic column shear (V_{col}) normalized by the supported dead loads ($P_{Dl,pier}$) of the pier is presented for non-isolated bridge. And in Fig 3.18 the bearing deformation Δ_b of both reference and validation model for the specific soft site condition and near ground motion are presented.

Table: 3.4 Comparison of Periods in Both Longitudinal and Transverse Direction.

	Non-isolated Bridge		PRSI Bridge	
Direction	**Reference Model (Tsai, 2008)**	**Validation Model**	**Reference Model (Tsai, 2008)**	**Validation Model**
Longitudinal	0.79	0.81	2.01	2.29
Transverse	0.46	0.49	0.69	0.71

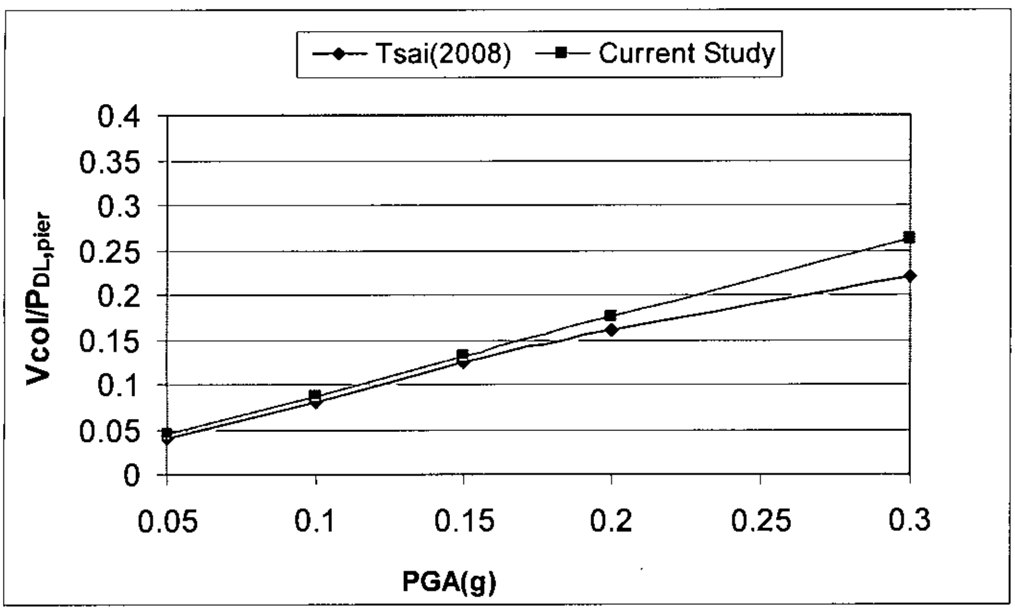


Fig 3.17 Normalized Seismic Column Shear for Non-Isolated Bridge

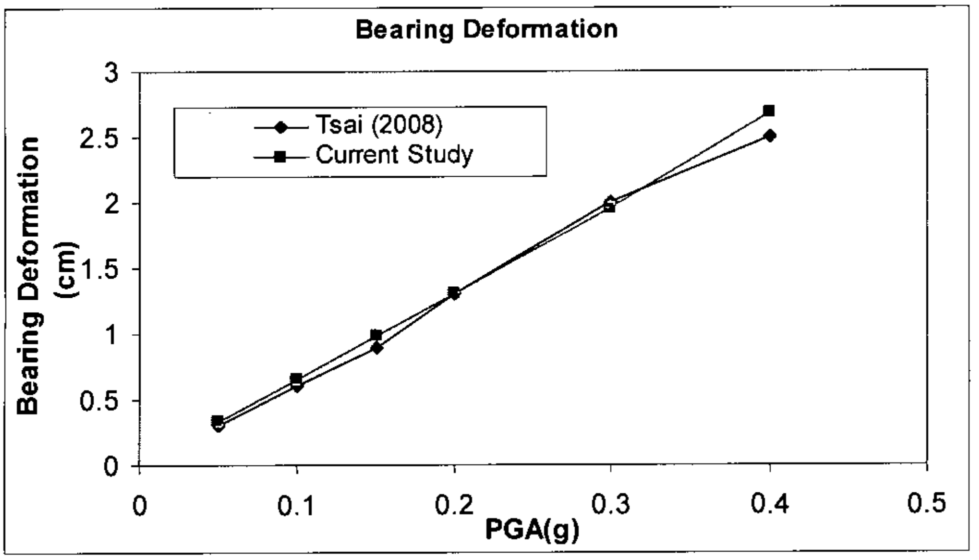


Fig 3.18 Comparison of Peak Bearing Deformation in PRSI (isolated) Bridge for different PGA

From the comparison of results, it is observed that the validation models (both non-isolated and isolated) agree with published results of Tsai (2008). Hence, it is expected the adopted modeling technique and analysis procedure may be extended for use in the parametric studies of other bridges of similar nature.

Chapter 4

PARAMETRIC STUDY ON RESPONSE OF NON-ISOLATED FLYOVER BRIDGES UNDER SEISMIC LOADING

4.1 Introduction

The response of bridges under seismic loading is a complex phenomenon. It depends on many factors of bridge as well as ground motion characteristics. Bridge factors include stiffness, geometry, damping characteristics, yielding properties and many more. Ground motion characteristics involve peak ground acceleration, frequency content and duration of shaking. Also there are other elements in the bridge whose non-linear behavior influences the overall bridge response. Previous studies have already identified the effects of many variables on the response of bridges. In this study some other variables will be investigated.

In this chapter the present problem will be discussed briefly and key bridge responses will be identified. The results of parametric study will be presented and discussed elaborately.

4.2 Statement of the Problem

For the purpose of parametric study simple dynamic models of bridge have been used. The bridges considered for study are of concrete box type, continuous over two spans resting on three identical piers. Each span length is 50 m and pier height is 8 m. For straight bridge span length is taken from one pier to another pier and for curved bridges the arc length along the bridge deck from one pier to another is considered as span length. The major parameter considered in this study is bridge curvature keeping the same bridge length, seven different curvatures of bridge are considered, 0^0, 30^0, 60^0, 90^0, 120^0, 150^0 and 180^0 as shown in Fig 4.1. The straight bridge elevation is shown in Fig 4.2.

Three different pier sections Pier 1, Pier 2 and Pier 3 are considered in the analysis. The box-girder section and pier sections are shown in Fig 4.3 to Fig 4.6. The cross sectional properties of box section and pier section are presented in Table 4.1 and 4.2. Only one pier section (Pier 1) is used to evaluate the effect of curvature and others are used to measure the effect of pier stiffness on bridge response. The piers are considered to be fixed at its base, soil flexibility is ignored.

Four different pairs of earthquake excitations have been selected. The peak acceleration value of each earthquake is scaled to 0.15g. Each components of a pair of earthquake excitation is applied from the two orthogonal directions simultaneously. Bridge vibration in both longitudinal and transverse modes is considered as their properties in both directions aren't the same. Only non-isolated bridges are analyzed in this chapter. The key responses considered in this study are pier top displacement and acceleration in both longitudinal and transverse direction, mid span acceleration and displacement in both longitudinal and transverse direction, pier forces (shear and moments) and deck forces (moments).The moment at bottom of pier is considered as pier moment. Among the results, 'transverse shear' represents the shear along transverse axis; 'longitudinal shear' represents the shear along longitudinal axis, 'deck torsion' represents the moment about longitudinal axis of deck. The axes of deck and pier are shown in Fig 4.3 and Fig 4.4 respectively.

Only the peak values of responses due to time history analysis are considered here. In the related figures, 'El Centro', 'Taft', 'Loma P' and 'Hachinohe' stand for the response due to El Centro Earthquake records, Taft Earthquake records, Loma Prieta Earthquake records and Hachinohe Earthquake records respectively.

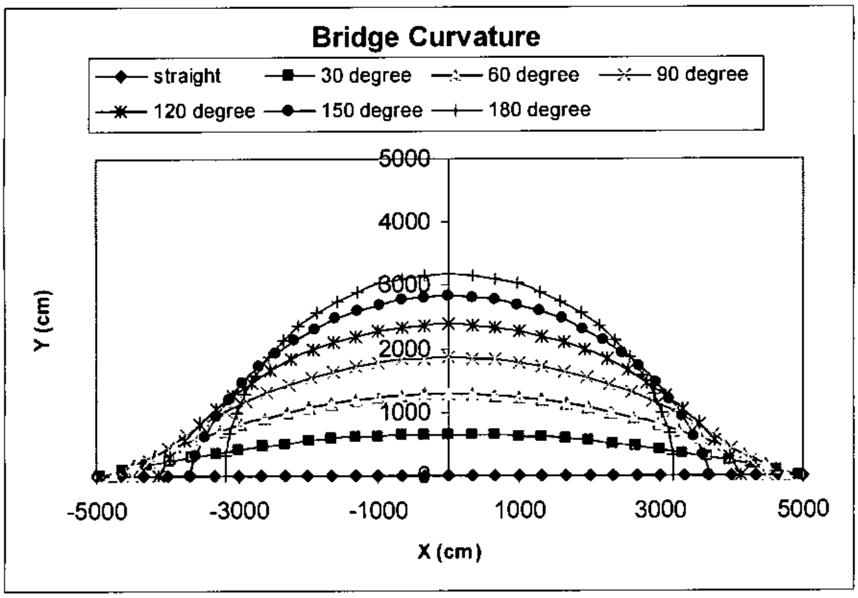


Fig 4.1 Bridge Curvature (Schematic Plan View)

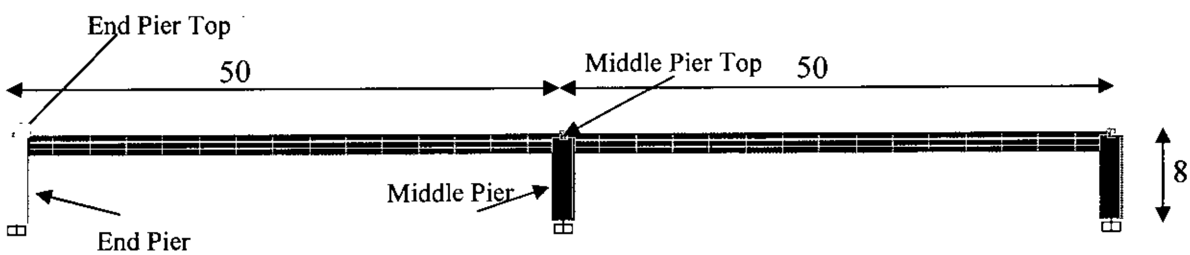


Fig 4.2 Bridge Elevation (Straight Bridge) (All dimension in meter)

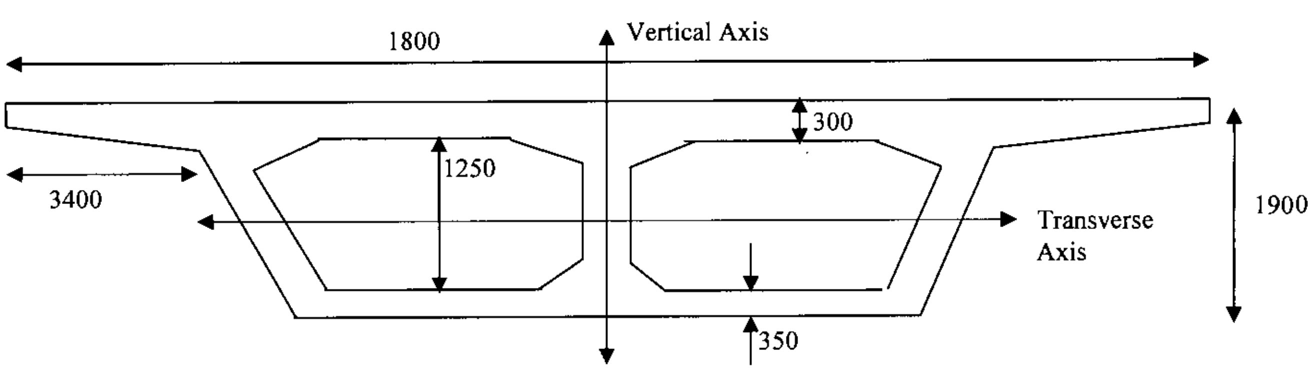


Fig 4.3 Box Girder Section (All dimensions in mm)

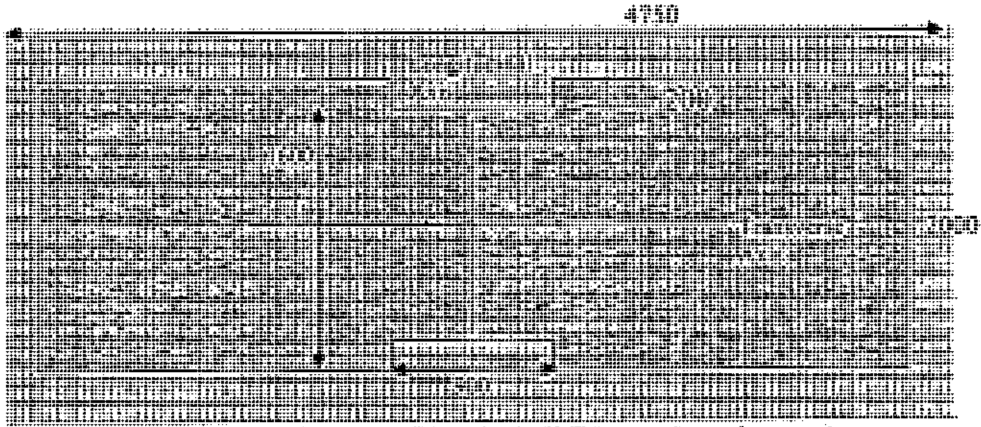


Fig 4.4 Pier Section (Pier 1) (All Dimensions in mm)

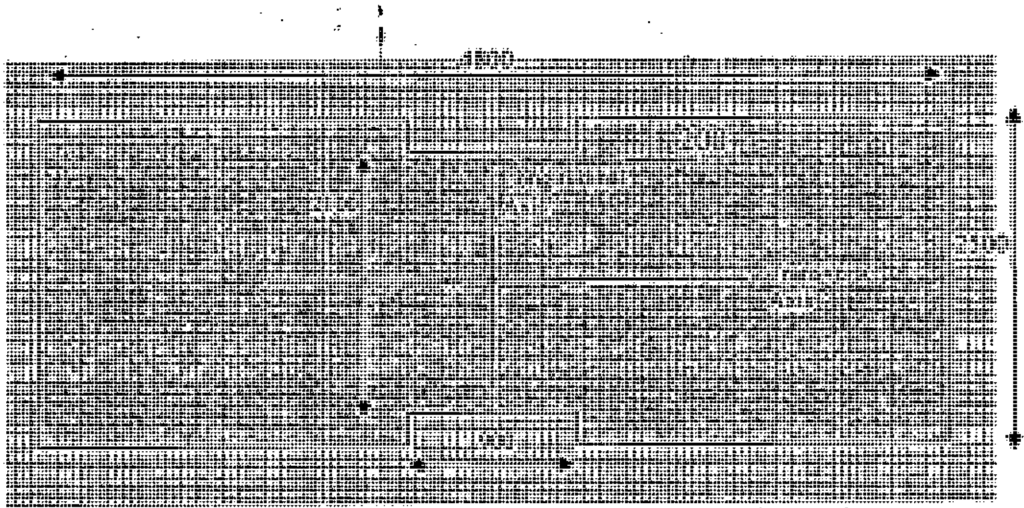


Fig 4.5 Pier section (Pier 2) (All Dimensions in mm)

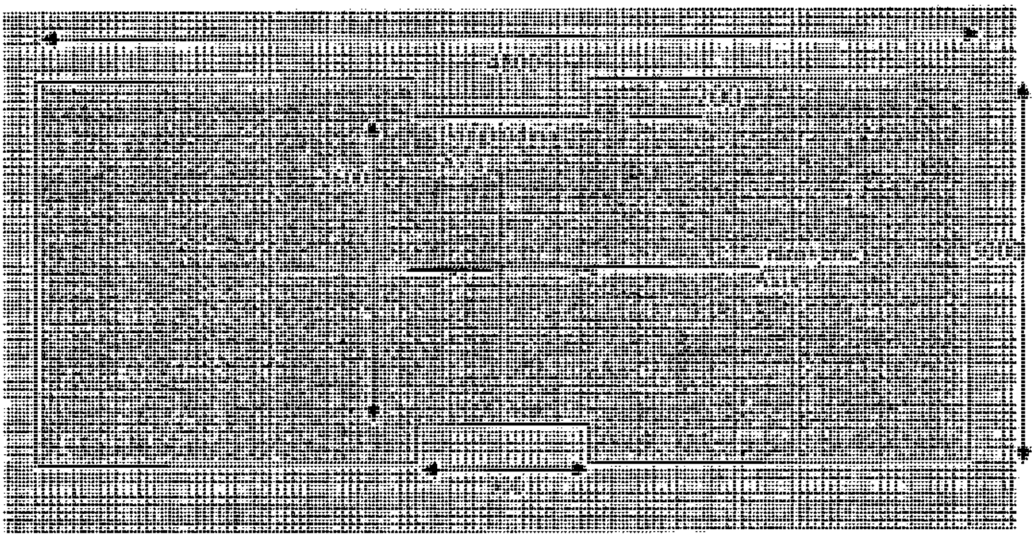


Fig 4.6 Pier Section (Pier 3) (All Dimensions in mm)

Table 4.1 Box Section Properties

Sectional Properties	
Area, m^2	12.7
Moment of Inertia about transverse axis, m^4	6.04
Moment of Inertia about vertical axis, m^4	254

Table 4.2 Pier Section Properties

Sectional Properties	Pier Section		
	Pier 1	Pier 2	Pier 3
Area, A (m^2)	9.538	12.01	14.04
Moment of Inertia about transverse axis, (m^4)	3.09	6.15	10.09
Moment of Inertia about longitudinal axis (m^4)	19.61	24.52	27.62

4.3 Effect of Curvature on Transverse Response

The analytical results of the effect of curvature on transverse responses are presented below. Respective response is normalized with respect to response for straight bridge.

4.3.1 Pier Top Displacement and Acceleration

In Fig 4.7 and Fig 4.8 the effect of curvature on end pier top transverse displacement and acceleration are shown. Displacement of the end pier top is found to increase gradually with increase in curvature for Loma Prieta and Taft Earthquake. For the El Centro and Hachinohe earthquake, this displacement is found to be decreasing with increased curvature except for 60^0 curvature for which it reaches its peak value and then decrease gradually with increase in curvature. Transverse acceleration of end pier top is found to decrease for El Centro, Loma Prieta ad Hachinohe earthquake. However for Hachinohe case, the pier top acceleration is greater than straight bridge for 60^0 and 90^0 curved bridges.

Effect of curvature on transverse displacement and acceleration of middle pier top are similar in nature (Fig 4.9 and Fig 4.10). In general both transverse displacement and acceleration has downward trend beyond 60^0 curvature. Initially, displacement and acceleration on the middle pier top has shown slight lower value at 30^0 curvature compared to straight ones. It reaches its peak values at 60^0 and then gradually decreases with increase in curvature. The time variation of end pier top acceleration for 0^0 and 180^0 curvature is shown in Fig 4.11.

4.3.2 Deck Midspan Displacement and Acceleration

Deck midspan responses with curvature are shown in Fig 4.12 and 4.13. They show that, for El Centro and Hachinohe earthquake values decrease from 60 degree curvature.

Displacement decrease from 129% at 60 degree to 79% at 180 degree (El Centro) and acceleration from 113 % at 60 degree to 61% at 180 degree (Hachinohe).

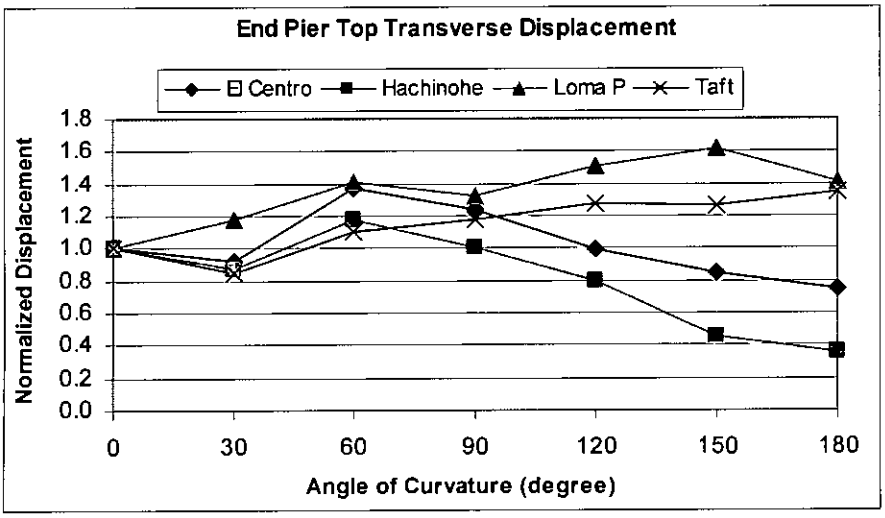


Fig 4.7 Effect of Curvature on End Pier Top Transverse Displacement

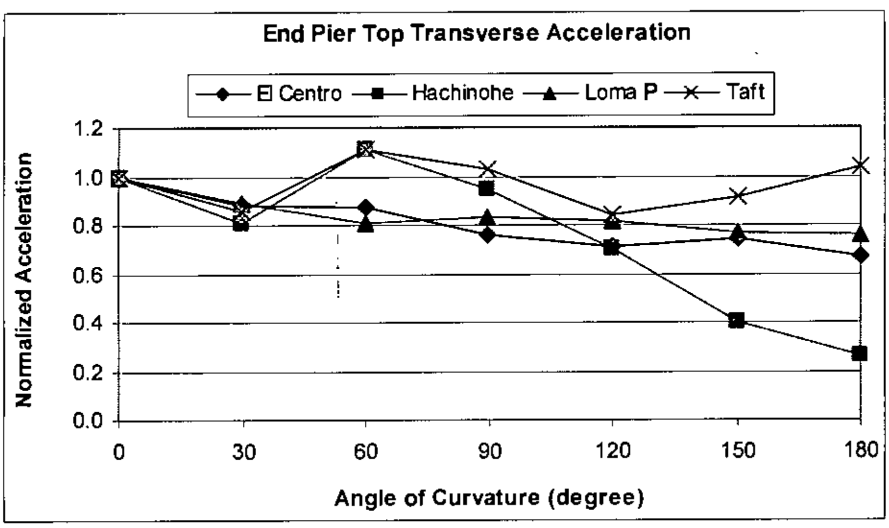


Fig 4.8 Effect of Curvature on End Pier Top Transverse Acceleration

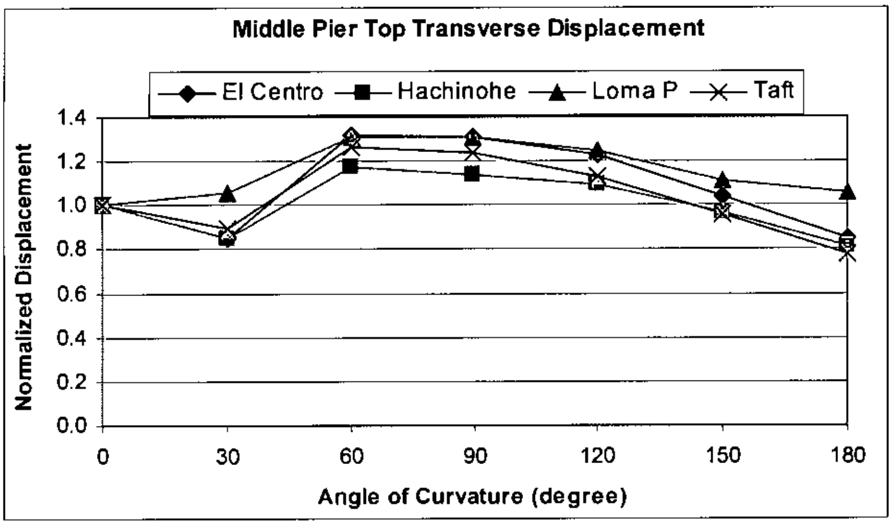


Fig 4.9 Effect of Curvature on Middle Pier Top Transverse Displacement

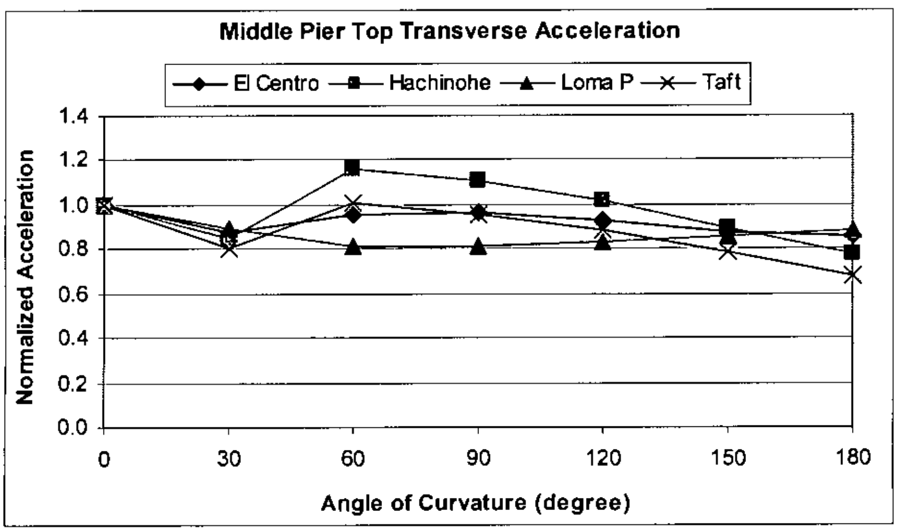


Fig 4.10 Effect of Curvature on Middle Pier Top Transverse Acceleration

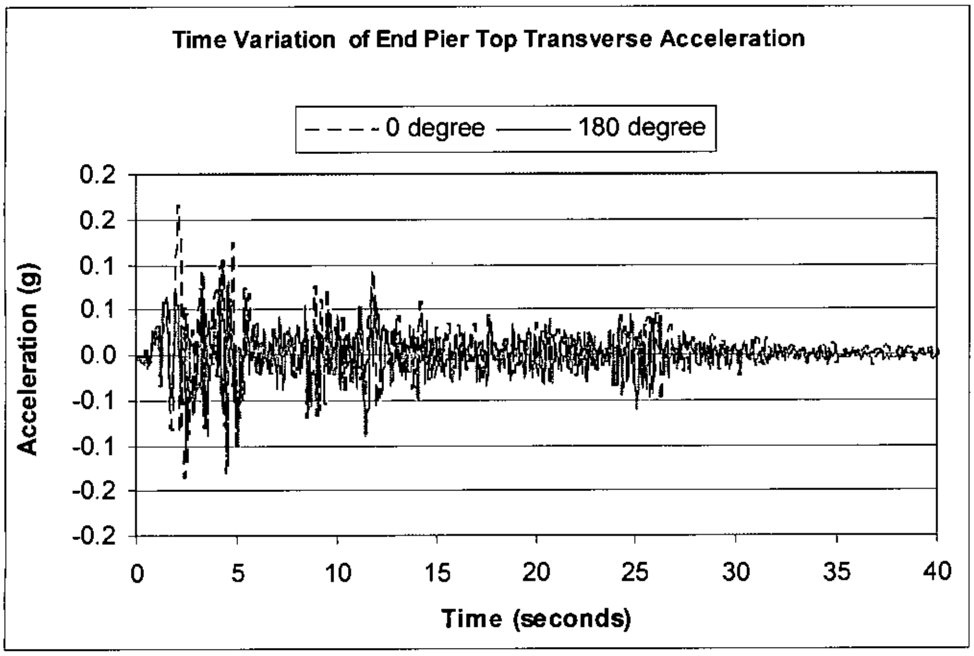


Fig 4.11 Time Variation of End Pier Top Acceleration for 0^0 And 180^0 Curvature (El Centro Earthquake)

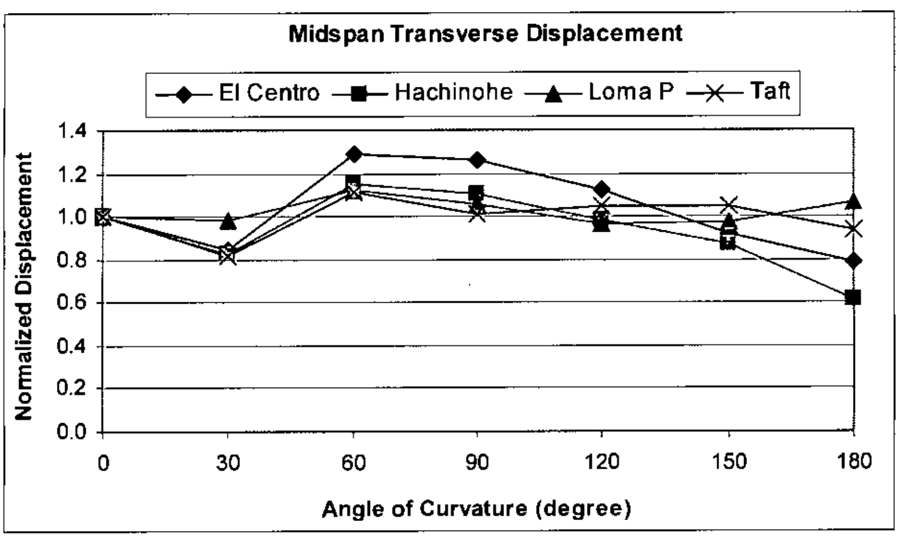


Fig 4.12 Effect of Curvature on Midspan Transverse Displacement

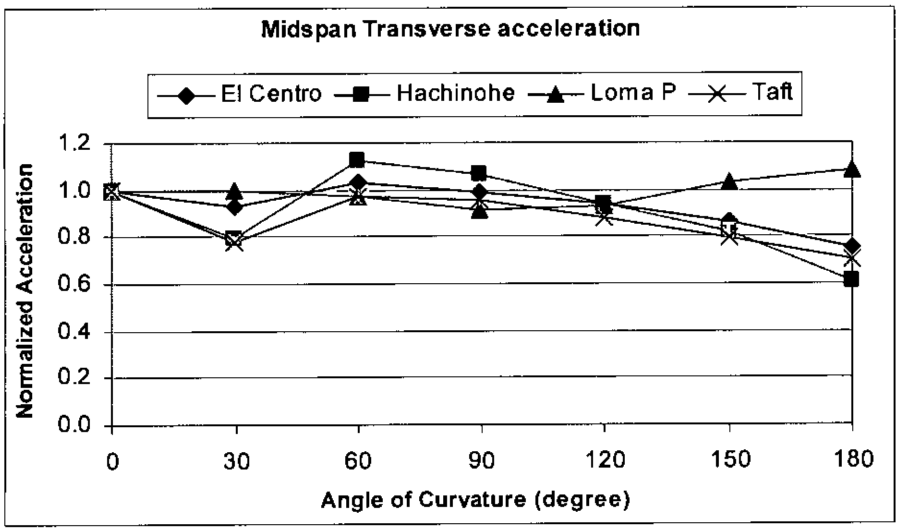


Fig 4.13 Effect of Curvature on Midspan Transverse Acceleration

4.3.3 Pier Forces

From the analysis results, as shown in Fig 14 to Fig 17, following observations have been made about the effect of curvature on pier force. Middle pier transverse shear force and moment about longitudinal axis decrease at 30^0 curvature and then reaches its peak at 60^0 curvature. It is then gradually decrease with increase in curvature. For end pier, transverse shear and moment about longitudinal axis increase at 60^0 and then decrease gradually with further increase in curvature for El Centro earthquake. But in case of Loma Prieta and Taft earthquake, moment about longitudinal axis increases with increase in curvature.

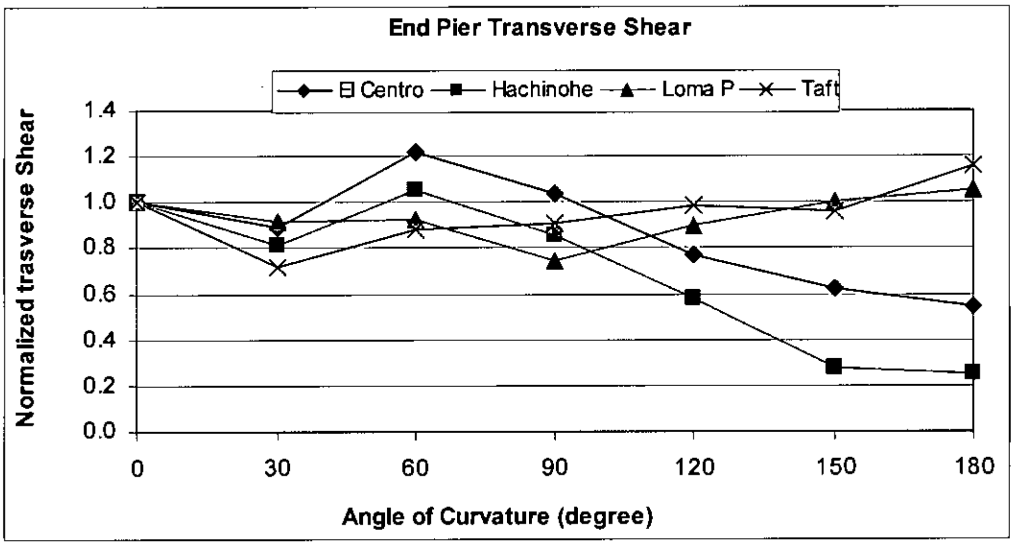


Fig 4.14 Effect of Curvature on End Pier Transverse Shear

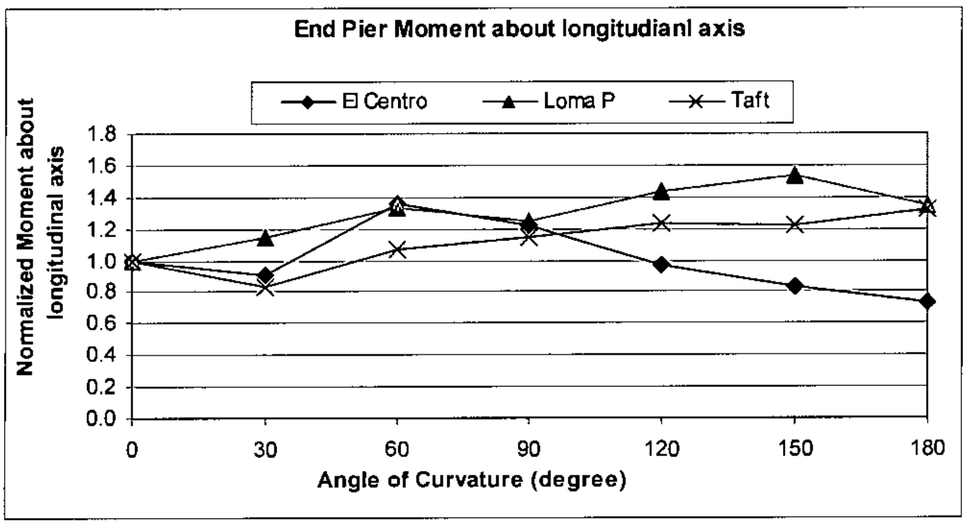


Fig 4.15 Effect of Curvature on End Pier Moment about Longitudinal Axis

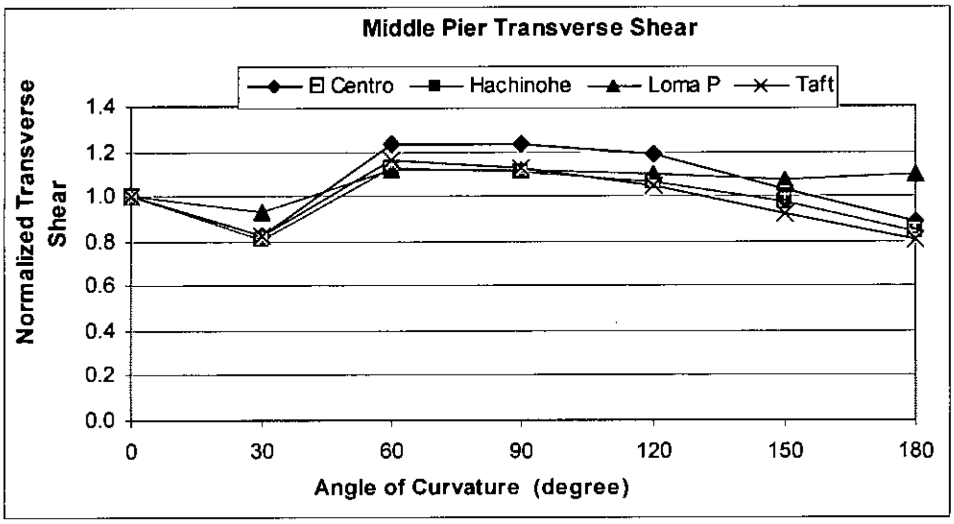


Fig 4.16 Effect of Curvature on Middle Pier Transverse Shear

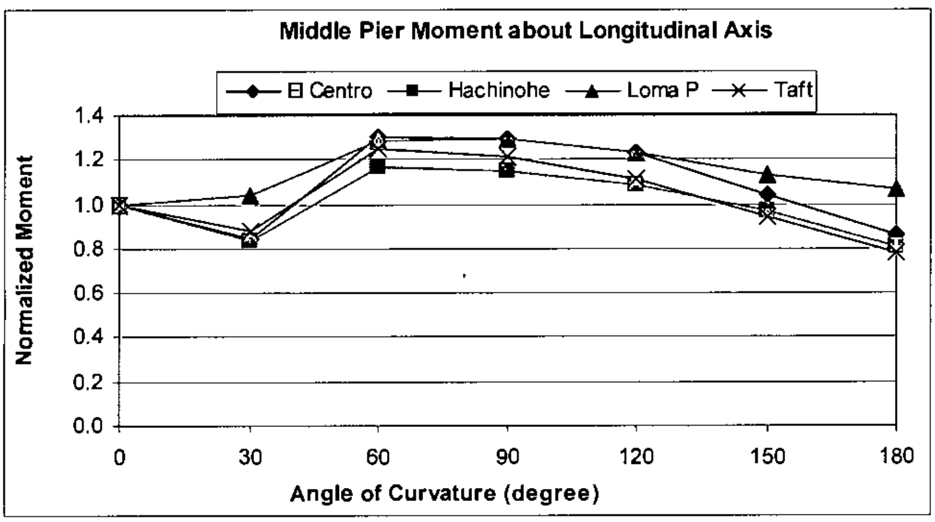


Fig 4.17 Effect of Curvature on Middle Pier Moment about Longitudinal Axis

4.4 Effect of Curvature on Longitudinal Response

4.4.1 Pier Top Displacement and Acceleration

From Fig 4.18 to 4.21, it is shown that end pier and middle pier top longitudinal displacement increase with increase in curvature. End pier top displacement increase almost 200% for Hachinohe, Loma Prieta and Hachinohe earthquake. Due to curvature, end pier acceleration decreases to its lowest value at 300 curvature then gradually increase with increase in curvature for El Centro, Taft and Hachinohe earthquake, except for Loma Prieta earthquake where it decrease with increase in curvature.

The time variation of end pier top acceleration for 0^0 and 180^0 curvature is shown in Fig 4.18.

4.4.2 Deck Midspan Displacement and Acceleration

Fig 4.19 and 4.20 show that midspan displacement increases with curvature for all earthquake loading. Displacement increases to 180% and acceleration increase to 147% at 180 degree curvature for Hachinohe earthquake. But for acceleration, it increases only for Taft and Hachinohe earthquake, while for other two earthquake no effect due to curvature noticed.

With some exceptions, longitudinal acceleration of curved bridge pier is also greater than straight bridge

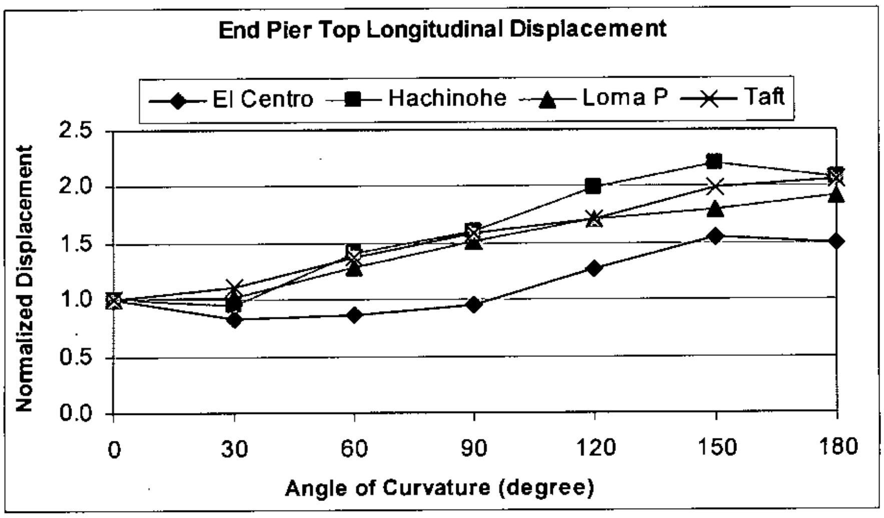


Fig 4.18 Effect of Curvature on End Pier Top Longitudinal Displacement

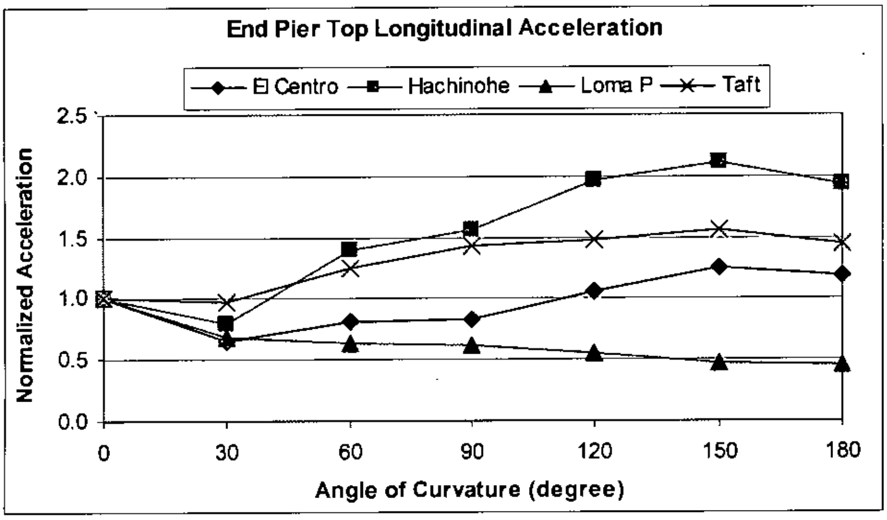


Fig 4.19 Effect of Curvature on End Pier Top Longitudinal Acceleration

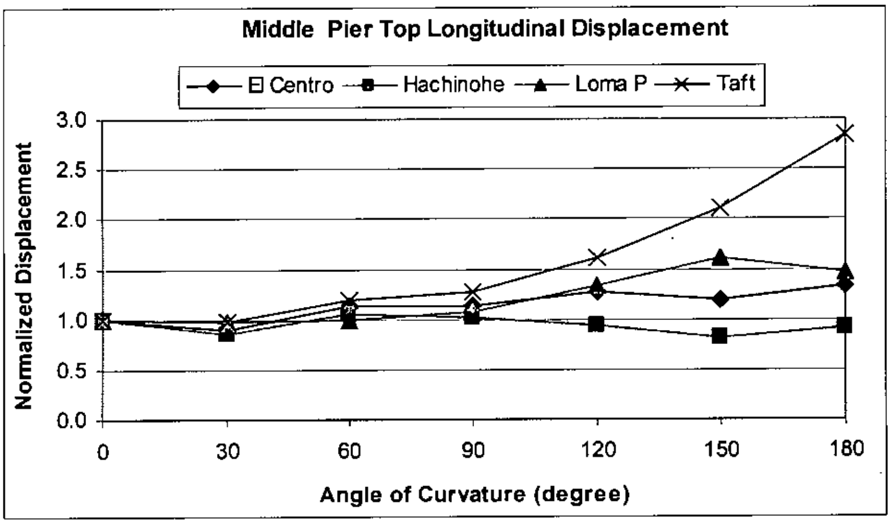


Fig 4.20 Effect of Curvature on Middle Pier Top Longitudinal Displacement

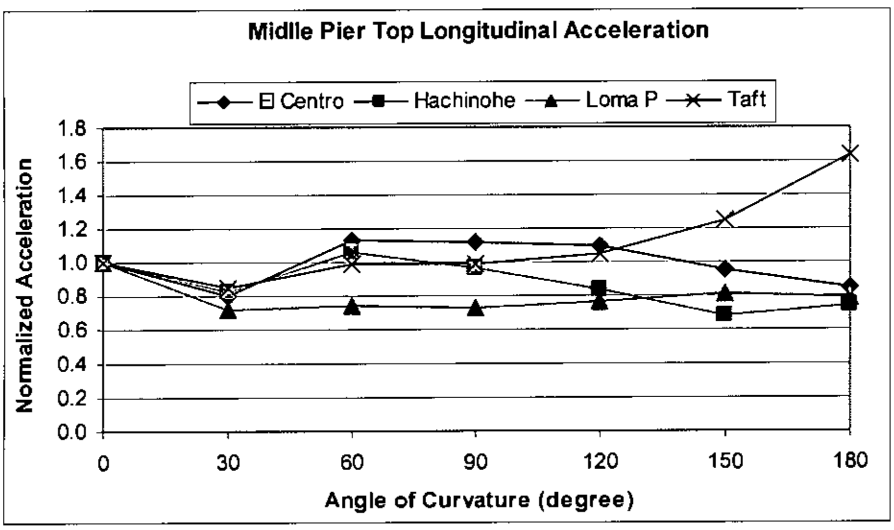


Fig 4.21 Effect of Curvature on Middle Pier Top Longitudinal Acceleration

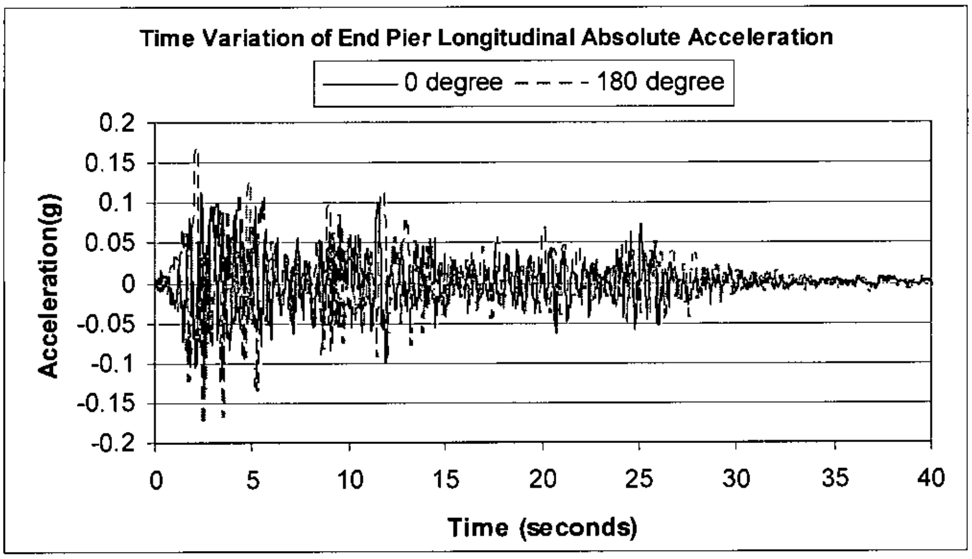


Fig 4.22 Time Variation of End Pier Longitudinal Absolute Acceleration (El Centro Earthquake) (Non-isolated)

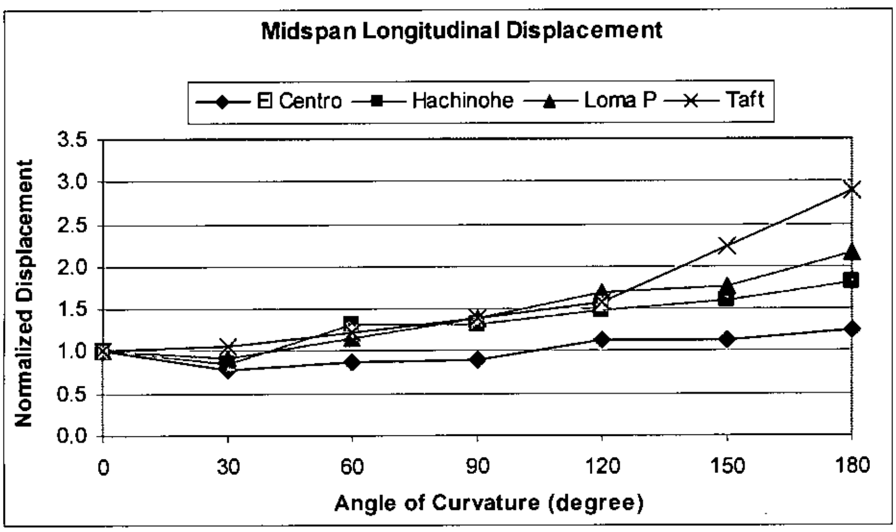


Fig 4.23 Effect of Curvature on Midspan Longitudinal Displacement

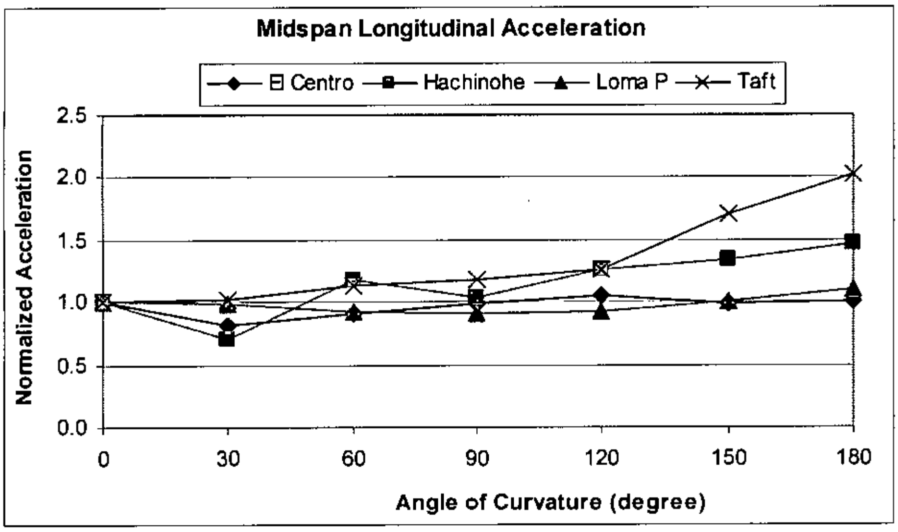


Fig 4.24 Effect of Curvature on Midspan Longitudinal Acceleration

4.4.3 Pier Forces

End pier longitudinal shear and end pier moment about transverse axis increase for all earthquake loads with increase in curvature. Moments increased almost 100% for Loma Prieta, Taft and Hachinohe earthquake. Middle pier longitudinal shear and moent about transverse axis also increase with increase in curvature. Moments increased by 50% at 1800 curvature for Hachinohe and Loma Prieta earthquake. All relevant results are shown in Fig 4.25 to Fig 4.28.

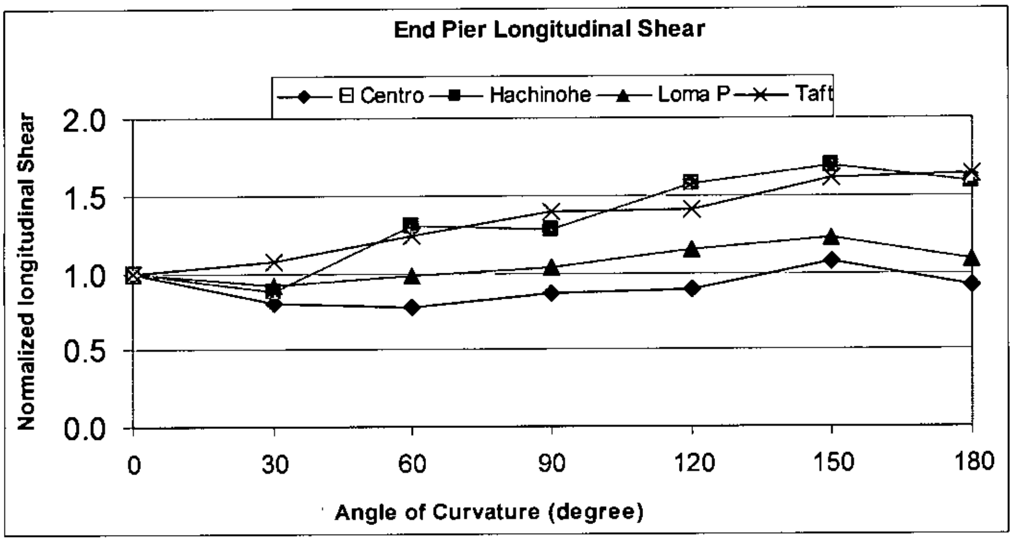


Fig 4.25 Effect of Curvature on End Pier Longitudinal Shear

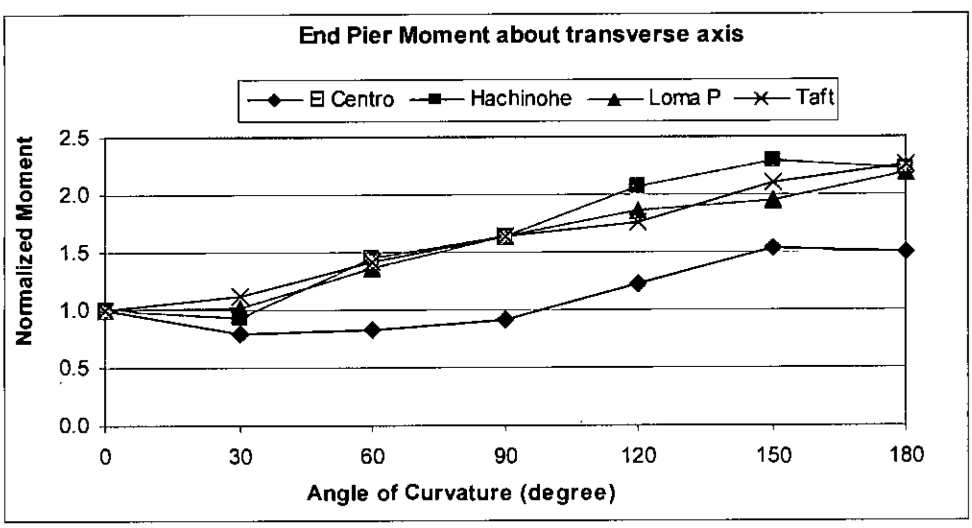


Fig 4.26 Effect of Curvature on End Pier Moment about Transverse Axis

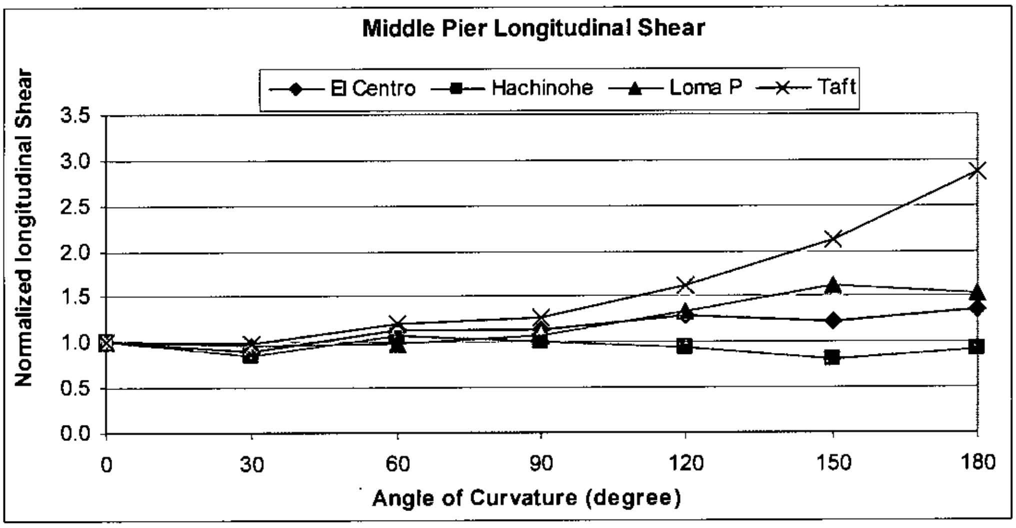


Fig 4.27 Effect of Curvature on Middle Pier Longitudinal Shear

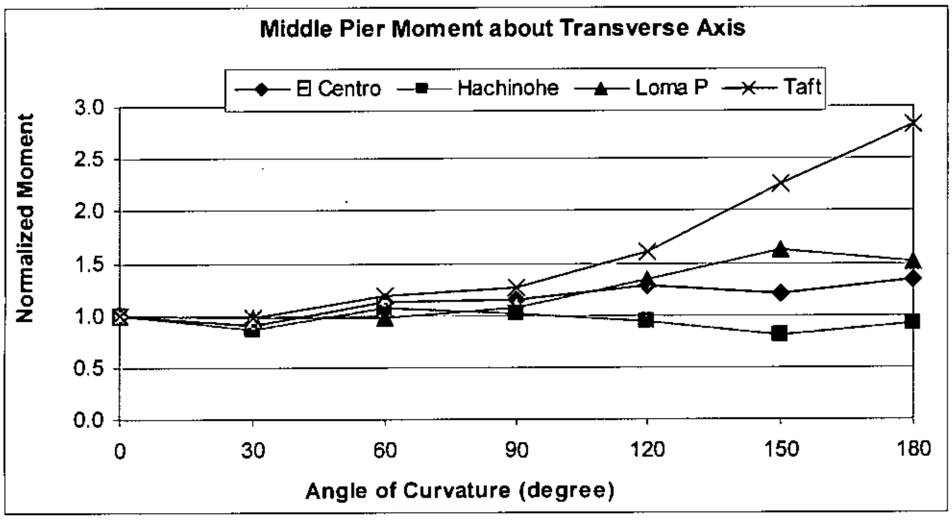


Fig 4.28 Effect of Curvature on Middle Pier Moment about Transverse Axis

4.5 Effect of Curvature on Deck (Mid Span) Moments

The deck moments considered here is the torsion, moments about deck transverse axis and about deck vertical axis. In Fig 4.29 to 4.31 the effect of curvature on moments of bridge midspan are shown. These moments are of midspan only and may not be the maximum moments in whole span.

From results it is seen that deck midspan torsion and deck moment about transverse axis increase with curvature. Torsion increases very significantly with the increase of curvature. This increased deck torsion causes torsional shear stress in addition to other stresses within the deck section. Hence, for curved bridge the section should be adequately designed to minimize the effect additional torsional moment developed due to curvature. For El Centro and Hachinohe, torsion increases almost 1100% of the value for 30 degree curvature. Also moment about transverse axis increase about 20% from the value of straight bridge. But moment about vertical axis decreases with curvature. At 180 degree curvature, moment is about 40% of the value of straight bridge.

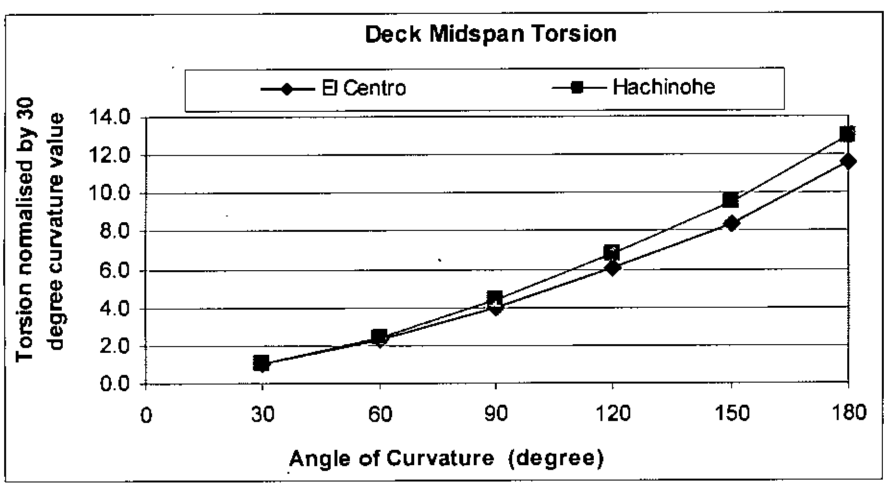


Fig 4.29 Effect of Curvature on Deck Midspan Torsion

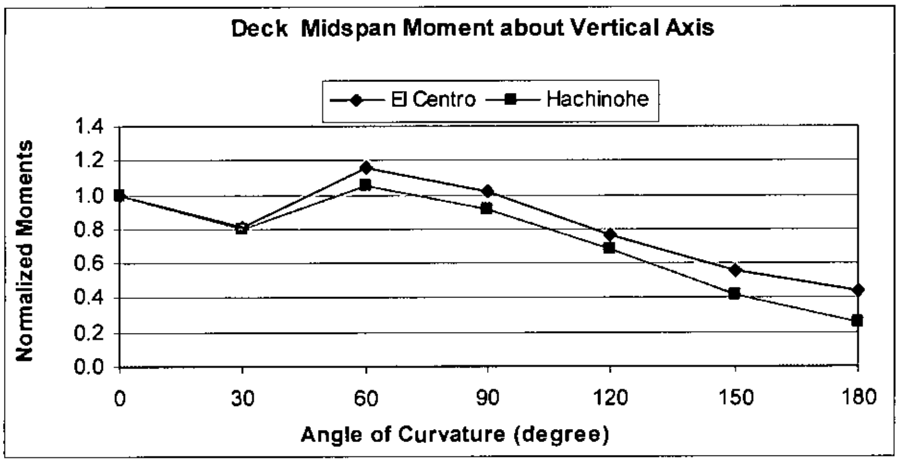


Fig 4.30 Effect of Curvature on Midspan Moment about Vertical Axis

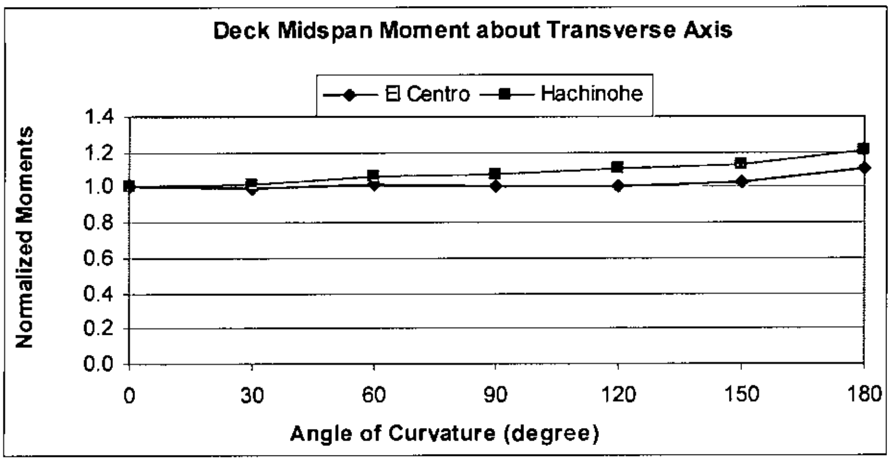


Fig 4.31 Effect of Curvature on Midspan Moment about Transverse Axis

4.6 Effect of Pier Stiffness on Bridge Response

In this parametric study another parameter considered is pier stiffness. For this purpose three different pier sections Pier1, Pier 2 and Pier 3 has been used (keeping the total amount and arrangement of steel reinforcement same in all piers). Pier 1 is the least stiff and Pier 3 is the most. The sectional properties of pier sections are shown in Table 4.2

4.6.1 Displacements

Pier stiffness has very prominent effect on pier top displacement. As the stiffness increased, the displacement in both longitudinal and transverse direction decreased. Pier 1 has I = 19.61 m4, and Pier 2 has I = 24.52m^4. Due to 25% increase in stiffness, the longitudinal displacement decrease almost 35%.

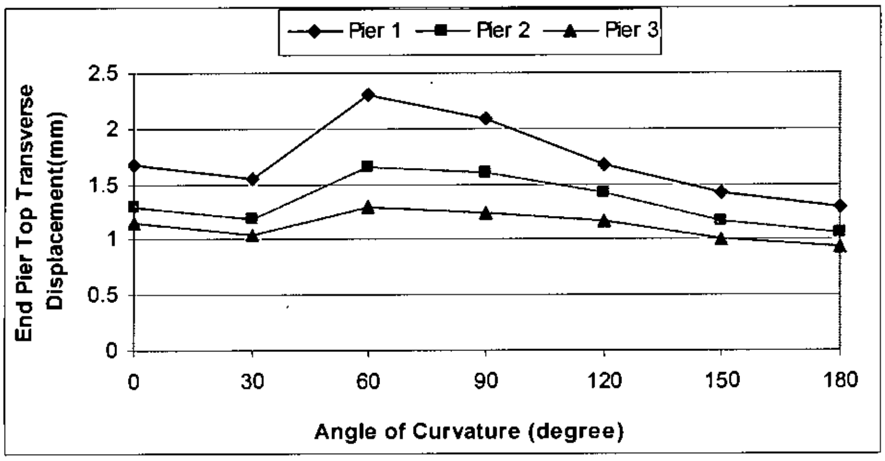


Fig 4.32 Effect of Pier Stiffness on End Pier Top Transverse Displacement (El Centro Earthquake) (Non-isolated Bridge)

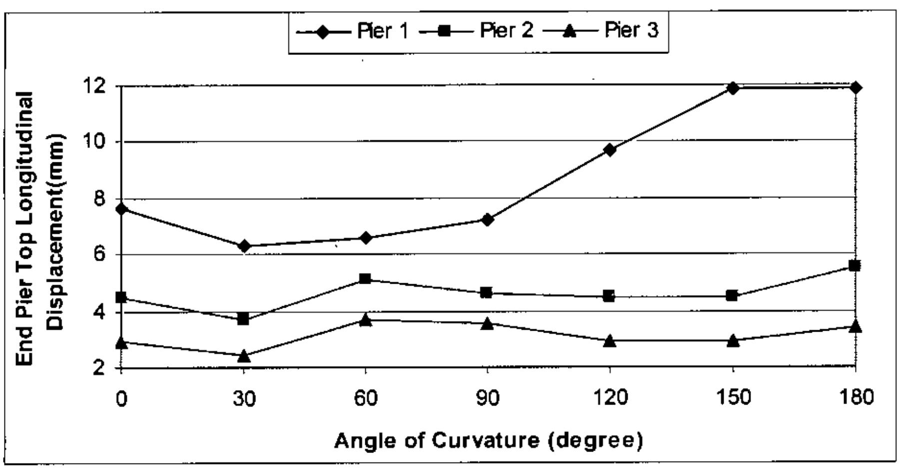


Fig 4.33 Effect of Pier Stiffness on End Pier Top Longitudinal Displacement (El Centro Earthquake) (Non-isolated Bridge)

4.6.2 Accelerations

The Fig 4.34 to 4.36 shows the effect of pier stiffness on accelerations of pier top- both end pier and middle pier. From these results it is seen that middle pier acceleration in both directions significantly reduced as higher stiffness section has been used (almost 30-40% decrease). For end pier acceleration pier stiffness doesn't have much significant effect.

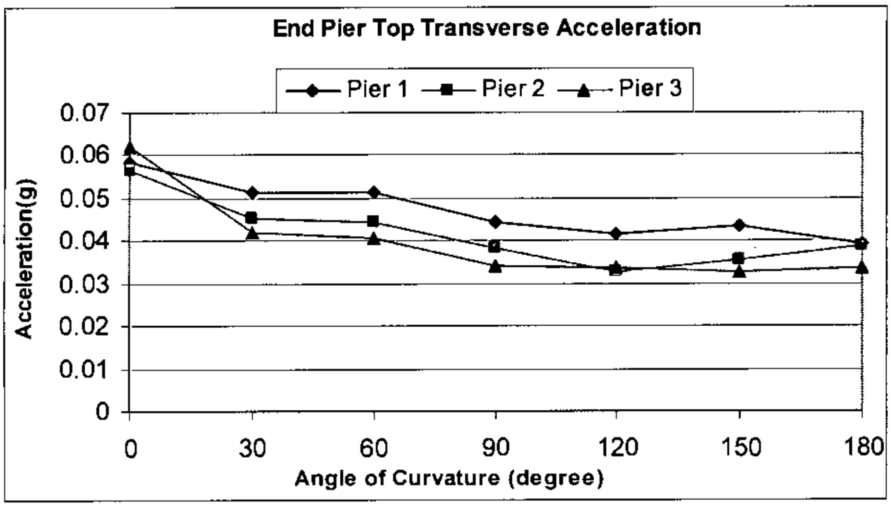


Fig 4.34 Effect of pier stiffness on End Pier Top Transverse Acceleration

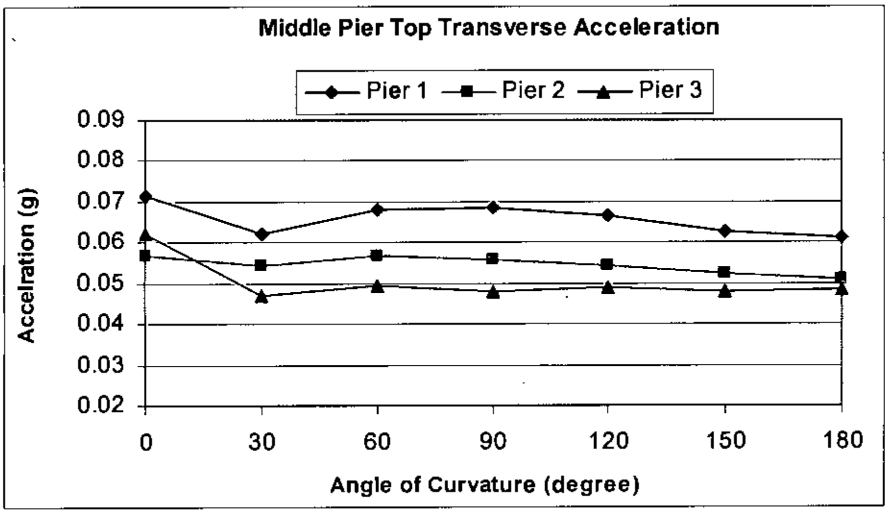


Fig 4.35 Effect of pier stiffness on Middle Pier Top Transverse Acceleration

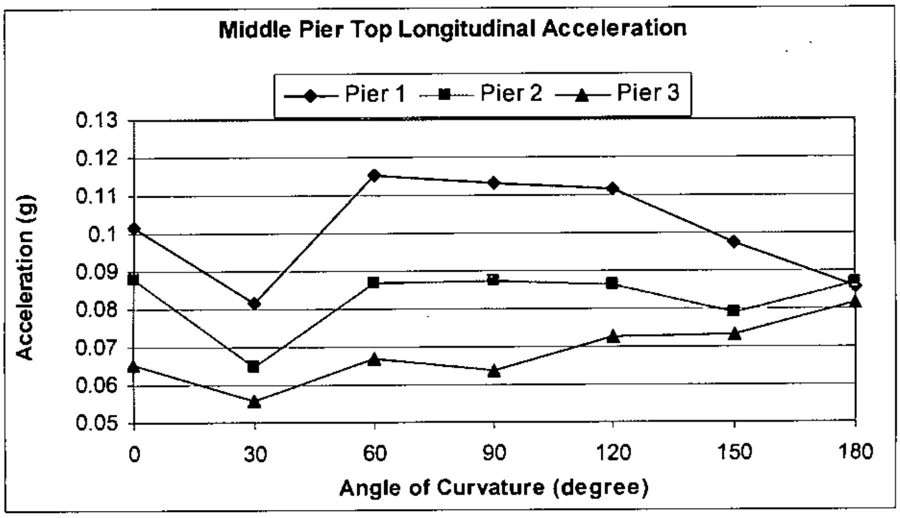


Fig 4.36 Effect of pier stiffness on Middle Pier Top Longitudinal Acceleration

4.6.3 Moments and Shears

The effect of pier stiffness on pier forces is presented in Fig 4.37 to Fig 4.44. End pier and middle pier longitudinal shear and moment about transverse axis increase if stiffer pier is used up to 60^0 curvature. Middle pier and end pier transverse shear and moment about longitudinal axis decrease if stiffer pier is used in between 30^0 to 120^0 curvature. For other curvatures the values are same for all piers. But Takeda et al., (1991) found that increase in pier stiffness increase the value of bending moment at bottom of pier.

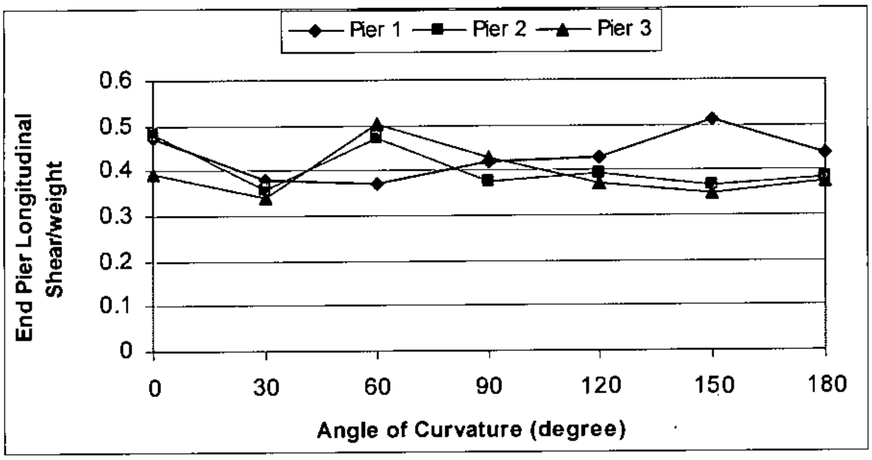


Fig 4.37 Effect of pier stiffness on end pier longitudinal shear

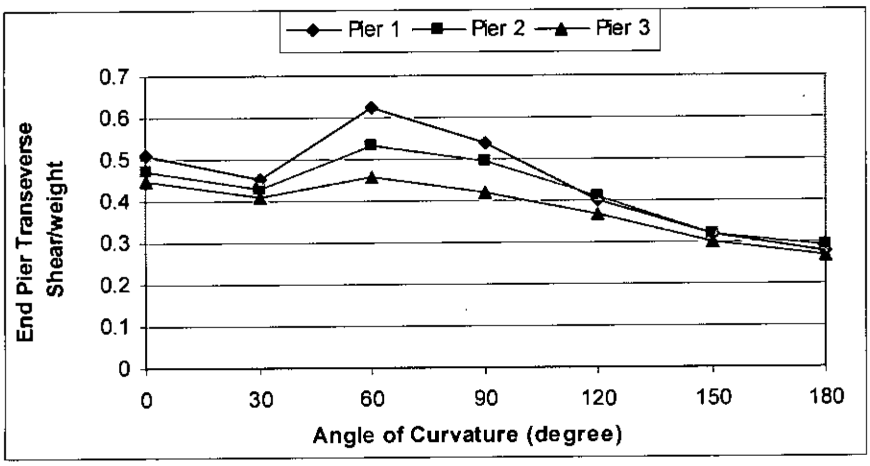


Fig 4.38 Effect of pier stiffness on end pier transverse shear

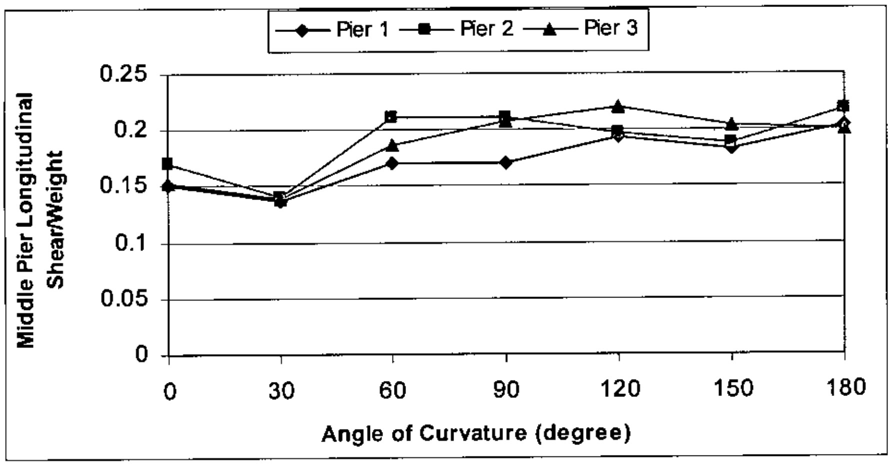


Fig 4.39 Effect of pier stiffness on middle pier longitudinal shear

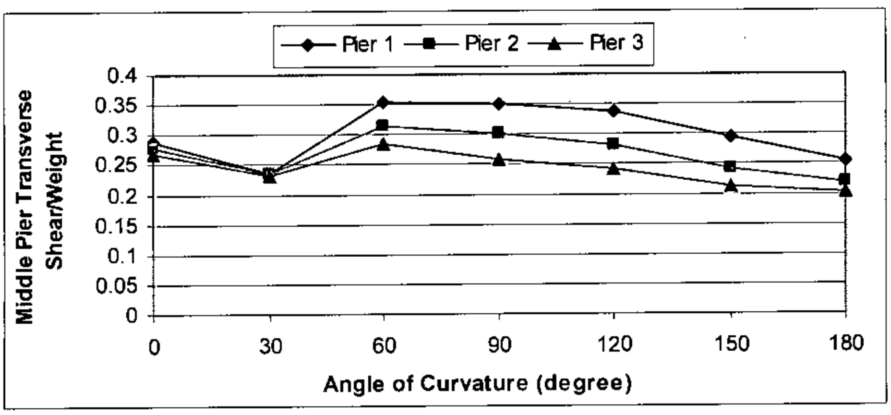


Fig 4.40 Effect of pier stiffness on middle pier transverse shear

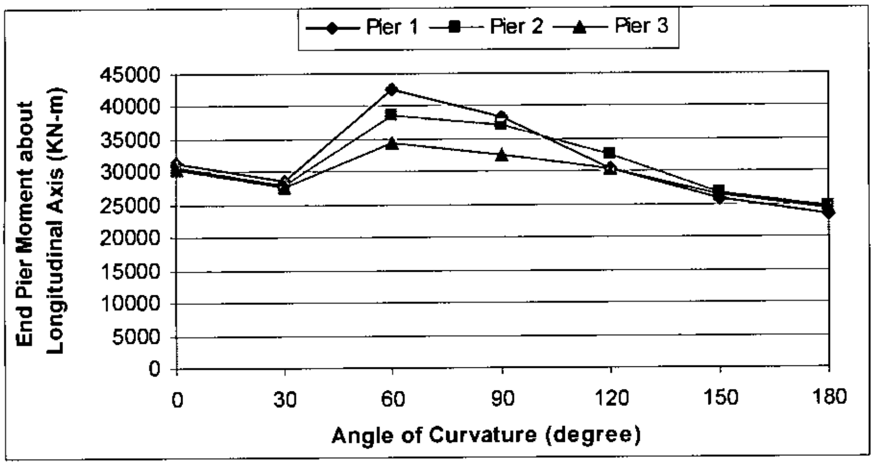


Fig 4.41 Effect of pier stiffness on end pier moment about longitudinal axis

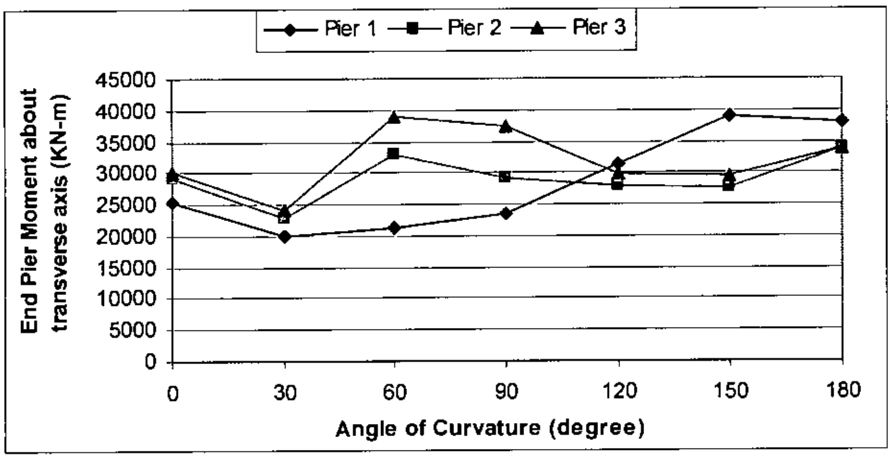


Fig 4.42 Effect of pier stiffness on end pier moment about transverse axis

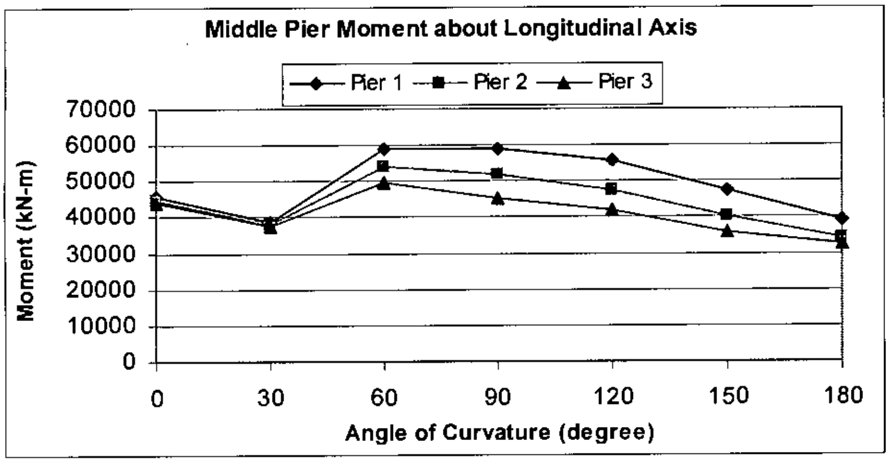


Fig 4.43 Effect of pier stiffness on middle pier moment about longitudinal axis

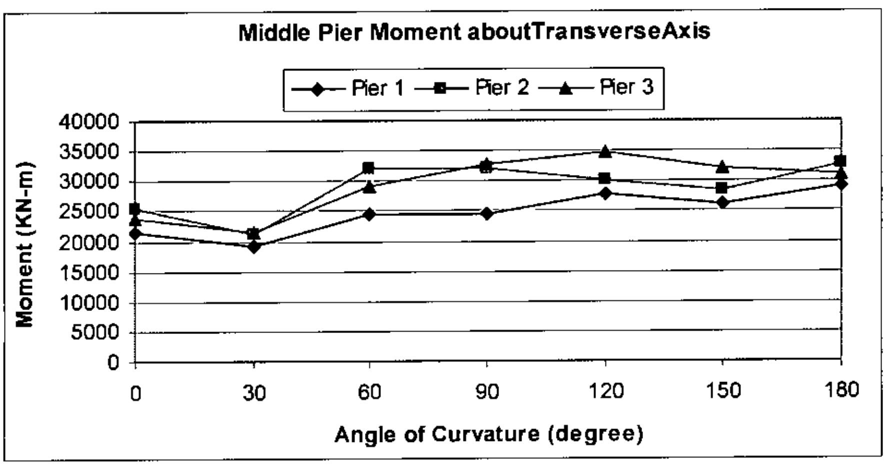


Fig 4.44 Effect of pier stiffness on middle pier moment about transverse axis

Chapter 5

PARAMETRIC STUDY ON RESPONSE OF ISOLATED FLYOVER BRIDGE UNDER SEISMIC LOADING

5.1 Introduction

Isolation is a widely used system for seismic protection. In this chapter, the effect of curvature on various responses of isolated bridge has been presented.

5.2 Statement of the Problem

For the parametric study, simple dynamic models of bridge have been used. The bridge geometry, the pier section and box section of isolated bridges are same as mentioned in the chapter 4. The angles of curvatures are measured in degree. Lead Rubber Isolator has been used in the bridges. Their properties are mentioned in Table 5.1. The earthquake motions are applied in transverse and longitudinal direction simultaneously. The key responses considered in this parametric study are pier and deck midspan responses (displacement, acceleration) in both longitudinal and transverse direction, pier forces (shear, moment), deck midspan moments and isolator bearing displacements. The moment at bottom of pier is considered as pier moment.

In this chapter, the peak responses due to time history analysis of bridges have been presented and values are normalized by peak responses of straight bridge otherwise specified.

Table 5.1 Isolator Properties

Isolation type	Lead Rubber Bearing
Diameter	0.635m,
Number of layers	45
Isolator height	0.3048 m
Lead Diameter	0.127 m
Elastic Stiffness, K_i	30200 KN/m
Characteristic Strength Q	470 KN

Table 5.1 Isolator Properties (contd.)

Compression Stiffness K_c	3677600 KN/m
Ratio of post-yield stiffness to elastic stiffness	0.1538
Effective Stiffness K_e (at 11.6 cm)	8730 KN/m

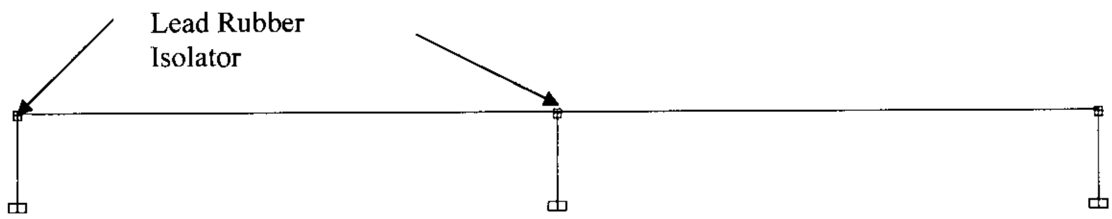


Fig 5.1 Elevation of Isolated Bridge (Straight Bridge)

5.3 Effect of Curvature on Transverse Response

5.3.1 Pier Top Displacement and Acceleration

The effect of curvature on pier displacement and acceleration of isolated bridges is shown in Fig 5.2 to 5.5. The values of displacements are normalized by the response of straight isolated bridge. From the results it is observed that due to curvature the transverse displacement of end pier top and middle pier top both decrease from the value at 30 degree curvature. For Hachinohe Earthquake, end pier displacement value increases to 120% at 30 degree curvature and then gradually decrease to 60% (at 180 degree curvature) of the straight bridge response. Middle pier top shows similar behavior but their values always greater than straight bridge response.

For end pier top transverse accelerations, only for El Centro earthquake it decreases with curvature. In case of middle pier top transverse acceleration, no change of values occurs from the 30 degree curvature response.

5.3.2 Deck Midspan Displacement and Acceleration

No effect of curvature on deck midspan joint displacement is observed up to 30^0 curvature, but after further increase in curvature the value decreases. Almost similar effect is observed in case of deck midspan transverse acceleration (Fig 5.6 and 5.7). For Taft earthquake, midspan joint transverse displacement decrease 25% and midspan joint transverse acceleration decrease almost 26% of the value of straight bridge response at 180 degree curvature.

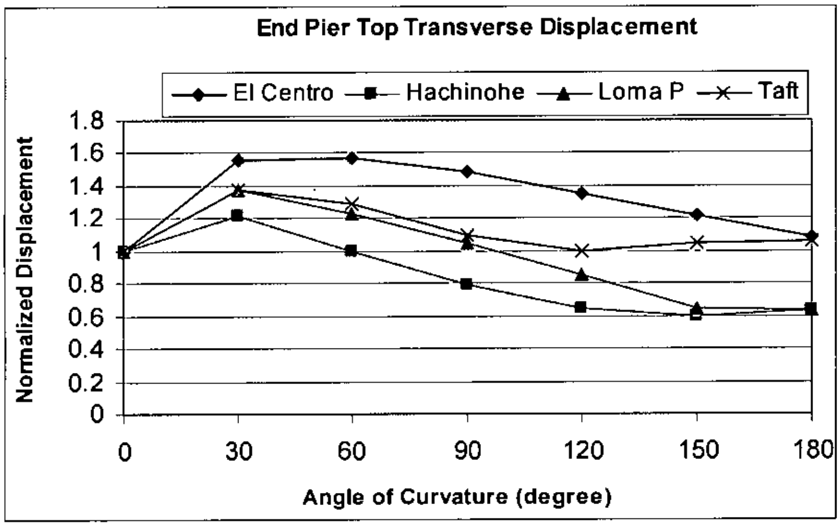


Fig 5.2 Effect of Curvature on End Pier Top Transverse Displacement

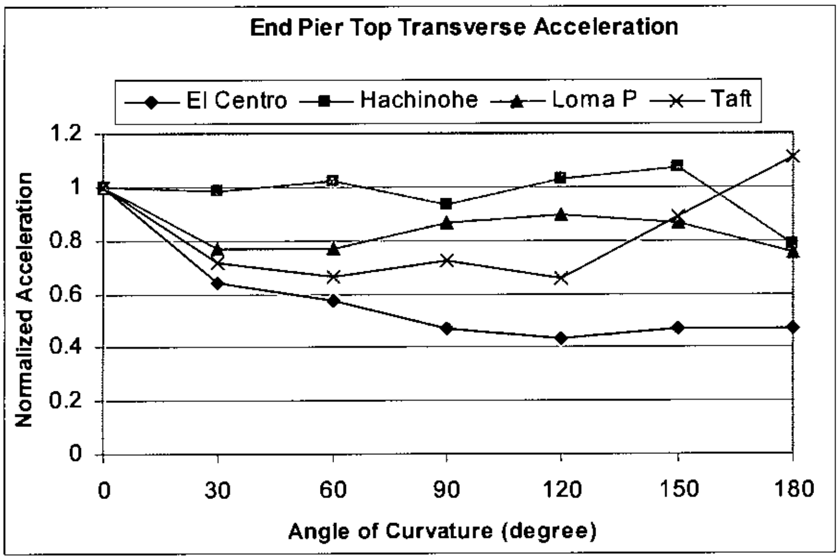


Fig 5.3 Effect of Curvature on End Pier Top Transverse Acceleration

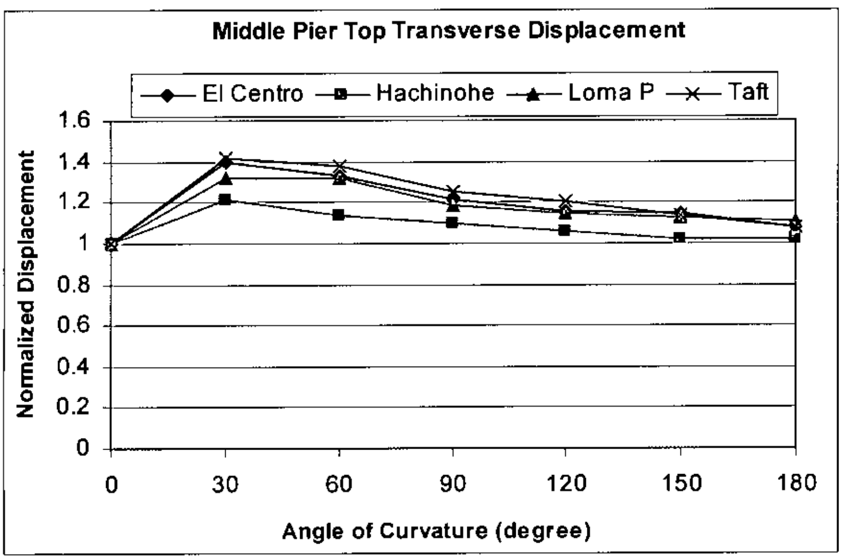


Fig 5.4 Effect of Curvature on Middle Pier Top Transverse Displacement

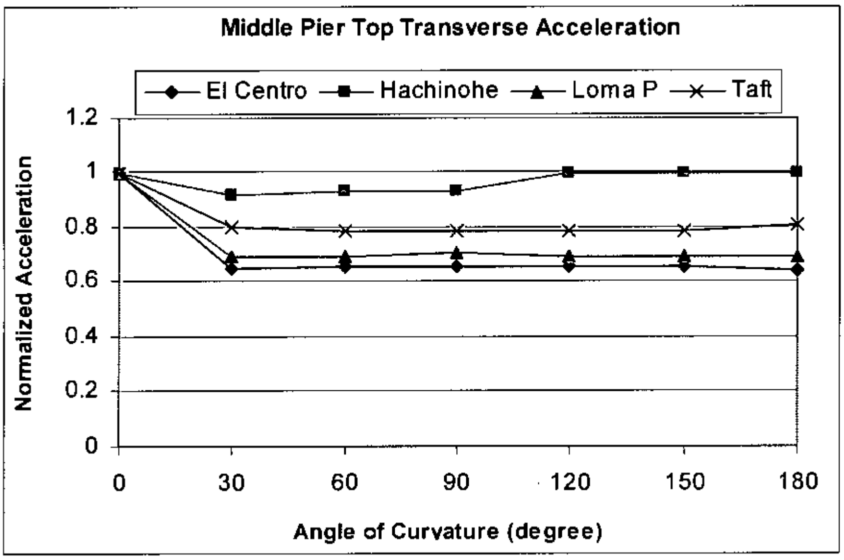


Fig 5.5 Effect of Curvature on Middle Pier Top Transverse Acceleration

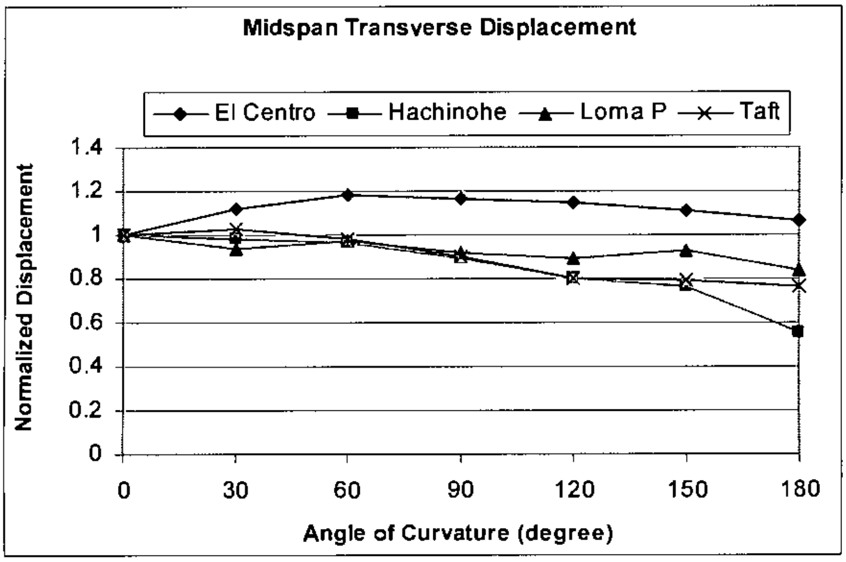


Fig 5.6 Effect of Curvature on Midspan Transverse Displacement

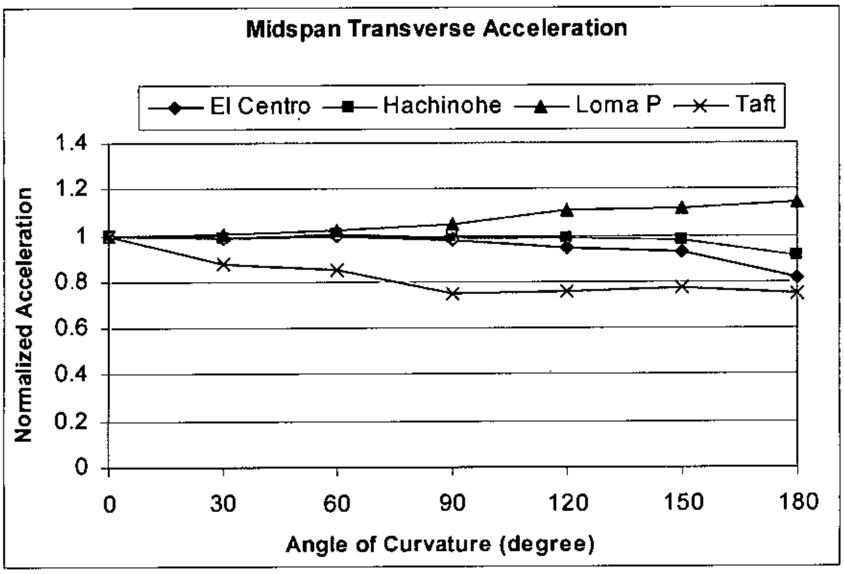


Fig 5.7 Effect of Curvature on Midspan Transverse Acceleration

5.3.3 Pier forces

The effect of curvature on these pier forces are presented in Fig 5.8 to Fig 5.11. From results it is observed that end pier transverse shear decreases with curvature for Hachinohe, Loma Prieta and Taft earthquake. For moment about longitudinal axis, it reaches its peak value at 30^0 and then gradually declines with increase in curvature. For middle pier, curvature has almost no effect on transverse shear, but moment about longitudinal axis reaches its peak at 300 curvature and then decreases gradually with increased curvature.

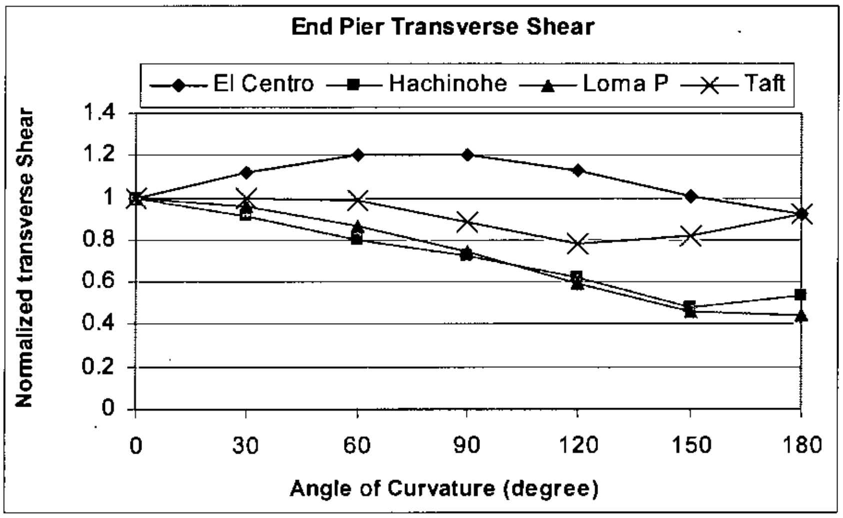


Fig 5.8 Effect of Curvature on End Pier Transverse Shear

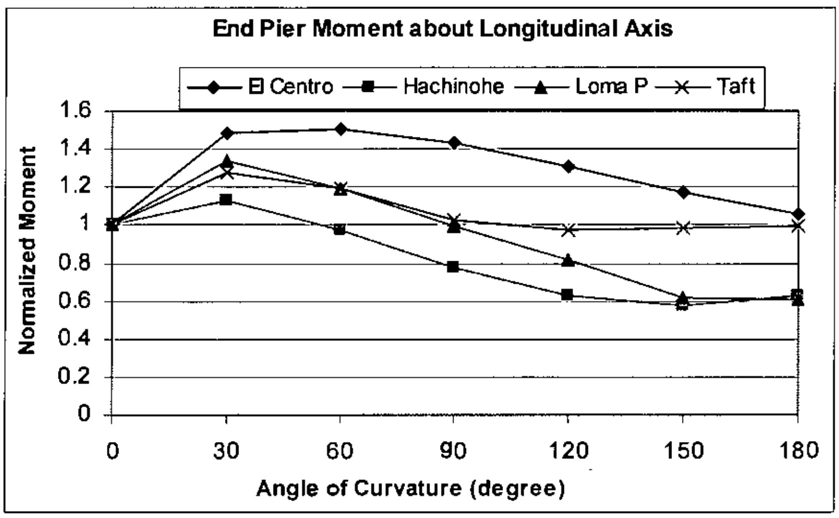


Fig 5.9 Effect of Curvature on End Pier Moment about Longitudinal Axis

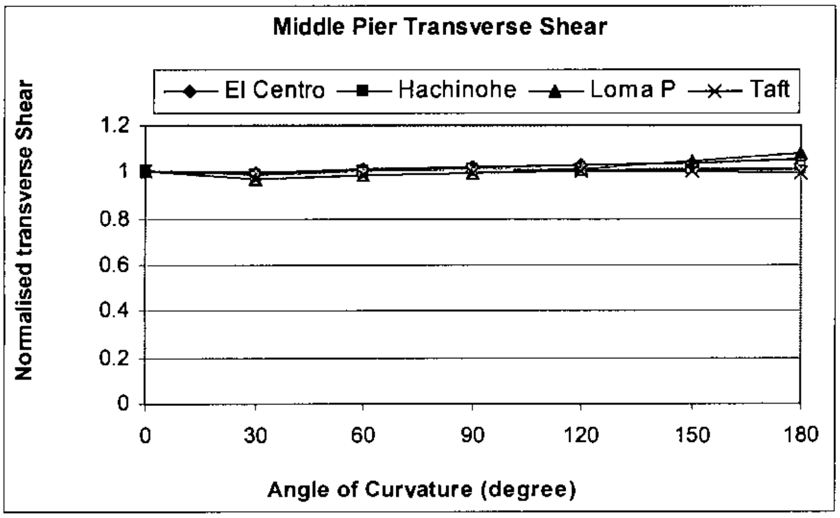


Fig 5.10 Effect of Curvature on Middle Pier Transverse Shear

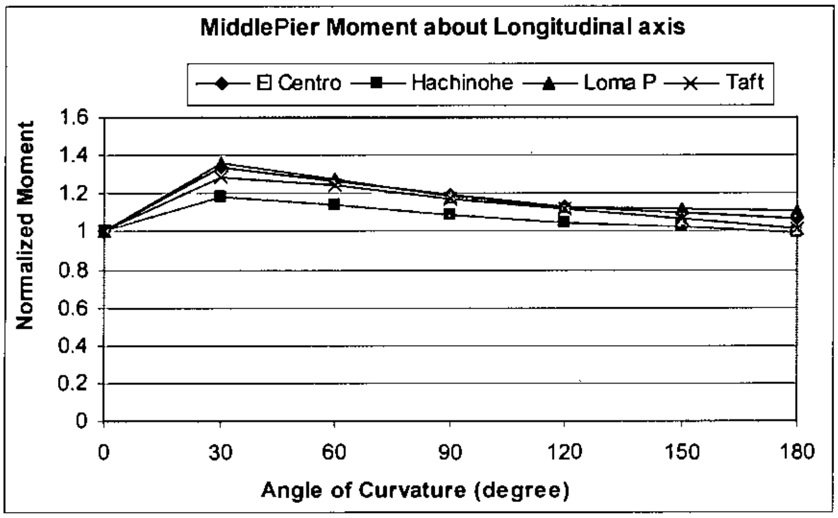


Fig 5.11 Effect of Curvature on Middle Pier Moment about Longitudinal Axis

5.4 Effect of Curvature on Longitudinal Response

In this section longitudinal acceleration and displacement of pier and deck has been presented

5.4.1 Pier Top Displacement and Acceleration

Effect of curvature on longitudinal responses of end pier and middle pier are presented in Fig 5.12 to Fig 5.15. It is seen that end pier top longitudinal acceleration decrease at 30^0 curvature and then gradually increase with increase in curvature for Hachinohe and El Centro earthquake. But longitudinal displacement increase with curvature for all earthquakes. For Loma Prieta Earthquake it increases almost 100% from its value for straight bridge (0 degree curvature). For middle pier top displacement this increase is not so significant (about 20% increase for Loma Prieta EQ). In case of longitudinal acceleration of middle pier, curvature has almost no effect. It decreases at 30^0 curvature and then remain constant for other increased curvature.

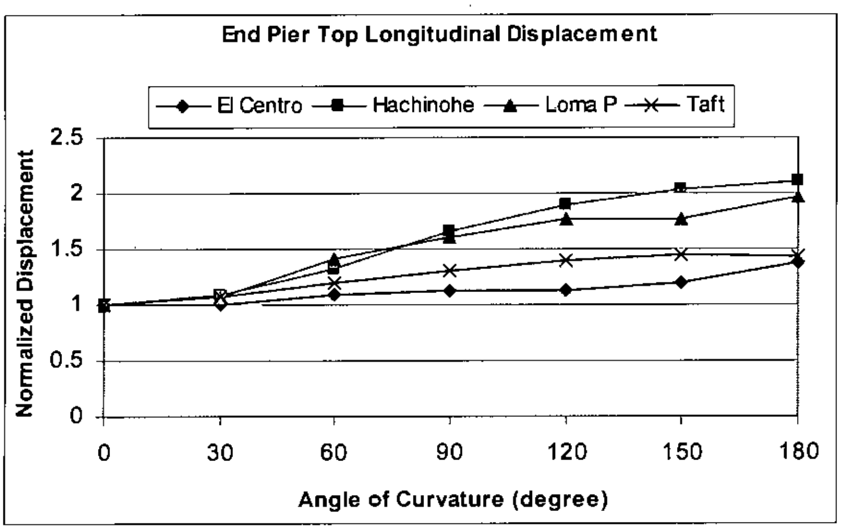


Fig 5.12 Effect of Curvature on End Pier Top Longitudinal Displacement

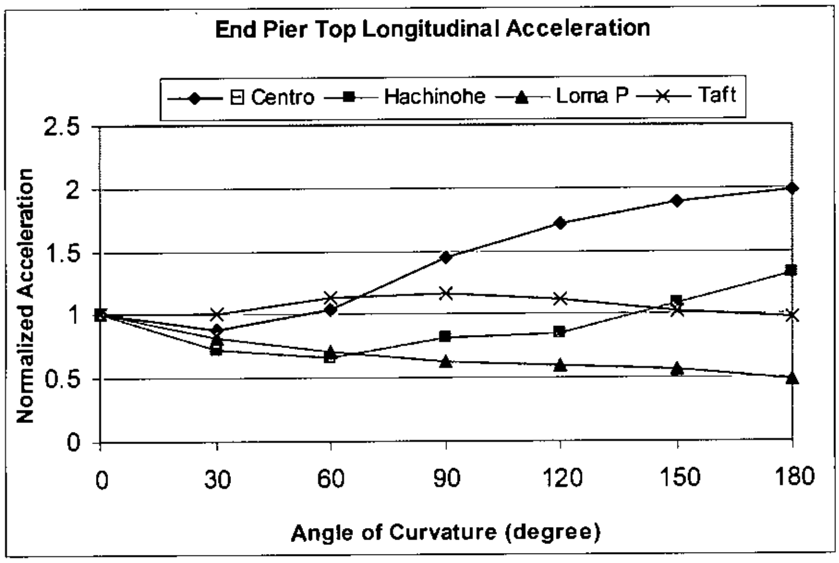


Fig 5.13 Effect of Curvature on End Pier Top Longitudinal Acceleration

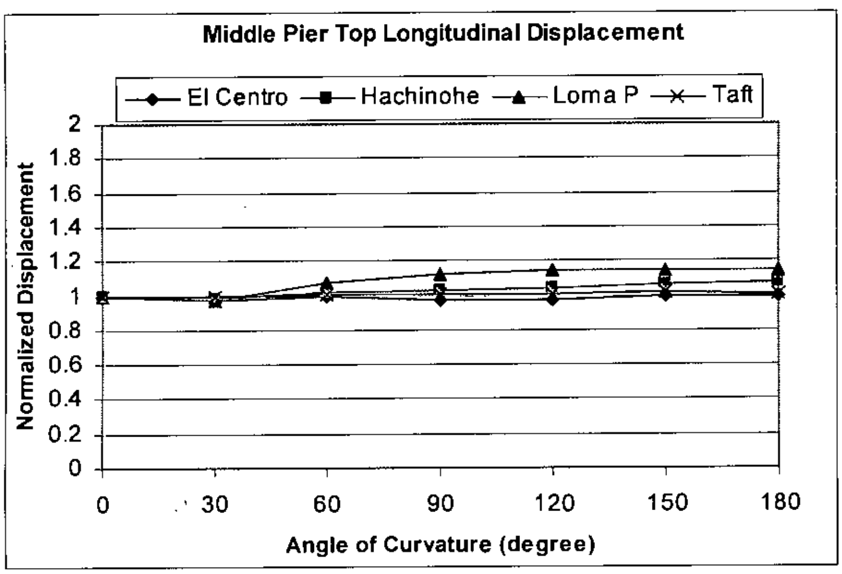


Fig 5.14 Effect of Curvature on Middle Pier Top Longitudinal Displacement

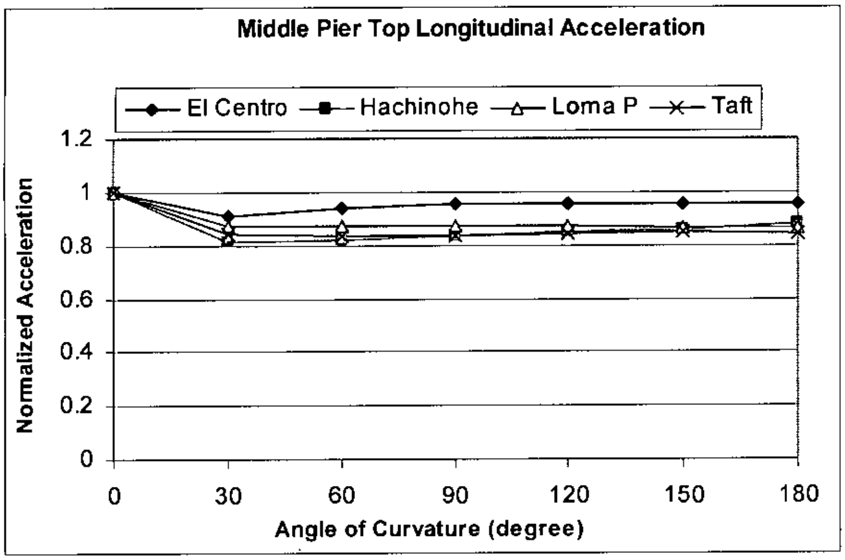


Fig 5.15 Effect of Curvature on Middle Pier Top Longitudinal Acceleration

5.4.2 Deck Midspan Displacement and Acceleration

Midspan longitudinal responses are shown in Fig 5.16 and 5.17. Result shows that midspan displacement and acceleration has higher values at other curved bridges than that of straight bridge. For Hachinohe earthquake load, the value increase gradually. At 180 degree curvature this value increases almost 90 % from the value of straight bridge.

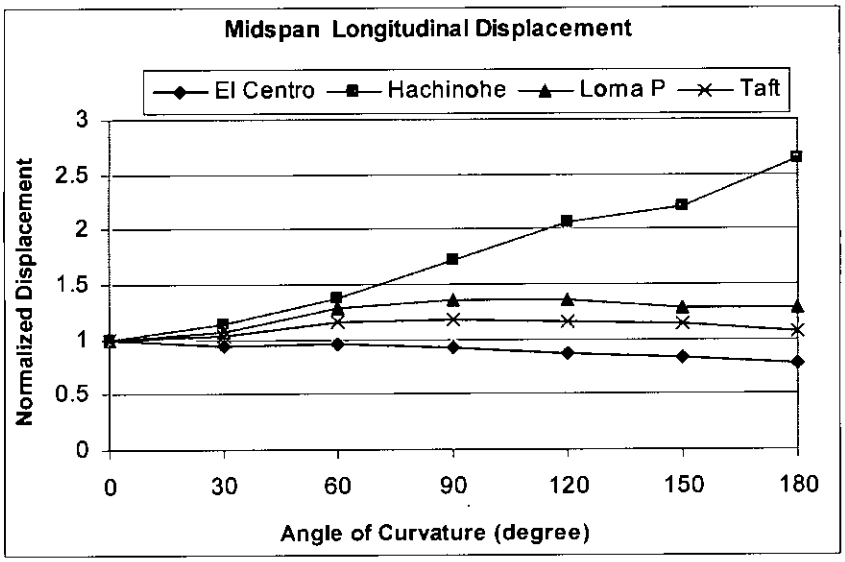


Fig 5.16 Effect of Curvature on Midspan Longitudinal Displacement

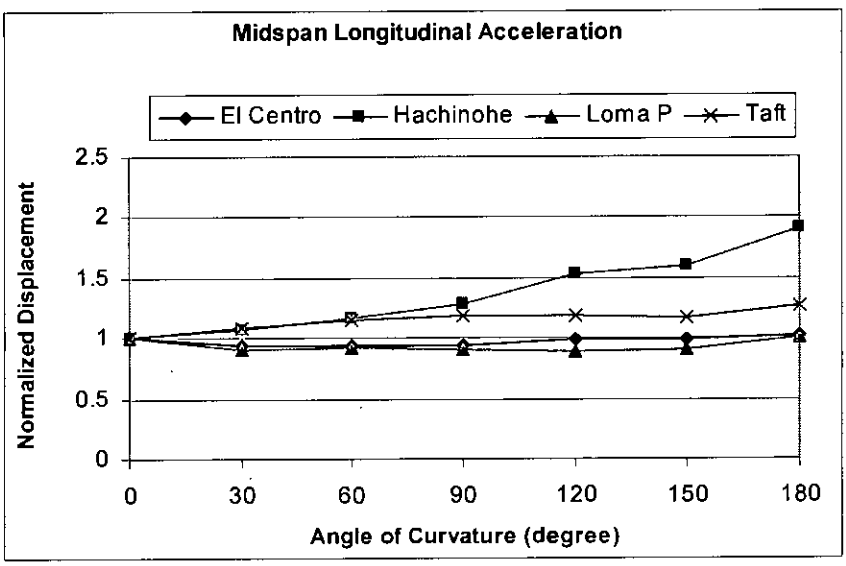


Fig 5.17 Effect of Curvature on Midspan Longitudinal Acceleration

5.4.3 Pier Forces

From the analysis results, it is observed that end pier longitudinal shear and end pier moment about transverse axis both increase with increase in curvature for all earthquake.

Again, middle pier longitudinal shear and moment about transverse axis remain unaffected with increase in curvature, as shown in Fig 5.18 to Fig 5.21.

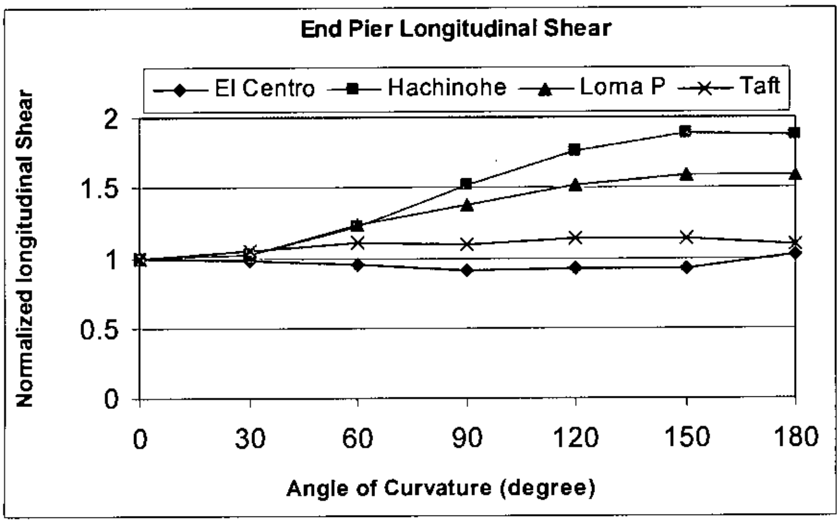


Fig 5.18 Effect of Curvature on End Pier Longitudinal Shear

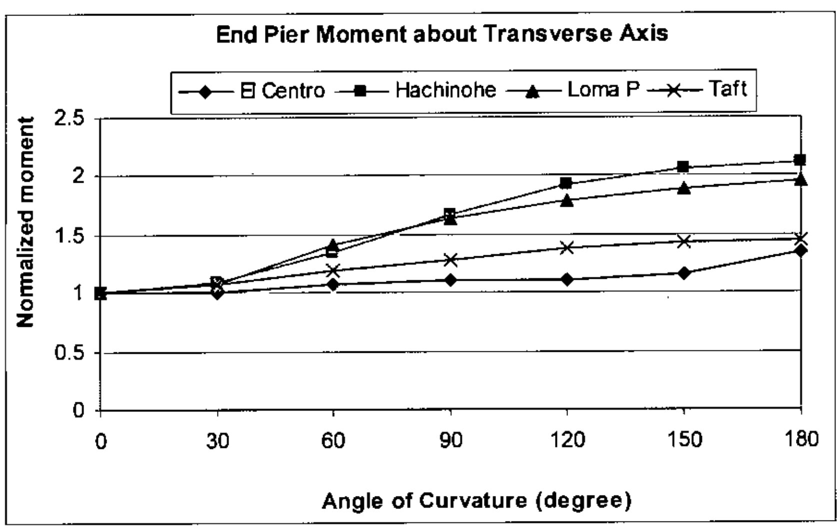


Fig 5.19 Effect of Curvature on End Pier Moment about Transverse Axis

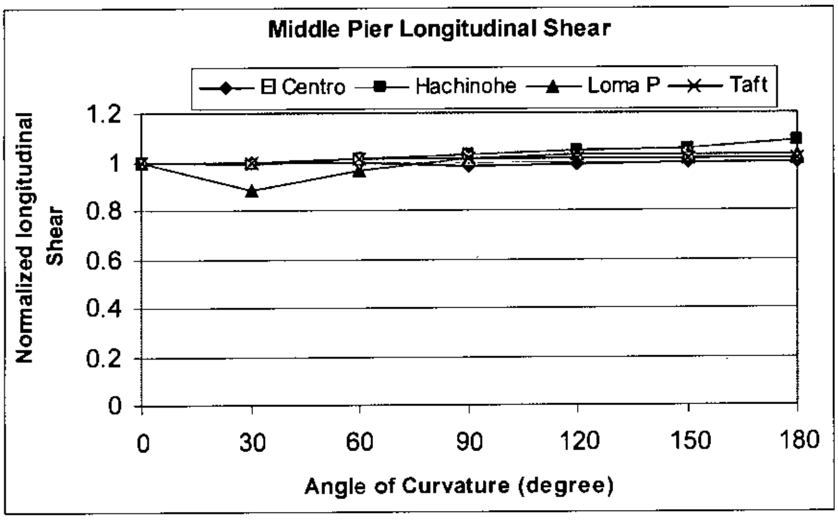


Fig 5.20 Effect of Curvature on Middle Pier Longitudinal Shear

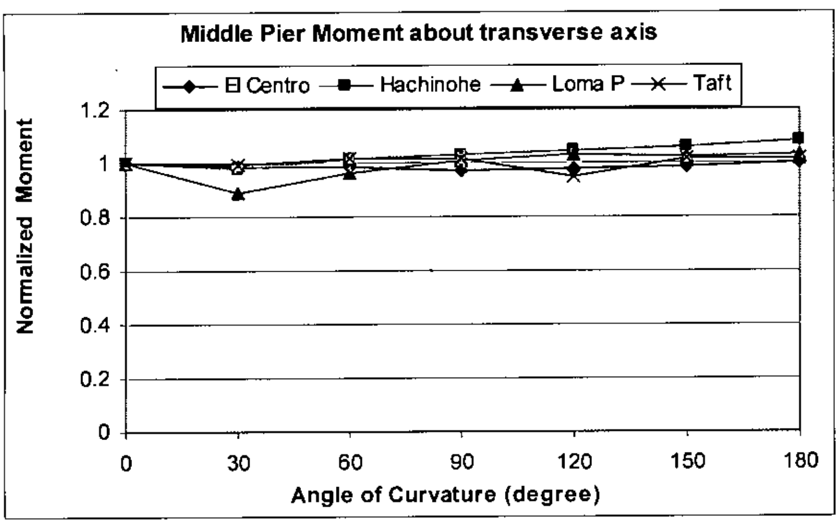


Fig 5.21 Effect of Curvature on Middle Pier Moment about Transverse Axis

5.5 Effect of Curvature on Deck Mid Span Moments

To evaluate the effect of curvature midspan torsion, midspan moments about horizontal axis and midspan moment about vertical axis are considered. Fig 5.22 shows that midspan torsion significantly increases with curvature. At 180 degree curvature it increases almost 900% of the value at 30 degree curvature. Other moments at midspan decrease with increase in curvature as shown in Fig 5.23 and 5.24. It is to be noted that midspan moment isn't the maximum moment. Usually, maximum moment with respect to transverse axis increase with curvature as shown in Fig 5.25.

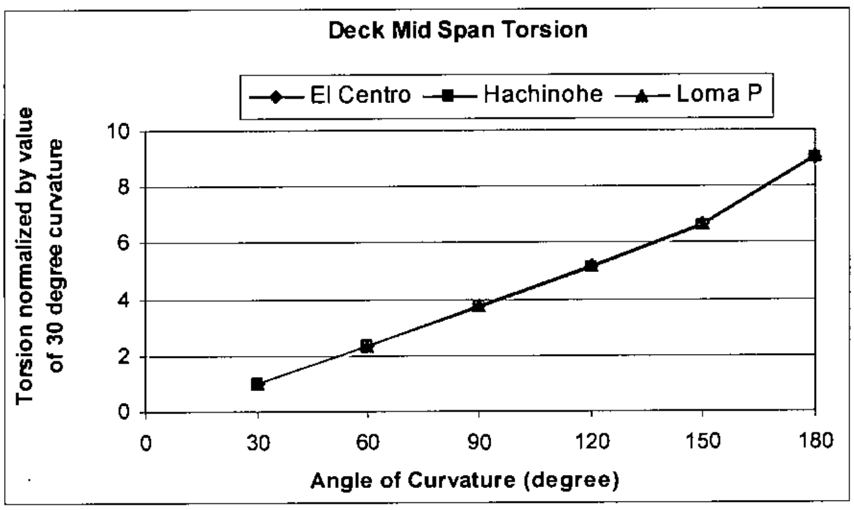


Fig 5.22 Effect of Curvature on Midspan Torsion

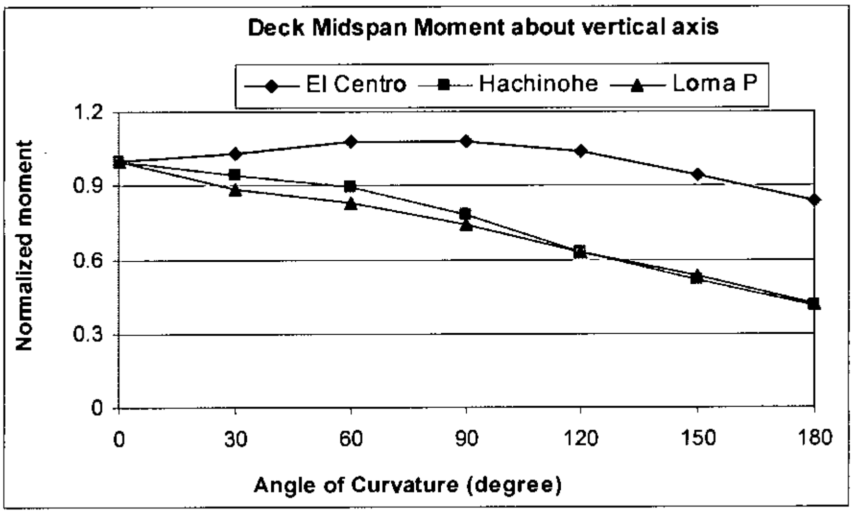


Fig 5.23 Effect of Curvature on Midspan Moment about Vertical Axis

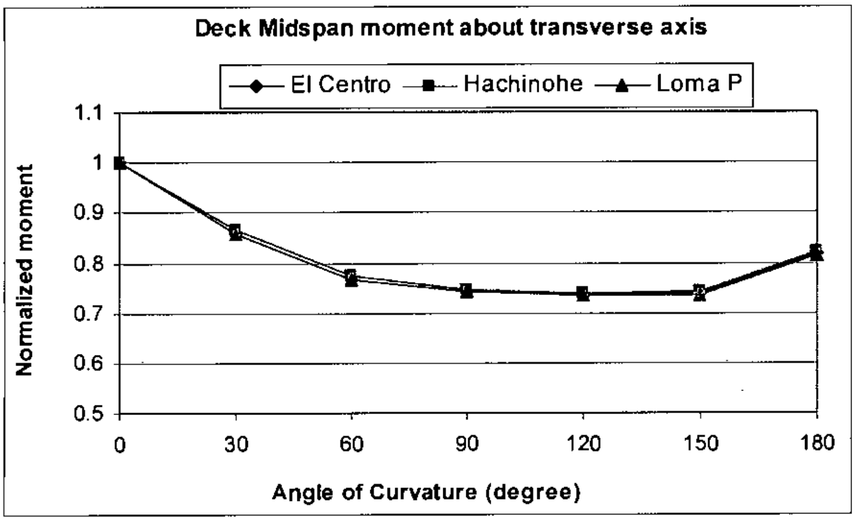
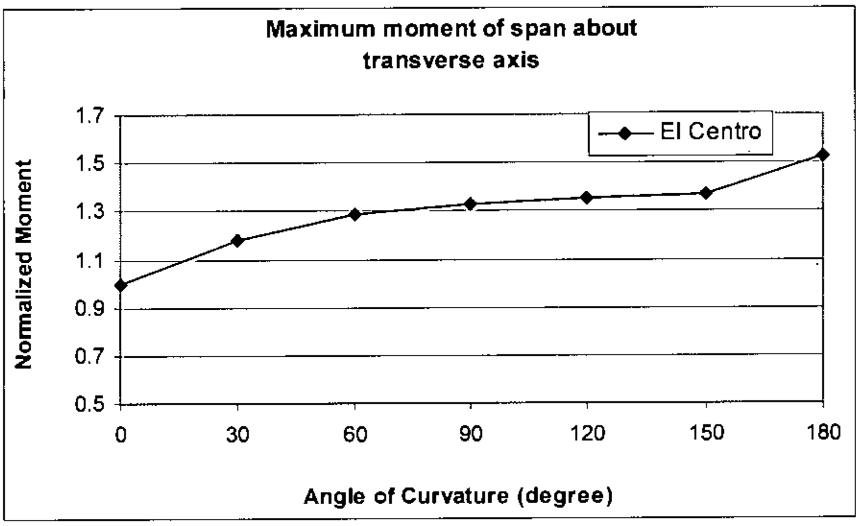


Fig 5.24 Effect of Curvature on Midspan Moment about Transverse Axis



5.25 Effect of Curvature on Maximum Moment of the Span about Transverse Axis

5.6 Effect of Curvature on Isolator Displacement

Isolator displacement is a very important response of an isolated bridge. In isolated bridge, isolator displacement is generally high with compared to the non-isolated bridge as it provide seismic protection by deformation. In bridge design, maximum isolator displacement should be estimated and sufficient provision should be maintained to accommodate this displacement. Here, bearing between end pier and deck is considered. Fig 5.26 and 5.27 shows the effect of curvature on isolator displacement in both longitudinal and transverse direction.

It is observed that isolator longitudinal displacement increase with increase in curvature for some earthquake loads and isolator transverse displacements decrease with increase in curvature. Hosada et al., (1991) also found that, for curved isolated bridge, seismic response of isolator in the tangential direction is larger than the corresponding response of the straight bridge in the longitudinal direction, while the seismic responses in the normal direction are smaller than the corresponding responses of the straight bridge in the transverse direction.
For Hachinohe earthquake, bearing displacement for 180 degree curvature is 250% of the displacement of straight bridge. Again, at 180 degree curvature the transverse

isolator displacements for El Centro, Hachinohe and Loma Prieta earthquakes are about 50%, 30% and 40% of the straight bridge isolator displacement respectively.

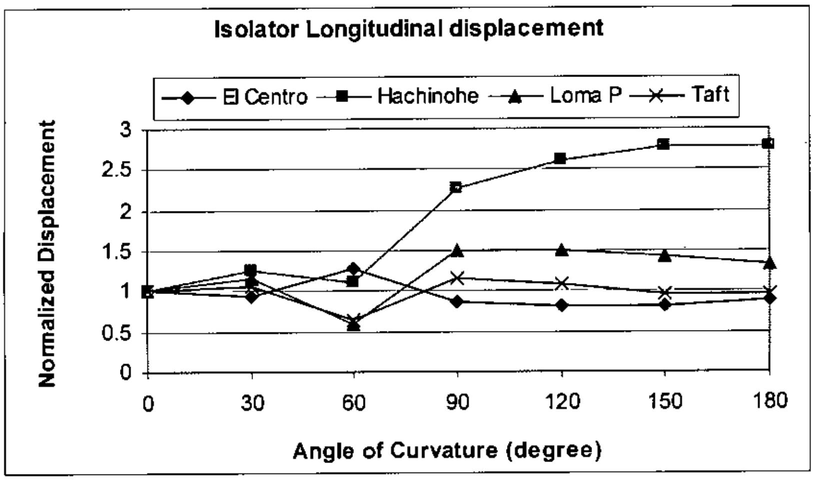


Fig 5.26Effect of Curvature on Isolator Longitudinal Displacement

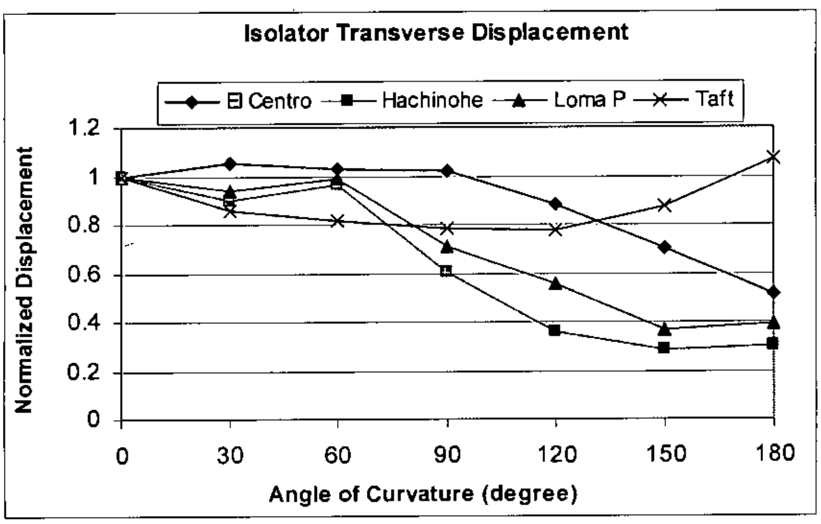


Fig 5.27 Effect of Curvature on Isolator Transverse Displacement

5.7 Effectiveness of Isolation: Comparison of Various Responses of Isolated and Non-Isolated Bridges

Isolation system is a widely adopted technique to minimize seismic forces. By lengthening the period of vibration it reduces the elastic forces to which bridge will be subjected and also resulting column forces (as shown in Fig 2.12). In this study, the effectiveness of seismic isolation is also evaluated. The responses for which isolated bridge is compared with non-isolated bridge are the transverse shear of end pier, end pier moment about transverse axis, end pier transverse displacement, end pier transverse acceleration, midspan displacement and midspan acceleration. The time variation of end pier transverse shear of both isolated and non-isolated bridge is also presented here. The effect of isolation for end pier transverse shear, end pier moment about longitudinal axis, longitudinal shear, moment about transverse axis, end pier transverse and longitudinal displacement and acceleration are presented in Fig 5.28 to Fig 5.38. It is observed that due to isolation end pier transverse shear and moment about transverse axis decrease with compared to non-isolate bridge. When curvature is low, the reduction of transverse shear is significant for Hachinohe and El Centro earthquake (almost 70-80%). Isolation also decreases the value of pier moment about transverse axis and this reduction is relatively low at lower curvature and high at higher curvature. In isolated bridge the end pier transverse displacement and transverse acceleration significantly reduced (almost 50%) from non-isolated bridge. Jangid (2004) studied the effect of bidirectional interaction of restoring forces of isolation bearing on the seismic response of a continuous span isolated bridge. The responses are also compared with the responses of non-isolated bridge. It is found that, for Kobe Earthquake and for bridge 3, longitudinal pier base shear decrease by 63% and transverse base shear decrease by 85% due to isolation. For Loma Prieta Earthquake and for bridge 3, longitudinal pier base shear decrease by 80% and transverse base shear decrease by 80% due to isolation.

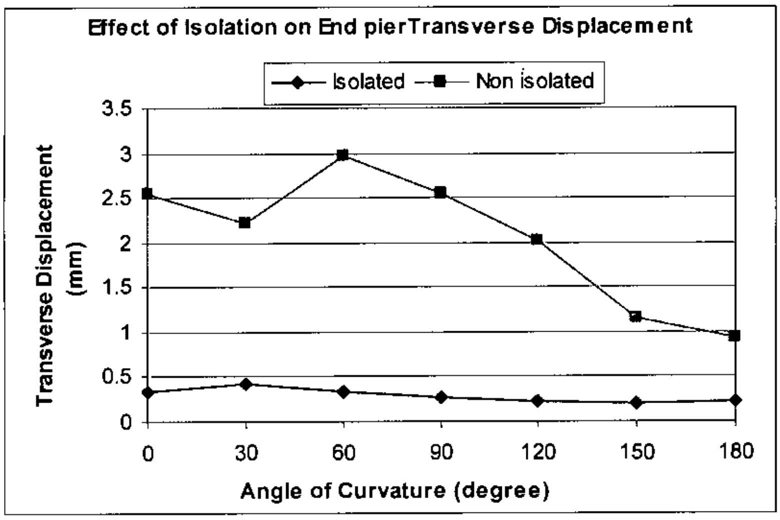


Fig 5.28 Effect of Isolation on End Pier Top Transverse Displacement (Hachinohe)

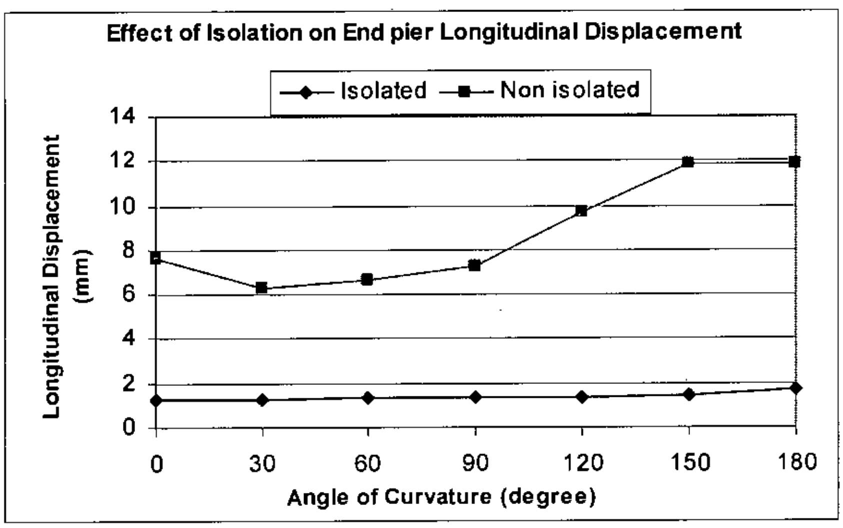


Fig 5.29 Effect of Isolation on End Pier Top Longitudinal Displacement (El Centro)

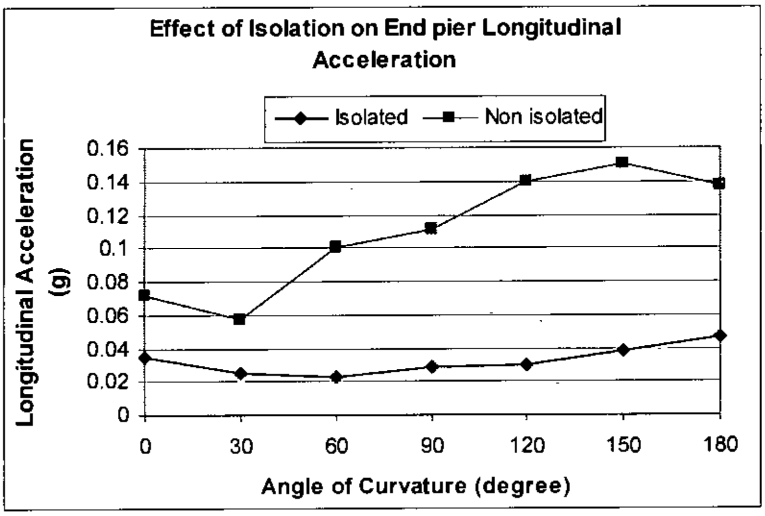


Fig 5.30 Effect of Isolation on End Pier Top Longitudinal Acceleration (Hachinohe)

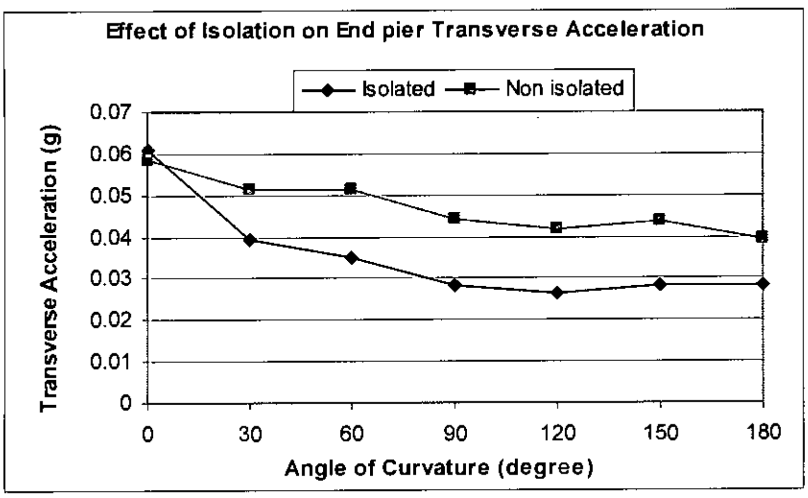


Fig 5.31 Effect of Isolation on End Pier Top Transverse Acceleration (El Centro)

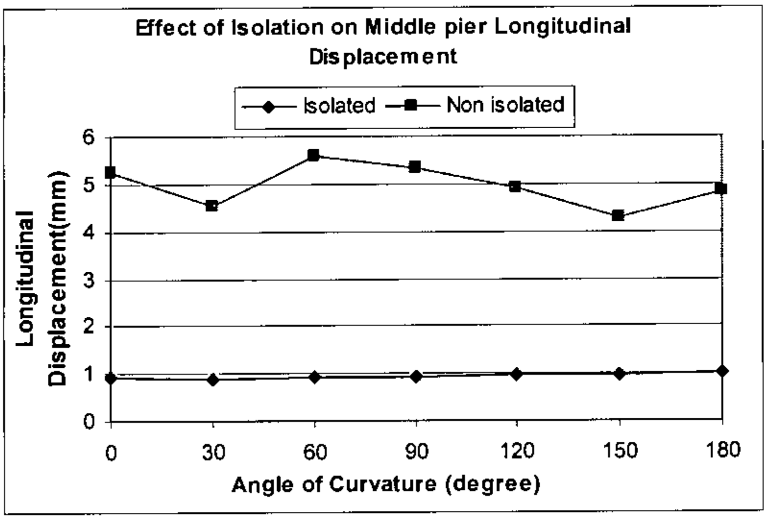


Fig 5.32 Effect of Isolation on Middle Pier Top Longitudinal Displacement (Hachinohe)

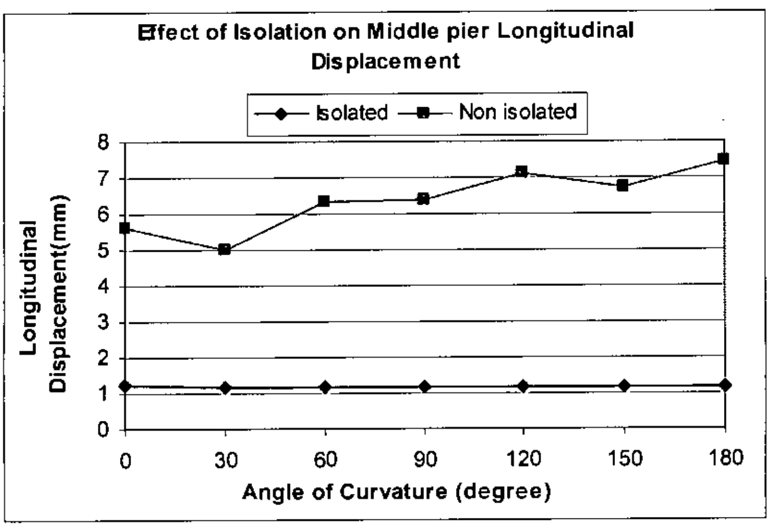


Fig 5.33 Effect of Isolation on Middle Pier Top Longitudinal Displacement (El Centro)

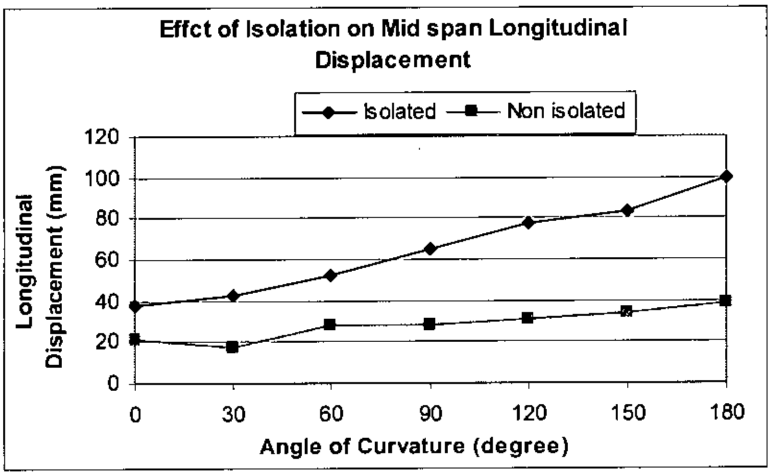


Fig 5.34 Effect of Isolation on Midspan Longitudinal Displacement (Hachinohe)

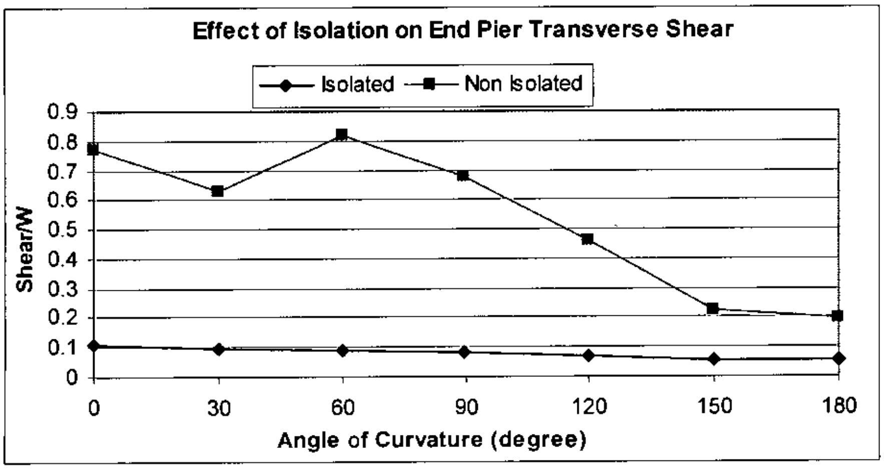


Fig 5.35 Effect of Isolation on End Pier Transverse Shear (Hachinohe)

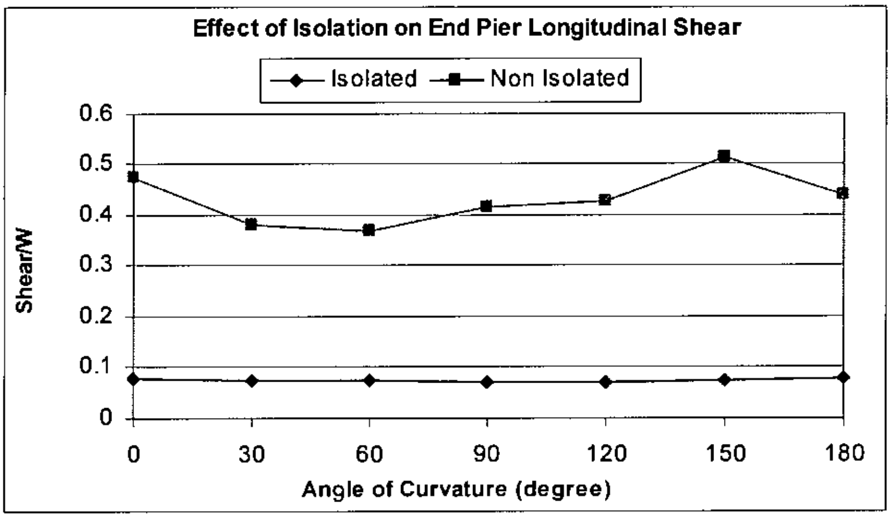


Fig 5.36 Effect of Isolation on End Pier Longitudinal Shear (El Centro)

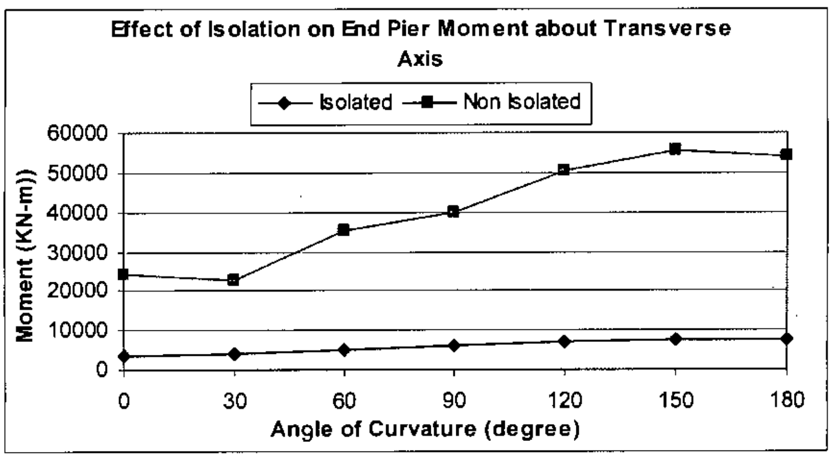


Fig 5.37 Effect of Isolation on End Pier Moment about Transverse Axis (Hachinohe)

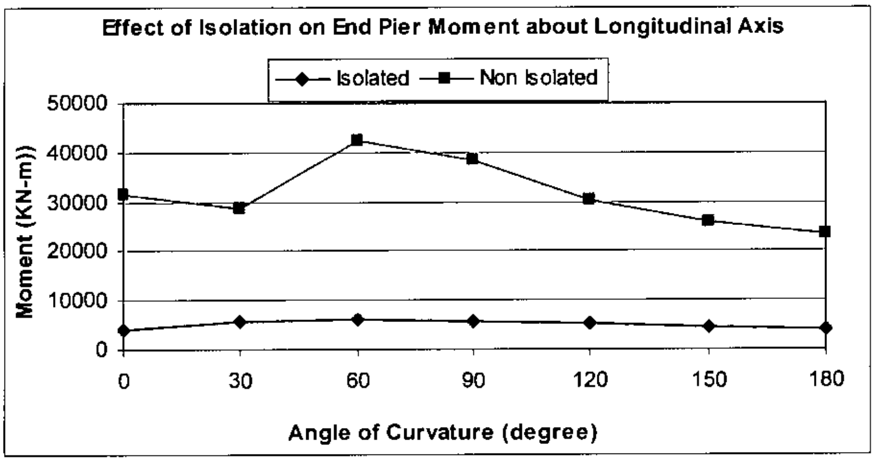


Fig 5.38 Effect of Isolation on End Pier Moment about Longitudinal Axis (El Centro)

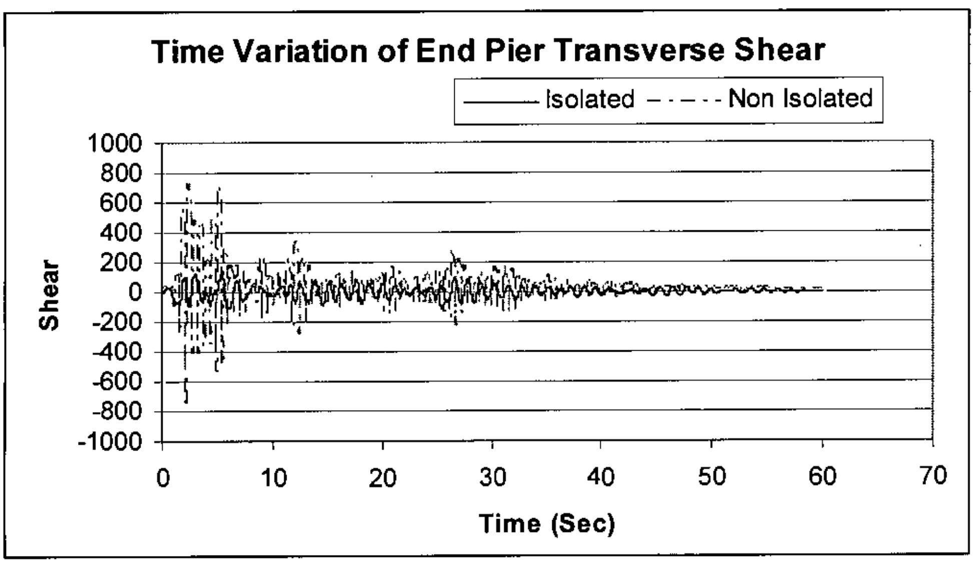


Fig 5.39 Time Variation of End Pier Transverse Shear (El Centro) (30^0 curvature)

Chapter 6

SEISMIC RESPONSE OF MOHAKHALI FLYOVER BRIDGE – A CASE STUDY

6.1 Introduction

Mohakhali Flyover is the first flyover type structure of Dhaka City. This was planned as part of Dhaka Urban Transport Project aiming to ease traffic congestion and to provide better transport services at Mohakhali rail-crossing area of the Dhaka City. The plan was undertaken in 1996. The total project cost of Tk. 113 crore was financed jointly by the Government of Bangladesh and the World Bank. It took 5 years to finalize the process and the contract was awarded to Chinese Company in Nov 2001. The flyover was completed and opened to traffic in Nov 2004 (Fig 6.1). It connects Banani at its north and Old Airport at its west. The objectives of the present study are to establish a simplified model of the bridge that can be effectively used for dynamic analysis and examine the effect of bridge curvature on the pier force and bridge responses.

6.2 Brief Description of Mohakhali Flyover

Total length of the bridge structure along the centerline of deck is 687 m. The whole structure is of reinforced concrete with four-lane box girder deck. The structure has 19 spans in total, but they are divided into three structural modules (Fig 6.2). The each of the two modules at the ends are made of pre-cast segments and have typical span length of 38 meter with around 27 meter long end spans at both ends. The module in the middle is cast in situ and has a long span of 63 meter over the railroad. The other spans in this module range from around 31 to 35 meters (DSM Consultants 2002). To minimize the effect of seismic forces, Shock Transmission Unit (STU) has been used in Mohakhali Flyover Bridge in between girders and the piers.

6.2.1 Pier Stem

The height of the pier stem varies from 2.0m to 9.95m and is constructed of reinforced concrete. Two different pier sections have been used. Fig 6.3 to Fig 6.6 shows the two pier cross sections and the elevations of the pier stem. The cross-sectional properties of the pier sections are given in Table 6.1.

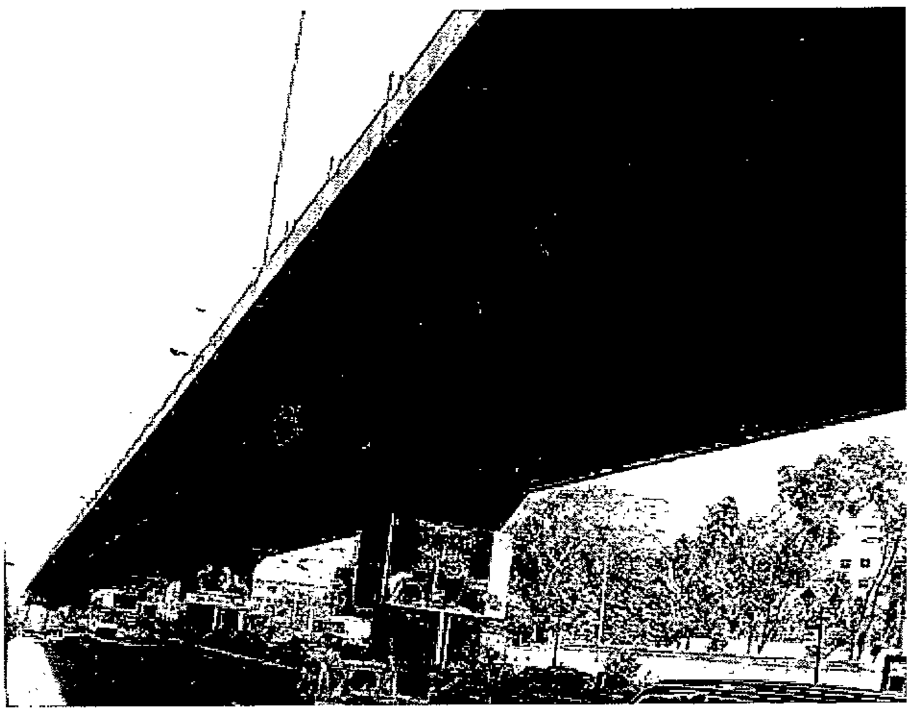


Fig 6.1 Mohakhali Flyover

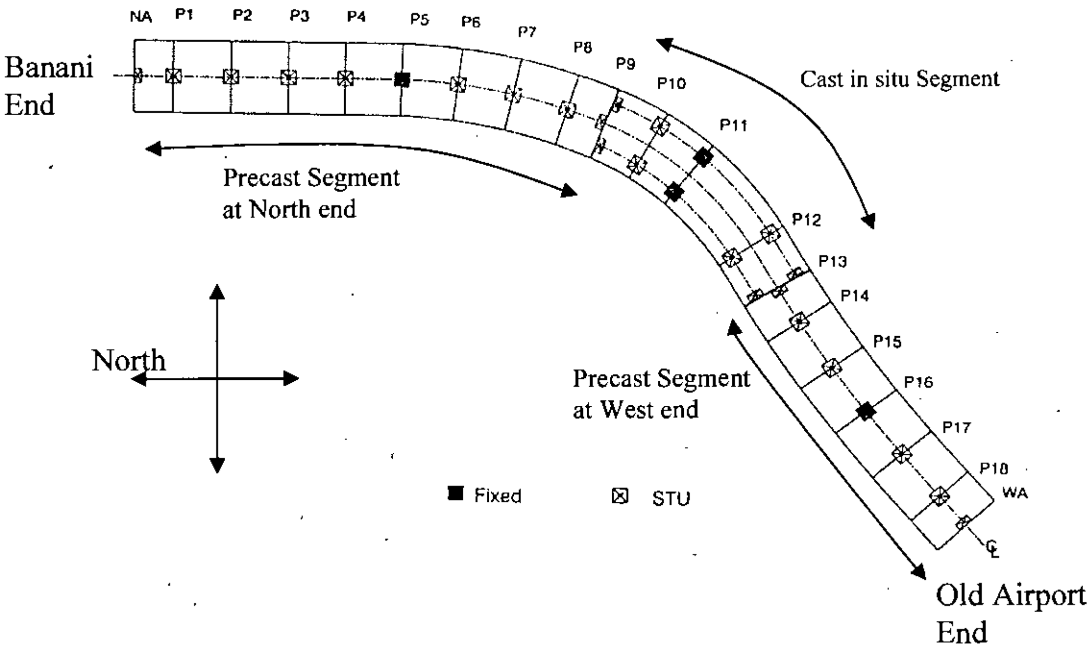


Fig 6.2 Different Segment of Mohakhali Flyover

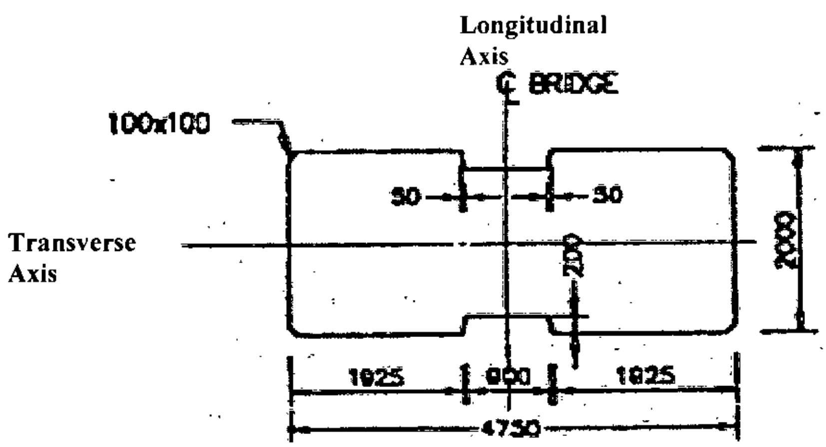


Fig 6.3 Section of Pier for P9, P10 and P13 (All Dimensions in mm)

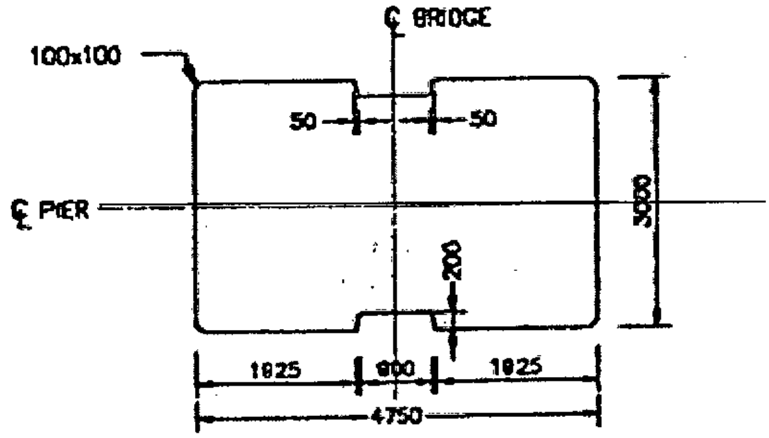


Fig 6.4 Section of Pier for P11 and P12 (All Dimension in mm)

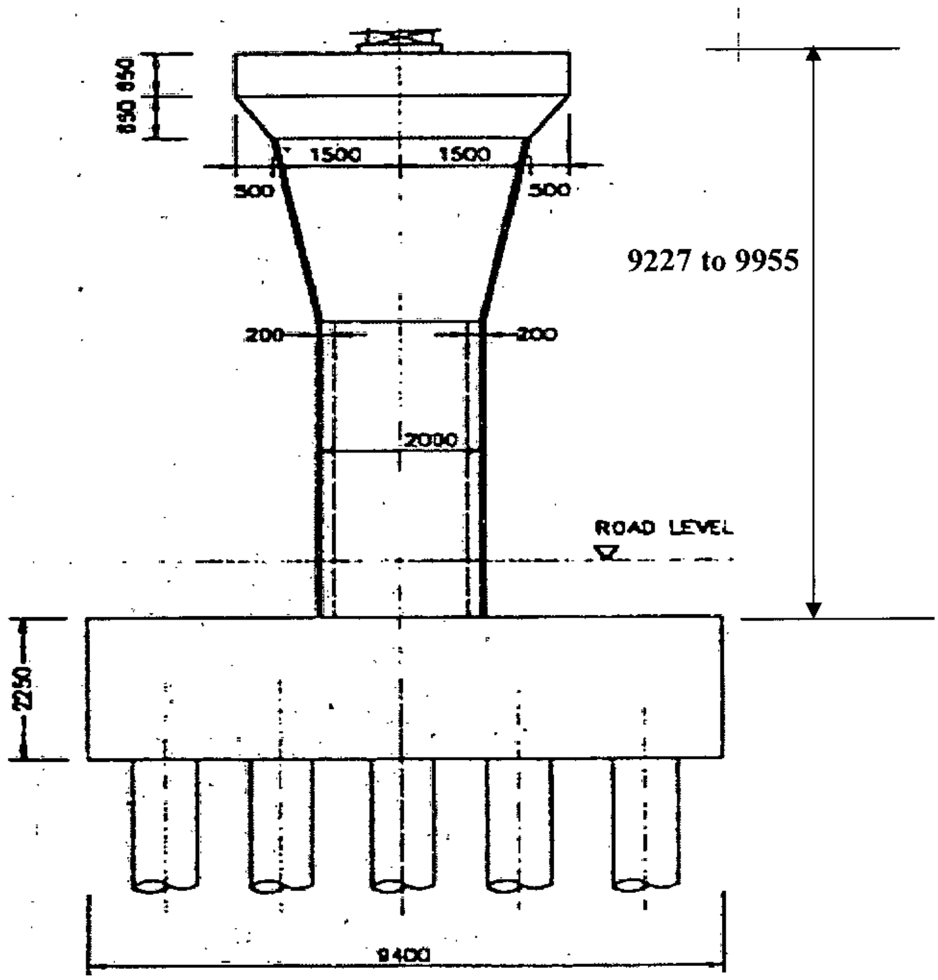


Fig 6.5 Elevation at Pier (P9, P10 and P13) (All Dimensions in mm)

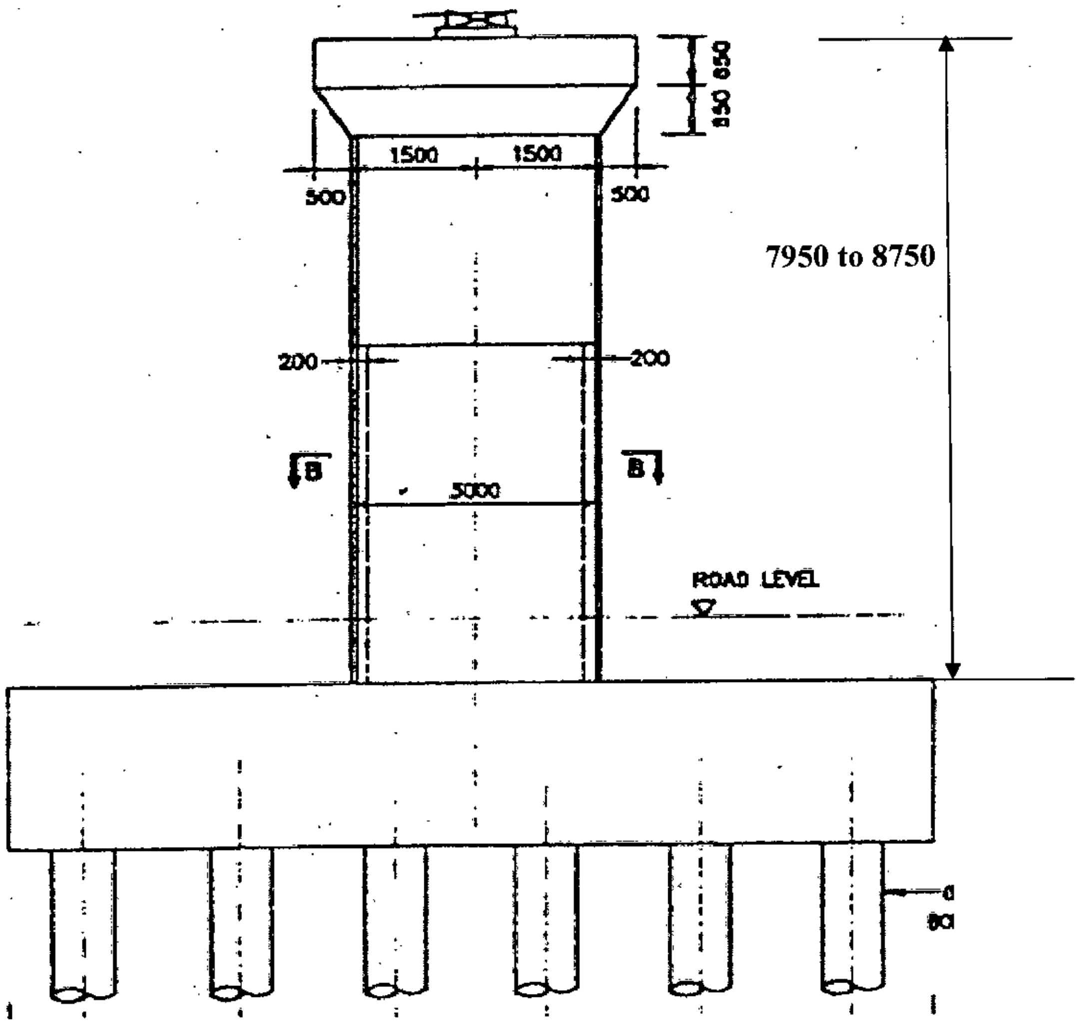


Fig 6.6 Elevation of Pier (P11 & P12) (All Dimensions in mm)

Table 6.1 Cross Sectional Properties of Pier Stem

Section Type	Area m^2	Moment of Inertia (longitudinal) m^4	Moment of Inertia (Transverse) m^4
Pier (P9, P10 & P13)	9.12	2.85	17.73
Pier (P11 & P12)	13.87	9.93	26.56

6.2.2 Deck Configuration

The deck is of pre-stressed post-tensioned concrete with twin box section. Several cross sections of deck have been used. The width of the box section is the same for all sections but the depth varies from 1.72m to 3m.

The typical deck section is shown in Fig 6.7.

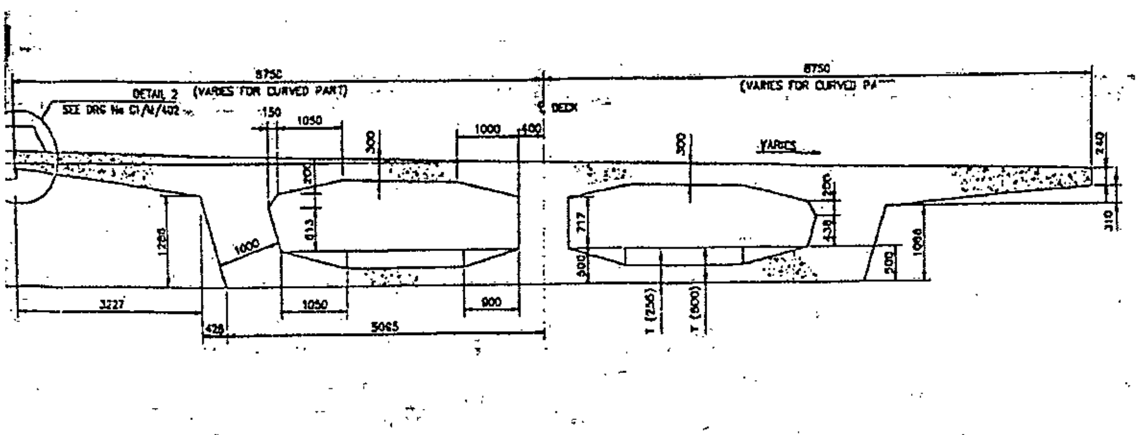


Fig 6.7 Typical Deck Section

6.2.3 Shock Transmission Unit

Shock Transmission Units (STU) has been used in Mohakhali Flyover as seismic protection. STUs are placed between the deck and the pier at various locations in an arrangement similar to that of Fig 6.2. During an earthquake all the STUs provide restrains in the longitudinal direction. The connections are in such a way that they are fixed in the transverse direction all the time. So the deck would be restrained at all support locations in case of a seismic event. As Mohakhali flyover has curvature, the shear keys present in each segment are aligned towards the fixed restraint of that module.

In total 42 STUs have been used for the whole structure. The requirements in terms of force and displacement capacities are comparable. So multiple number of devices with the same capacity can be used at different locations. Like the piers of the module in the middle have larger forces and four STUs are used in these locations where two are used at typical piers of other modules. Again, force capacity requirements at both ends of the modules are less and that is why only one STU is attached at these locations (Iqbal 2005).

The basic components of STU are,

- A piece of hydraulic cylinder
- Two bolted flanges that close the ends of the cylinder and
- A double headed piston rod that creates two chambers within cylinder.

When seismic event occurs, different piers share the seismic loads because STU lock up the bridge deck with the pier.

6.2.4 Finite Element Modeling of the Bridge

In this study the whole bridge structure is considered. It has 19 span and 18 piers. The North pre-cast segment has 9 spans and resting on 9 piers and an abutment, the middle cast-in-situ segment has 4 spans on 5 piers and the west pre-cast segment has 6 spans and resting on 6 piers and an abutment. In finite element modeling the following simplified assumptions have been made.

- Deck and piers are modeled using simple beam –column element
- Masses are lumped at nodes
- The variation of deck section along the length of the bridge is ignored.
- The bridge is founded on firm soil or rock, and the earthquake excitation is perfectly correlated at all of the supports

- The bridge piers are assumed to be rigidly fixed at the foundation level which means that the soil-structure interaction is ignored.
- The variation in pier section for P9, P10 and P13 is ignored.
- PMM Hinges have been used near base of pier.

Another straight bridge with same span length and same deck section and pier sections has been modeled. And it is assumed to be aligned in North-South direction. The response of the Mohakhali flyover is compared with that of the straight flyover.

6.2.5 Earthquake Motion used

As it is a comparative study of flyover response, widely used El Centro (Imperial Valley, May 1940), which is scaled down to a PGA value of 0.15g, has been applied here for both Mohakhali flyover and equivalent straight bridge. El Centro NS component is applied in north-south direction and EW component is applied in east-west direction of the bridges.

6.3 Results of Analysis and Discussion

The finite element models of Mohakhali Flyover and equivalent straight flyover described in section 6.2.4 are subjected to earthquake loads as mentioned in section 6.2.5. Also, Default PMM Hinge as mentioned in the Finite Element Software SAP2000, is used in the Mohakhali Flyover to evaluate whether the bridge will behave elastically due to applied seismic excitation. P-M2-M3 hinge yielding is based on the interaction of axial force and bending moments at the hinge location. Default hinge properties are incorporated in the program based on FEMA-356 (FEMA, 2000) criteria. As the hinge properties are not specified for the exact concrete and reinforcement properties of Mohakhali Flyover bridge piers and deck, the following results may not be considered representative of the actual situation. So any observations mentioned in the discussion are only tentative and needs more rigorous and exact analysis in future.

The results of this analysis are presented in the subsequent section. Responses of typical piers, P3 (one of the piers of North end pre-cast segment), P12 (one of the piers of cast-in-situ segment) and P14 (one of the piers of West end pre-cast segment). Hereafter straight bridge will denote the equivalent straight bridge of the Mohakhali Flyover.

6.3.1 Pier forces

The comparisons of pier forces are presented in Table 6.2. The forces include longitudinal and transverse shear, moment about longitudinal and transverse axis.

Table 6.2 Pier forces of Different Piers

	Pier 3		Pier 12		Pier 14	
Pier Forces	**Straight Flyover**	**Mohakhali Flyover**	**Straight Flyover**	**Mohakhali Flyover**	**Straight Flyover**	**Mohakhali Flyover**
Longitudinal Shear (kN)	4985	2849	29353	7108	4200	1026
Transverse Shear (kN)	125	50	7125	3768	5543	2610
Moment about longitudinal axis (kN-m)	560	196	57264	30000	53026	25780
Moment about transverse axis (kN-m)	15910	9022	91887	57924	19910	9950

From these results it is observed that straight bridge experience more shear in both transverse and longitudinal axis. Moments about longitudinal and transverse axis are also greater in straight bridge compared to Mohakhali Bridge.

6.3.2 Pier Top Responses (Displacement & Acceleration)

The comparison of absolute acceleration of different pier top has been presented here in Table 6.3 and 6.4. Time variation of Pier 12 top longitudinal displacement is presented in Fig 6.8. In the P3, no effect is observed for longitudinal acceleration. But in P14 both longitudinal and transverse accelerations increase. Pier top displacement is greater in straight bridge compared to Mohakhali Flyover.

Table 6.3 Pier Top Accelerations of Bridges

	Pier 3		Pier 12		Pier 14	
Relative Acceleration (g)	**Straight Flyover**	**Mohakhali Flyover**	**Straight Flyover**	**Mohakhali Flyover**	**Straight Flyover**	**Mohakhali Flyover**
Longitudinal	0.096	0.067	0.370	0.360	0.305	0.309
Transverse	0.002	0.001	0.087	0.109	0.261	0.301

Table 6.4 Pier Top Displacements of Bridges

	Pier 3		Pier 12		Pier 14	
Relative displacement (mm)	**Straight Flyover**	**Mohakhali Flyover**	**Straight Flyover**	**Mohakhali Flyover**	**Straight Flyover**	**Mohakhali Flyover**
Longitudinal	1.600	0.889	5.944	5.461	4.674	4.521
Transverse	0.015	0.005	2.311	1.194	4.445	2.184

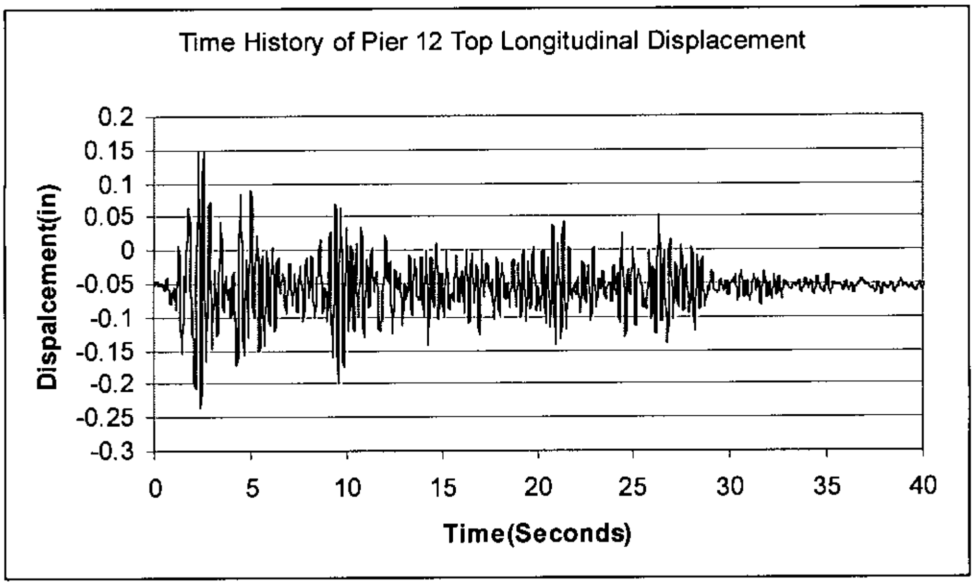


Fig 6.8 Time History of Pier 12 Top Longitudinal Displacement

Again, hinges provided at bottom of piers of Mohakhali Flyover shows that they were in elastic state. It means that, the bridge behaves elastically under the applied seismic load, and no plastic hinge will form. So, it may be stated that Mohakhali Flyover Bridge is safe under a seismic excitation with PGA of 0.15g.

Chapter 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

In this study, seismic response of curved flyover bridges is evaluated by varying bridge curvature as well as bridge pier stiffness. Analysis is performed on both non-isolated and isolated bridges. Also, as a case study, the seismic response of Mohakhali Flyover Bridge is analyzed and its response is compared with the response of a similar straight bridge. From these analysis results following conclusions may be made:

(i) In non-isolated bridges, pier top and midspan transverse displacement and acceleration decrease with increase in angle of curvature and pier top and midspan longitudinal displacement and acceleration decrease with curvature.

(ii) In non-isolated bridge, pier longitudinal shear increases but transverse shear decreases with increase in angle of curvature.

(iii) Pier moment about transverse axis increases and moment about longitudinal axis decrease with increase in angle of curvature.

(iv) Due to curvature significant torsional moment (about longitudinal axis) is generated at deck midspan, which may cause additional shear stress to the deck section. So, in designing curved flyover decks, increased torsional moment should be adequately taken into account.

(v) Deck Midspan moment about vertical axis reduces significantly with increased curvature angle.

(vi) Study also suggest that, stiffer pier reduce the pier longitudinal and transverse responses.

(vii) For isolated bridge, increase in curvature angle decreases the value of transverse displacement and acceleration of end pier and midspan.

(viii) Angle of Curvature of bridge has insignificant effect on the longitudinal displacement and acceleration of middle pier top.

(ix) For isolated bridges, end pier longitudinal shear and moment about transverse axis are found to increase and transverse shear and moment about longitudinal axis decrease with increase in curvature—in general.

(x) Curvature has been found to have no effect on middle pier forces in this limited study.

(xi) Isolator displacements increase in longitudinal direction and decrease in transverse direction with increase in curvature angle.

(xii) Isolation also significantly reduces the pier transverse shear.

(xiii) Isolation is effective in reducing different pier response like pier top displacement in both directions, pier top acceleration in both directions. But deck midspan displacement and acceleration increase due to isolation.

(xiv) From the case study of Mohakhali Flyover, it behaves elastically under the applied seismic load. No plastic hinge formed in the bridge. It can be stated that Mohakhali flyover is safe for the seismic load with PGA is 0.15g, which is the zoning coefficient of Dhaka according to BNBC (1993). Although the hinge properties provided may not represent the true behavior of the bridge.

7.2 Recommendation for Future Study

The following recommendations are put forward:

(i) Here, soil-structure interaction has been ignored. Effect of curvature may be studied considering soil-structure interaction.

(ii) More detailed analysis of different types of bridges may be undertaken

(iii) Effect of vertical ground acceleration may be included.

(iv) Only horizontally curved flyover bridges are analyzed here, effect of vertical curvature may be studied in future.

(v) Non linear modeling of bridge structure incorporating formation of plastic hinges for non-isolated bridges may be considered for future studies.

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