

## **DYNAMIC BEHAVIOR OF MULTI SPAN CONTINUOUS GIRDER BRIDGE WITH ISOLATION BEARINGS**

S.S. Roy<sup>1</sup> and S.R. Dash<sup>2</sup>

<sup>1</sup>University of Houston, Texas, USA.

<sup>2</sup>School of Infrastructure, IIT Bhubaneswar, Jatni, Pin - 752050, Odisha, India.  
e-mail: sroy6@uh.edu, srdash@iitbbs.ac.in

**ABSTRACT:** Bridges are one of the most important lifeline facilities, which are expected to be functional even after a catastrophic event like an earthquake. Prominent failure of bridges due to the complete collapse of piers has been observed in every major seismic event. In this paper, the dynamic behavior of a pre-stressed, continuous girder bridge is studied. Modal and nonlinear modal time history analysis is carried out for both non-isolated and seismically isolated bridge with lead rubber bearing (LRB). Simple elastic model is applied to model the various components of the bridge. Both abutments and pile caps are modeled as rigid members. First five mode shapes, modal characteristics, shifting of time periods, base shear, super structure and pier head displacement and deck acceleration are observed in this study. Changes in base shear are also investigated due to shifting of time period as per available codes of practice. The benefit that can be derived by the use of isolation bearing in the dynamic response of the bridge has been quantified in this paper with the incorporation of soil softening effect that may happen during earthquakes.

**KEYWORDS:** Base shear reduction; Depth of fixity; Isolation bearings; Pre-stressed Bridge; Seismic isolation; Spectral matching.

### **1 INTRODUCTION**

Bridges, in the highway, constitute an essential component in transportation lifeline system. They are certainly required to be in function during and after the catastrophic events like earthquakes. Bridges play a very vital role in smooth rescue and recovery operations post-disaster. Disruption of transportation traffic after an earthquake can cause severe loss of lives and may significantly affect the economy of the region. However, many recent earthquakes show that bridges are suffering from huge damages and causing significant economic losses. It was mainly short to medium span bridges with short piers those are primarily suffering from damages, as their fundamental period of vibration remains within the dominant period of vibration of the earthquake motion (M.N.

Haque *et al.* 2013). An example of bearing damage during Bhuj earthquake in 2001 is shown in Figure 1.

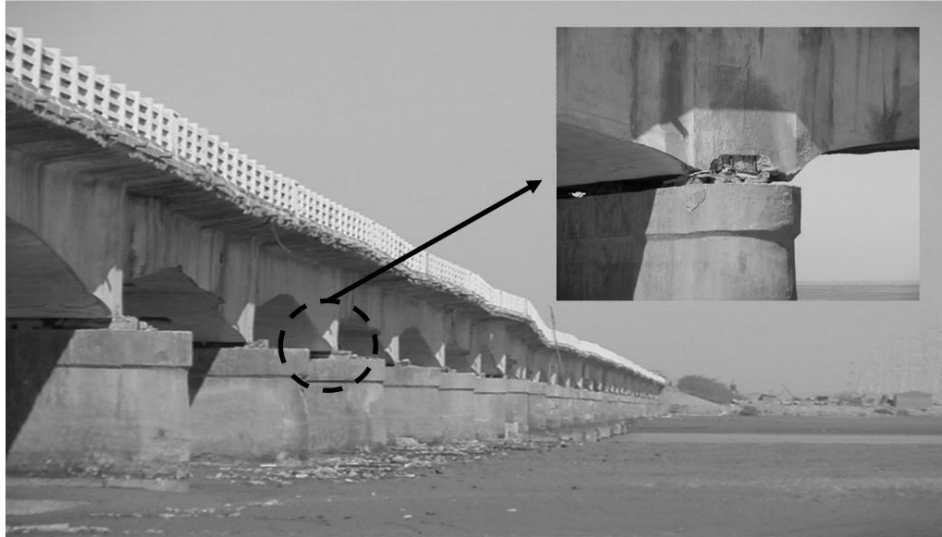


Figure 1. Crushing of the bearing and hence misalignment of the old Surajbadi bridge during 2001 Bhuj earthquake (Ref: Jain *et al.* 2001)

The traditional method of designing the earthquake resistance structure is to increase the strength and the energy absorbing capacity by its structural members (Jangid *et al.* 1998). However, restricting the transmission of earthquake forces and energy to the structure is another promising alternative, instead of increasing the strength and stiffness of the structure. To achieve the above alternative, seismic isolation strategy is adopted to isolate the superstructure from the substructure. The seismic isolation fundamentally differs from conventional seismic design approach by adopting two techniques, such as (a) lengthening the fundamental period of the structure thus attracting less seismic forces, and (b) use of material at suitable locations that are capable of high hysteretic-energy dissipation. Isolation devices (isolation bearings) are installed to isolate the superstructure from substructure at the location of bridge bearings. Figure 2 schematically shows the reduction of spectral acceleration demand on a structure whose fundamental time period has significantly increased due to isolation. In conventional approaches, it is accepted that considerable earthquake forces and energy will be transferred to structures from ground. However, in seismic isolation the basic aim is to substantially reduce the earthquake energy and force getting transferred to the superstructure, thus by reducing the inelastic deformation. Accordingly, by using isolation system, the super-structure and sub-structure are un-coupled in terms of seismic forces. Hence this is often used as an effective alternative approach to protect the

bridge structure against damage from earthquake hazards, rather resisting it.

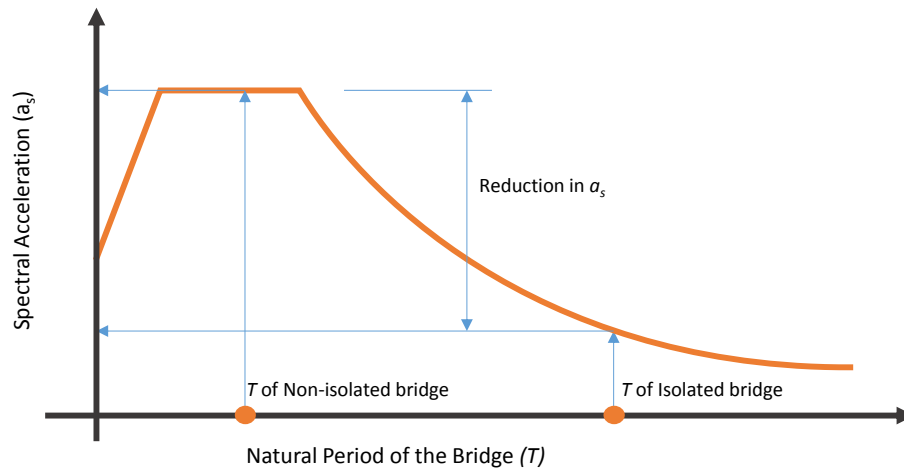


Figure 2. A typical representation of shifting of natural period ( $T$ ) and reduction in spectral acceleration ( $a_s$ ) due to use of isolation bearings in bridges

A variety of isolation devices such as laminated elastomeric bearings (L-EB), lead rubber bearings (LRB) and friction pendulum bearings (F-PB) are commonly used in practice. The technical details and suitability of these bearings are described in next section. Among these bearings, lead rubber bearings are most reliable and extensively used worldwide. The main philosophy of seismic isolation is to shift the natural frequency of the structure out of the region of dominant frequency of earthquake motion. Many researchers like Robinson (1982), Ghobarah and Ali (1988), Turkington (1989), Jangid (2004) and Haque and Bhuiyan (2010, 2013) have shown the reliability and effectiveness of LRB, mainly for short to medium span continuous slab/girder bridges. However, these studies do not look at the change in soil properties during the earthquake. It is well understood that during an earthquake, soft to medium soil may experience cyclic strength degradation during earthquakes, which may affect the dynamic characteristics of the structure as a whole. The degraded strength of soil leads to various depth of fixity.

Hence, the present study is carried out to investigate the effectiveness of isolation bearing (LRB) over non-isolation bearing (Pot-PTFE) for a pre-stressed, continuous, multi-span bridge with consideration of soil strength degradation (in terms of various depth of fixity). To observe the shift in period of natural vibration, mode shapes and modal characteristics a modal analysis has been carried out for the same bridge with and without isolation bearings. Also, to study the change in base shear, deck acceleration, deck displacement and top pier force for both the cases for a real earthquake scenario, a nonlinear

modal time history analysis is carried out, which is presented in this paper.

## **2 REVIEW OF ISOLATION BEARINGS FOR BRIDGES**

Depending upon the configuration, behavior, and material, the major isolation bearings can be categorized into (a) Laminated elastomeric bearing (LEB), (b) Lead rubber bearing (LRB) and (c) Friction pendulum system (FPS). The schematic representation of these three isolators is shown in Figure 3. Brief technical descriptions of these bearings are given below.

### **2.1 Laminated Elastomeric Bearing (LEB)**

Laminated elastomeric bearings are composed of alternating layers of natural, or synthetic, rubber bonded to intermediate steel shim plates at high temperature and pressure by vulcanization as shown in Figure 3(a). A rubber cover is also a part of the bearing to protect the internal rubber layers and steel plates from environmental degradation due to ozone attack and corrosion, respectively. Elastomeric bearings can be categorized as (1) low-damping natural (polyisoprene) or synthetic (Poly-Chloroprene) rubber bearing (LDRB); and (2) high-damping rubber bearing (HDRB).

### **2.2 Lead Rubber Bearing (LRB)**

Lead rubber bearing was first invented in New Zealand in 1975 by Bill Robinson. Since then lead rubber bearings have been more popular and extensively used as seismic isolation device throughout the world including Japan and United states. It mainly consists of rubber layers, reinforcing steel shims and lead plug as shown in Figure 3(b). Rubber layers support the structural weight, provide lateral flexibility and bring back the bearing to the original position. Steel shims provide vertical stiffness to support the superstructure's weight and act as a constraint to deformation of lead core. Lead plug provides the source of energy dissipation, thus reducing the overall displacement. The force deformation behavior of this bearing remains elastic under the service load and inelastic during an earthquake when the lead case deforms beyond its elastic limits. The horizontal force-deformation relationship of a lead-rubber bearing is characterized using a bi-linear model.

### **2.3 Friction Pendulum Bearing (FPB)**

Friction pendulum bearing is a modified type of sliding-type bearing (*i.e.*, LEB and LRB). The sliding-type bearings mainly lack restoring capacity, once deformed. To overcome this drawback, the friction pendulum bearing (FPB) is developed from sliding-type bearings by introducing a spherical sliding surface to provide restoring stiffness, while the friction between the sliding interfaces helps in dissipating energy. This results in lengthening the structure's fundamental time period like LRB and HDRB, along with some additional

advantages. The FPB consists of stainless steel concave surface, an articulated slider, and a cover plate. The slider is coated with a self-lubricating liner such as Teflon. During an earthquake, the articulated slider slides along the concave surface, causing the superstructure to move with a gentle pendulum motion. The swing motion of the FPB is shown in Figure 3(c).

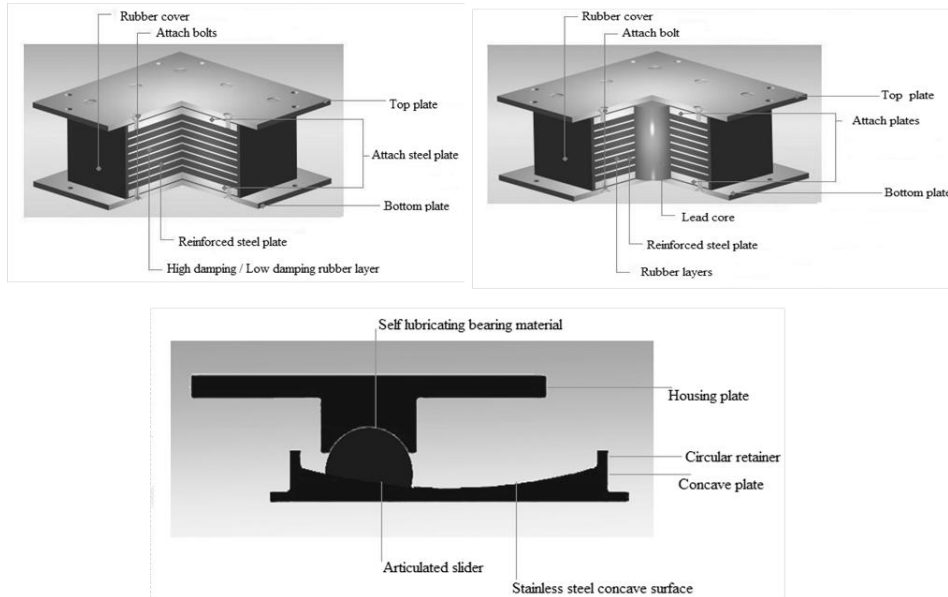


Figure 3. (a) Components of Low / High damping rubber bearing; (b) Components of lead rubber bearing; (c) Components of friction pendulum bearing

### 3 BRIDGE STRUCTURAL MODEL AND CASE STUDY

#### 3.1 Physical model

For the present study, a typical three-span, pre-stressed continuous bridge with 29.0 m span length and 15.2 m wide is selected as shown in Figure 4(a). The superstructure consists of 220 mm thick deck slab with 65 mm thick asphalt wearing a coat and supported on five numbers of pre-stressed continuous I-girders. The substructure consists of two rigid abutments at two ends and two intermediate circular piers. The longitudinal and transverse sectional elevation of sub-structure is shown in Figure 4(b) and 4(c) respectively. The substructure is supported by pile foundations. The foundation under each pier consists of 8 numbers of bored cast-in-situ piles of diameter 1.2 m each. The geometry and dimensions of different parts of the bridge is given in Table 1 while; Table 2 gives the properties of the materials used in the bridge. Originally the bridge was equipped with Pot-PTFE and pin bearings. Arrangement and type of bearings under each span is shown in Figure 4(d). In this study, the conventional bearings of the bridge are replaced with lead rubber bearings

(LRB) to make it an isolated bridge from the non-isolated bridge. The dynamic responses of both non-isolated and isolated bridges are studied further.

*Table 1.* Geometry and dimensions of different elements of the bridge  
(see Figure 4)

Components	Dimensions (m)
Number of columns in each pier location	1
Diameter of each pier	2.5
Clear height of each pier	5.4
Plan dimension of pier head	13.2 x 3.0
Cross-sectional dimension of pier head	3.0 x 2.5
Numbers of piles in each pier	8
Diameter of each pile ( $D$ )	1.2
As-built length of each pile	25.0
Calculated depth of fixity for piles in original bridge	10.65 ( $\sim 9D$ )

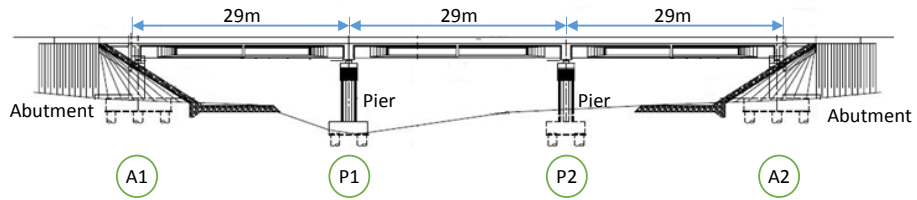
*Table 2.* Material properties considered for bridge modeling

Properties	Values
Characteristic compressive strength of concrete used in Substructure and pile foundations ( $f_{ck}$ , N/mm <sup>2</sup> )	35
Characteristic compressive strength of concrete used in Superstructure ( $f_{ck}$ , N/mm <sup>2</sup> )	50
Young's modulus of steel ( $E_s$ , N/mm <sup>2</sup> )	200,000
Young's modulus of concrete ( $E_c$ , N/mm <sup>2</sup> )	5000 $\sqrt{f_{ck}}$

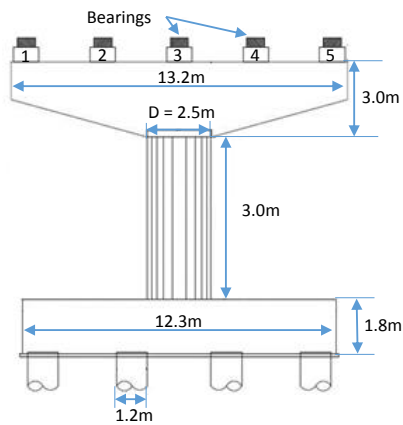
### 3.2 Analytical model

A 3D finite element model of the bridge is developed using SAP2000-V14® (2014) finite element software as shown in Figure 5. Modal and Nonlinear time history analysis is carried out for the pre-stressed bridge model using both lead rubber isolation bearing and existing Pot- PTFE bearing. For this analytical study, the following assumptions are made.

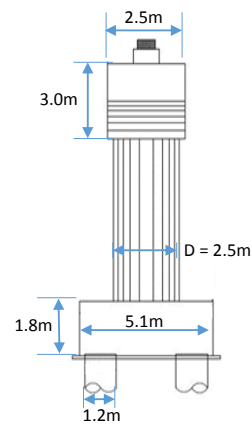
- The primary plastic behavior of the structural system is expected to occur at the isolation bearings.
- Bridge superstructure and piers are assumed to remain in the elastic state during earthquake excitation. This is the conventional assumption made for isolated structures (Jangid *et al.*, 1998).
- The superstructure of the bridge is considered to be resting upon lead rubber bearing both in abutments and piers.
- The bridge foundations (Piles) are assumed to be rigidly fixed at the depth of fixity level (as per IS 2911 Part-2, 2010).
- The abutments are considered to be rigid and fixed to the ground, similar to the modeling carried out by Kartoum *et al.*, 1992.



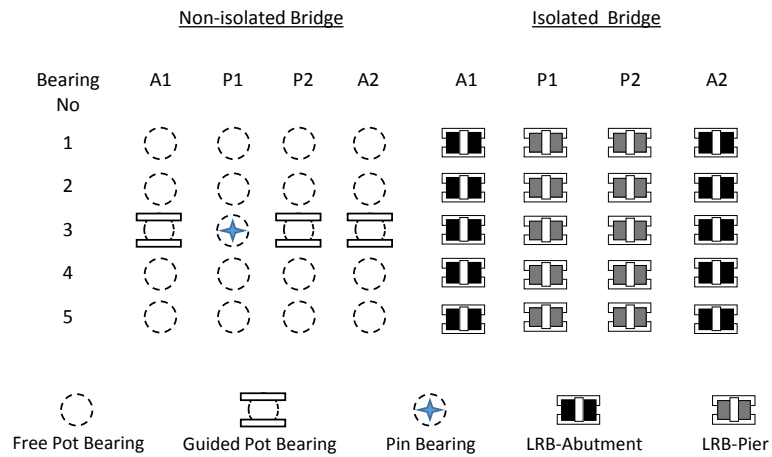
(a) Longitudinal section of bridge



(b) Longitudinal elevation of pier and pier head



(c) Transverse elevation of pier and pier head



(d) Arrangement of Bearings

Figure 4. General layout of bridge and arrangement of bearings for non-isolated and isolated bridge model

8 Dynamic behavior of multi span continuous girder bridge with isolation bearings

- The stiffness contribution by non-structural elements like kerbs, railings, parapet walls and wearing coat are considered to be negligible (Jangid *et al.* 1998).
- The lead rubber isolation bearings are considered to be isotropic with hysteresis behavior as discussed later.
- The Pot-PTFE bearings are considered as elastic springs at the pier and abutment locations.

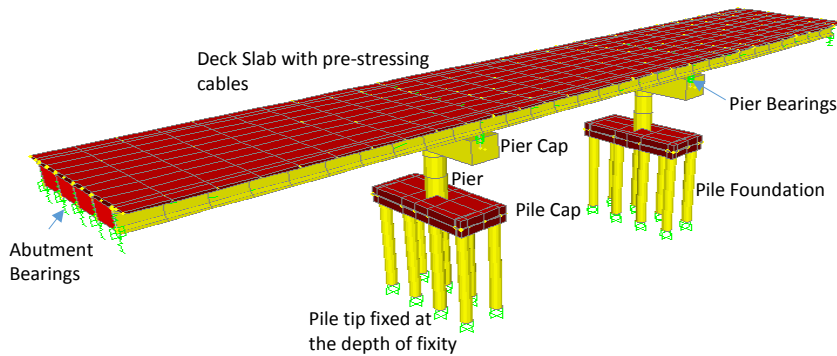


Figure 5. Pre-stressed continuous bridge model in SAP2000 V14®

Based on the above assumptions, the pre-stressed girders in the superstructure, pier caps, pier shafts, pile caps and pile foundations are modeled with simple elastic frame elements with required shapes and cross sections. The pile cap and pier cap has been made rigid. The soil properties available for the bridge location are considered to evaluate the depth of fixity as per IS 2911 Part-2 (2010). For the considered bridge site, the depth of fixity was found to be 10.65m ( $\sim 9D$ ) from the ground level. The piles are modeled till depth of fixity and made fixed at the tip as shown in Figure 5. This model does not require further representation of soil around the pile.

In order to describe the mechanical behavior of isolation bearing, the bilinear model specified in AASTHO 2010 is adopted, as shown in Figure 6. As per this model, lead rubber isolation bearing is characterized by initial stiffness  $K_u$ , post yield stiffness  $K_d$ , effective stiffness  $K_{eff}$  and the yield strength of the bearing  $F_y$  as shown in the figure below. Due to the significant difference in weight supported by the seismic isolators placed on piers and abutments, two different configurations are considered for the isolation system. Table 3 provides the bilinear model parameters used in this study for abutment and pier locations.

Lead rubber bearing is chosen based on the availability of expansion gap (200 mm for this case) in the original bridge and the total vertical load acting on the individual bearings. The vertical load including the seismic load on each bearing is estimated to be  $\sim 2000$  KN. Lead rubber bearings fulfilling above criteria are adopted from FIP industrial lead rubber bearing series catalogue

(FIP, 2015). Hence the seismic isolator bearings chosen here has axial load bearing capacity of 2290KN. In this study, these parameters are assigned to the model as backbone properties of a rubber link element in SAP2000-V14. The dimensions and parameters of LRB used in the analysis are described in Table.3. For the Pot-PTFE bearing, the force deformation behavior is modeled as linear elastic with  $K_{eff} = 151\text{kN/mm}$  in  $U_1$  (Longitudinal) and  $U_2$  (Transverse) direction and assigned as a linear link element in the model.

## 4 ANALYSIS

In this study two analyses have been performed, (a) modal analysis and (b) nonlinear modal time history analysis. The detailed analysis procedure for both of the analyses is presented below.

### 4.1 Modal analysis

To study the difference in dynamic behavior of non-isolated and isolated bridge, modal analysis is carried out to determine major initial modes of vibration of the bridge. Eigen vector modal analysis has been carried out for the unstressed structure to determine the un-damped free vibration mode shapes and frequency. In this study, first twelve modes of vibration of the structure are examined. Here modes are found in order of increasing distance of frequency from the shift. This continues until the cutoff is reached and requested number of modes is found. Parameters like frequency shift, cutoff frequency and convergence parameter are set to zero, zero and  $10^{-9}$  respectively. First five predominant natural periods and mode shapes for non-isolated bridge with pot-PTFE bearing and isolated bridge with lead rubber bearings are reported in this paper. The effective stiffness ( $K_{eff}$ ) of the isolated bearing as per Figure 6 is used for the modal analysis.

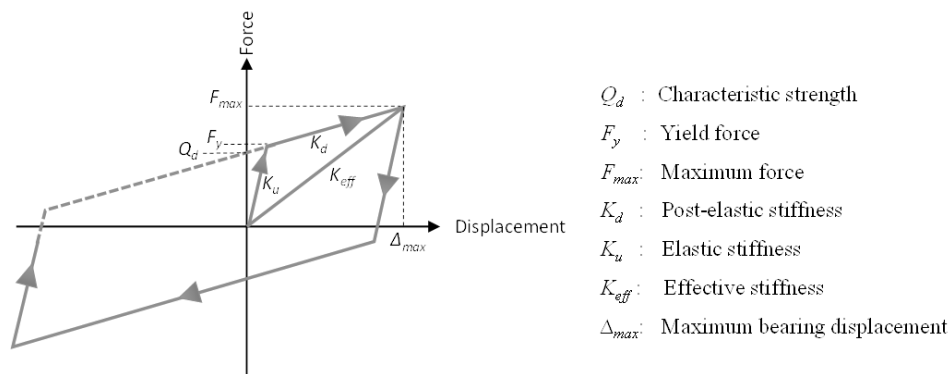


Figure 6. Bilinear model for LRB

*Table 3.* Parameters for the linear model of Pot-PTFE and bilinear model for the LRB

Parameters of LRB on Piers (LRB-Pier)	Values
Initial stiffness, $K_H$ (KN/mm)	14.33
Effective stiffness, $K_{eff}$ (KN/mm)	1.49
Yield strength, $F_y$ (KN)	172
Post yield ratio	0.16
Parameters of LRB on Abutments (LRB-Abutment)	
Initial stiffness, $K_H$ (KN/mm)	10.5
Effective stiffness $K_{eff}$ (KN/mm)	1.08
Yield strength, $F_y$ (KN)	126
Post yield ratio	0.16
Parameters of Pot – PTFE Bearings	
$K_{eff}$ in global X and Y directions (kN/mm)	151
$K_{eff}$ in global Z direction (kN/mm)	1510

#### 4.2 Nonlinear modal time history analysis

This method is extremely efficient and is widely used to find the response of structural systems to seismic loading, which is primarily linear elastic but has a limited number of predefined nonlinear elements. For this nonlinear modal time history model, all nonlinearity is restricted to the Link/Support elements. Equations that govern the dynamic responses of the bridge can be derived by considering the equilibrium of all forces acting on it using the D'Alembert's principle. The dynamic equilibrium equations of a linear elastic structure with predefined nonlinear link/support elements subjected to an arbitrary load can be written as:

$$M\ddot{u}(t) + C\dot{u}(t) + Ku(t) = r(t) - [r_N(t) - K_N u(t)] \quad (1)$$

$$K = K_L + K_N \quad (2)$$

Where:  $K_L$  is stiffness for the linear degrees of freedom of all the linear link/support elements,

$K_N$  is the linear effective-stiffness matrix for all the nonlinear degrees of freedom,

$C$  is proportional damping matrix,

$M$  is diagonal mass matrix,

$r_N$  is vector of forces from the nonlinear degrees of freedom in the link/support elements,

$r$  is vector of applied loads, and

$u$ ,  $\dot{u}$  and  $\ddot{u}$  is relative displacements, velocities, and accelerations with respect to the ground.

Furthermore, the Newton-Raphson iteration procedure consisting of corrective

unbalanced forces is employed within each time step until equilibrium condition is achieved.

#### **4.2.1 Spectrally matched time history**

In the absence of historical ground motion data in the region, the generation of artificial ground motion histories has become a common practice, to use in time history analysis. Various methods exist to generate artificial ground motion histories from existing earthquake records, from which the widely used methods include: (a) scaling the time history with a uniform factor (Shome and Cornell, 1998) or (b) scaling the ground motion response spectrum to match a particular design spectrum for the given period of the structure (Kunnath *et.al*, 2006) which is called as spectral matching of Time Histories. Spectral matching involves a time/frequency domain modification of an acceleration time history to make it compatible with a user-specified target spectrum. Abrahamson (1992) wrote the first computer code for spectral matching called *RSP Match*, which has been subsequently updated and widely-used. Spectral matching has the advantage of significantly reducing variability in the computed response because the time histories are often very close to the target response spectrum and give reliability on the calculated responses of the structure for design. However, the selection of the number of earthquake records to be used for analysis is a major source of concern for designers. UBC (1997) and IBC (2000) suggests designing the structure using the maximum responses obtained from the analyses if three records are taken, and the average should be considered if seven records are taken. The ISO 19901-2 (2004) proposes that a minimum of four time histories that comply with the site conditions and faulting style of the location can be used for the analysis.

In this study, total 16 time histories of different past earthquakes are considered to match the target response spectrum. Target response spectrum was obtained from IS: 1893 for hard rock strata, considering the structure falls into zone five with an importance factor of 1.5. Out of all time histories 6 earthquake records such as, Bhuj (2001), Friuli (1976), Loma Prieta (1989), Northridge (1994), Trinidad (1974) and Kocaeli (1999) records found to give the best mean that fits the target spectrum in the time period ranges of consideration. These 6 time histories obtained after spectral matching are used in further nonlinear modal time history analysis and the bridge responses are studied. The mean matched spectrum obtained from these six time histories is plotted in Figure 7 over the IS: 1893 target response spectrum.

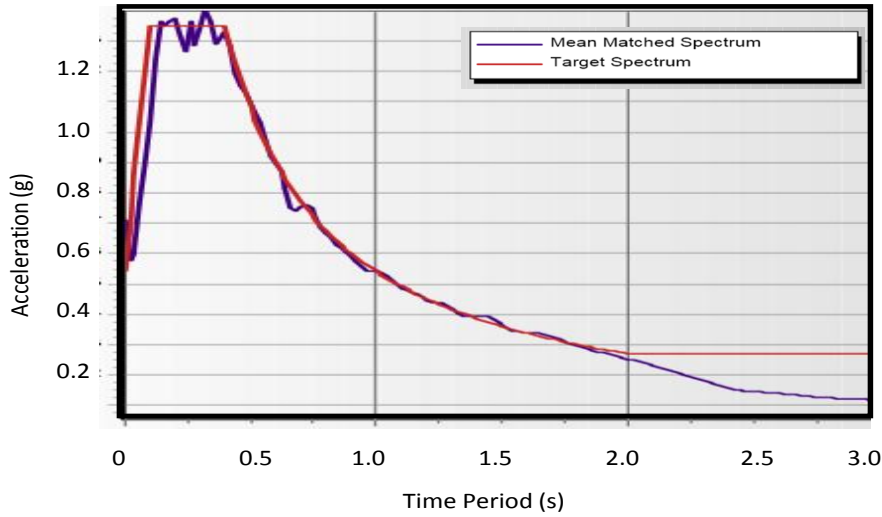


Figure 7. Mean matched spectrum used to produce target ground motions

**4.2.2 Verification of force-deformation behavior of Pot-PTFE and LRB**

To verify the force- deformation characteristics of both Pot-PTFE and lead rubber bearing, one of the adopted ground motions is employed to perform nonlinear modal time history analysis. From the output response, it can be observed that the Pot-PTFE bearing behaves linearly while the response of LRB is bilinear in nature (Figure 8), as defined in the model. Thus, the model is verified to work as expected in terms of non-linear performance of bearings. Further, analyses for both non-isolated and isolated bridge models are carried out for the matched time histories.

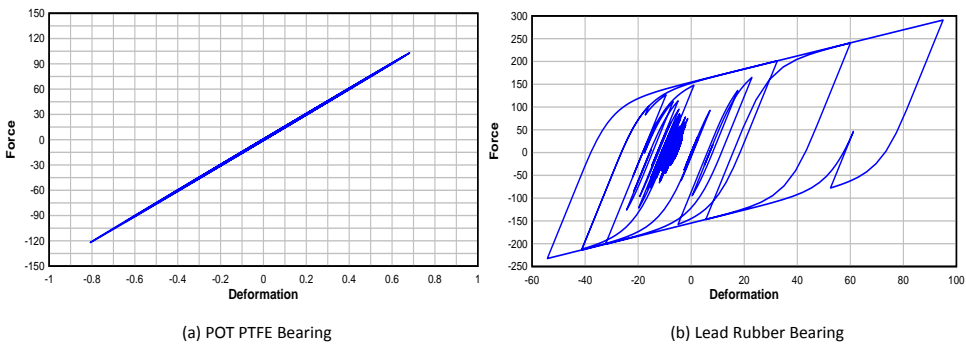


Figure 8. Force – deformation behavior of Pot-PTFE and lead rubber bearing under Loma Prieta earthquake ground motion

**4.2.3 Effect of depth of fixity study**

The depth of fixity considered for the pile foundation for the present soil condition is at about 10.65m (~9D, D = Diameter of the pile). However, the soil

can undergo cyclic degradation during earthquake shaking, if the soil is soft clay or liquefiable sand. Hence, the depth of fixity for pile foundations may vary during dynamic loading (like in earthquakes). Therefore, the same numerical model is analyzed further with varying Depth of fixity. Also, to see the effect of depth of fixity for various field situations (Very less for the hard soil to quite high for soft soil, this analysis considers four depth of fixities, such as; 6D, 9D, 12D, and 15D. The responses of isolated and non-isolated bridges have been monitored for the non-linear time history analysis, and the results are presented.

## 5 RESULTS AND DISCUSSION

### 5.1 Modal analysis

The modal analysis results for the first five predominant modes for both non-isolated and isolated bridge models are given in Table 4. The modal mass participation ratio for both the bridge models is also shown in Table 5 and Table 6 respectively. The mode shapes and time period of initial three predominant modes are also presented in Figure 9.

Table 4. Modal data for non-isolated and isolated bridge

Mode Number	Frequency (Hz)		Time period (Sec)		Mode characteristics	
	Non-isolated bridge	Isolated bridge	Non-isolated bridge	Isolated bridge	Non-isolated bridge	Isolated bridge
Mode - 1	2.1905	0.621495	0.456514	1.610077	Translational with rotation	Translational with rotation
Mode - 2	2.9443	0.630344	0.339638	1.586463	Rotational	Longitudinal
Mode - 3	3.0972	0.770154	0.322872	1.299602	Rotational	Rotational
Mode - 4	3.5207	2.104878	0.284038	0.475685	Longitudinal	Translational
Mode - 5	4.0588	2.290421	0.246379	0.436601	Vertical with rotation	Vertical with rotation

Table 5. Modal mass participation ratio for non-isolated bridge

Mode Number	Period	$U_x$	$U_y$	$U_z$	$R_x$	$R_y$	$R_z$
<i>Unit less</i>	<i>Sec</i>	<i>Unit less</i>	<i>Unit less</i>	<i>Unit less</i>	<i>Unit less</i>	<i>Unit less</i>	<i>Unit less</i>
Mode - 1	0.45651	7.94E-11	0.94	6.84E-11	0.29	2.58E-11	0.76
Mode - 2	0.33964	6.6E-11	0.003286	1.7E-08	0.55	1.41E-08	0.00265
Mode - 3	0.32287	2.05E-09	1.17E-10	1.17E-13	1.5E-09	2.55E-09	0.04157

Mode - 4	0.28404	0.45	1.69E-10	2.87E-11	1.66E-10	0.02882	1.01E-7
Mode - 5	0.24638	2.44E-11	6.5E-09	0.000322	6.37E-09	0.000257	5.77E-9

As expected, the isolation technique significantly shifts the time period and frequency in all cases. The natural period is observed to be higher for LRB (isolated) bridge model as compared to that of the Pot-PTFE type (non-isolated) bridge model. For example, the first period of the bridge is 0.46s which has increased to 1.61s, causing a 3.53 times increase in time period because of isolator bearings. Similarly, the second period and third period has also been increased by 4.66 times and 4.03 times respectively for LRB model. The modal characteristics are also different for both the bridge models. It can be observed from the Figure 9 that the major torsional modes of vibration are avoided in the isolated bridge that can be a major advantage of using isolation bearings.

Table 6. Modal mass participation ratio for isolated bridge

Mode Number	Period	U <sub>x</sub>	U <sub>y</sub>	U <sub>z</sub>	R <sub>x</sub>	R <sub>y</sub>	R <sub>z</sub>
<i>Unit less</i>	<i>Sec</i>	<i>Unit less</i>	<i>Unit less</i>	<i>Unit less</i>	<i>Unit less</i>	<i>Unit less</i>	<i>Unit less</i>
Mode - 1	1.61008	4.24E-09	0.58	4.77E-12	0.000938	6.56E-13	0.47
Mode - 2	1.58646	0.59	3.81E-09	4.4E-13	9.2E-13	0.000111	3.46E-9
Mode - 3	1.29960	1.17E-11	2.23E-13	2.62E-16	8.07E-13	1.27E-12	0.15
Mode - 4	0.47569	5.2E-09	0.12	5.34E-09	0.00155	2.79E-09	0.09321
Mode - 5	0.43660	2.41E-13	3.72E-11	5.92E-07	5.16E-12	4.69E-07	6.95E-9

## 5.2 Base shear reduction as per seismic coefficient method

As per IS 1893(Part -3): 2014, with all other terms remaining unchanged, the base shear is directly proportional to the spectral acceleration coefficient ( $S_d/g$ ). Hence, from design response spectrum it implies a 74% reduction in base shear for the 1<sup>st</sup> mode of vibration when LRB isolation system is used as compared to Pot-PTFE bridge, as illustrated in Figure 10.

## 5.3 Fundamental time period for various depth of fixity cases

Figure 11 gives the change in fundamental time period of the bridge for various depth of fixity consideration. For all the cases, isolated bridge time period is quite higher than that of the non-isolated bridge. Deeper the depth of fixity, the time period gets higher for both the bridges due to additional flexibility obtained due to longer depth of pile. However, in the isolated bridge, as the major flexibility is at the bearings, the effect of depth of fixity is not significant in the lengthening of the fundamental time period of the structure. For example,

for depth of fixity changing from 6D to 15D, there is 47% increase in time period in the non-isolated bridge, whereas for the isolated bridge this value is just 5%.

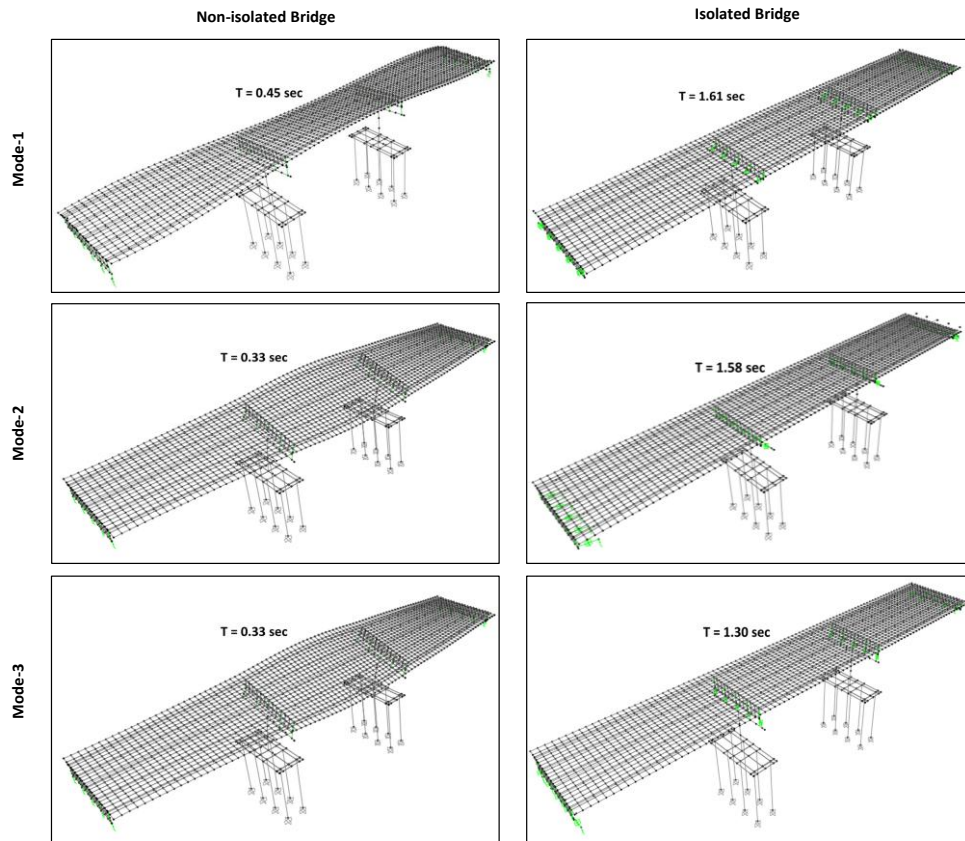


Figure 9. First 3 modes and time period of vibration for existing and isolated models

## 5.4 Nonlinear modal time history analysis

### 5.4.1 Substructure performance

Seismic response of both isolated and the non-isolated bridge is determined by nonlinear modal time history analysis under six spectrally matched earthquake ground motions. All earthquake ground motions are applied in the longitudinal direction of the bridge as a typical scenario. The substructure responses of both the bridges for all earthquake ground motions are tabulated in Table 7. It can be seen from the table that for all input earthquake ground motions, the substructure responses of isolated structure, such as resultant base shear and pier head displacement, significantly reduced as compared to the non-isolated structure. It can also be seen that from all considered cases, the maximum

reduction in base shear goes up to 78.3% which is the case for Bhuj (2001) earthquake ground motion. The minimum reduction of 69.3% is achieved in case of Friuli ground motion. And, the mean reduction in base shear is about 75% from all the spectrally matched time history analysis. Most importantly, the results obtained from nonlinear modal time history analysis closely agree with the base shear that has been calculated from the seismic coefficient method as illustrated in Figure 10. It can also be inferred that the bridge response is ground motion dependent, even though all the ground motions are spectrally matched. For this reason, various codes suggest multiple ground motions to be considered while carrying out analysis. The other important aspect that shall be highlighted is the pier head displacement. The pier head displacement gets reduced considerably in case of isolated structure, as major deformation and energy absorption is being claimed by the bearings. From Table 7, it can be observed that a maximum reduction of 56.7% and mean reduction of 46% is achieved for pier head displacement by using isolation bearings instead of non-isolated bearings, for the considered bridge model.

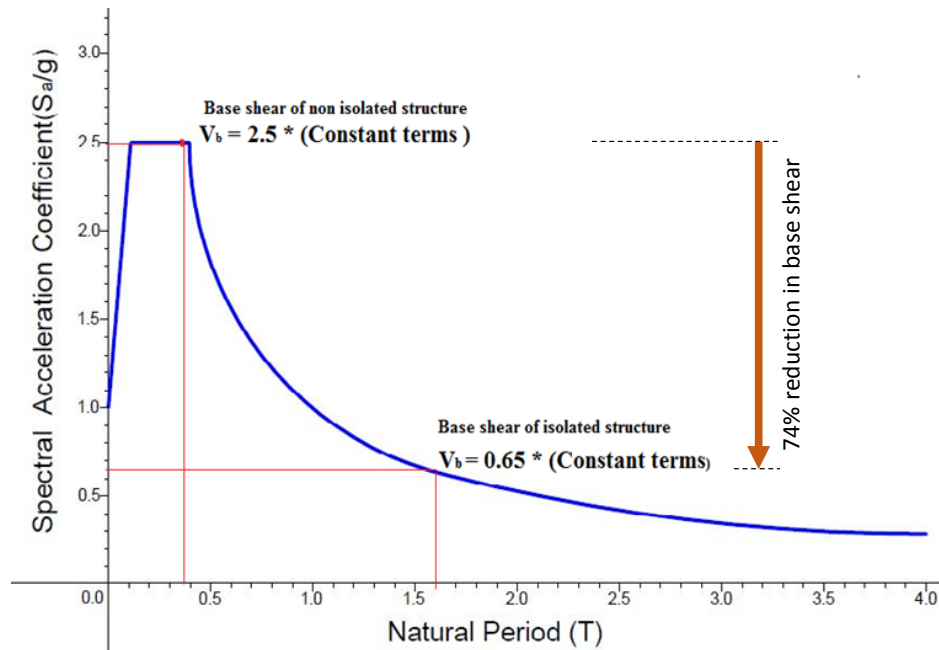


Figure 10. Illustration of change of base shear as per IS 1893(Part -3): 2014 for a typical case

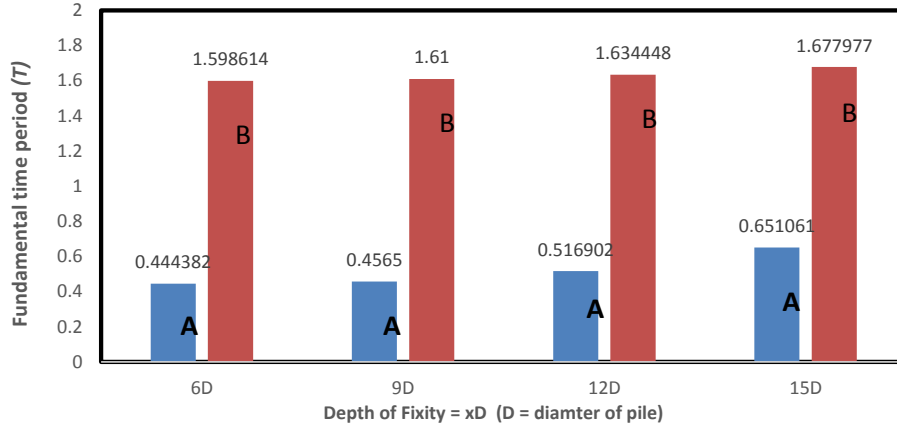


Figure 11. The fundamental period for the non-isolated bridge (A) and isolated bridge (B) for various depth of fixity

Table 7. Base shear and pier head displacement of both isolated and non-isolated bridges

Earthquake	Maximum Absolute Base Shear (kN)			Maximum Absolute Pier Head Displacement (mm)		
	Without isolation	With isolation	% Reduction	Without isolation	With isolation	% Reduction
Bhuj	32228	7999	75.2	30.1	22.2	26.2
Friuli	30026	9230	69.3	50.8	24.8	51.2
Loma Prieta	35957	9592	73.3	47.2	25.7	45.5
Trinidad	36418	7920	78.3	50.5	21.9	56.7
Kocaeli	32782	8541	73.9	39.1	23.5	39.9
Northridge	30652	7057	77.0	46.2	20.9	54.8
	Reduction in base shear due to isolation: Maximum = 78.3% Minimum = 69.3% Mean = 75%			Reduction in pier head displacement due to isolation: Maximum = 56.7% Minimum = 26.2% Mean = 46%		

The limit of maximum and minimum % reduction in base shear for various depths of fixity is plotted in Figure 12. It can be seen from the figure that with an increase in depth of fixity, the % reduction in base shear decreases, however, the decrease is not significant with respect to depth of fixity. Hence, it can fairly be said that due to isolation bearing the base shear can be reduced at least in the range of 50-80%. Similarly, the maximum reduction in pier head displacement for various depths of fixity is plotted in Figure 13. It can be seen that the reduction in pier head displacement is high for intermediate depth of fixities and less in very low and very high depth of fixities.

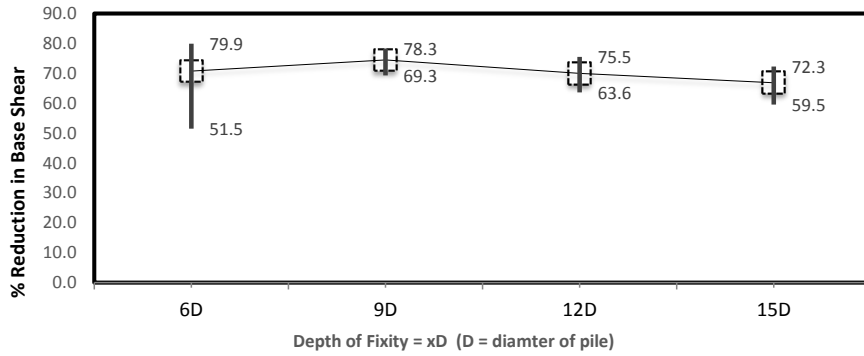


Figure 12. Base shear reduction for various depths of fixities

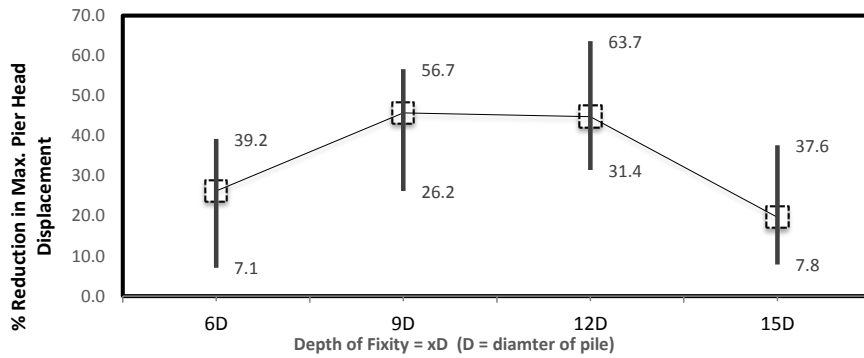


Figure 13. Maximum pier head displacement reduction for various depths of fixities

### 5.4.2 Superstructure performance

The deck displacement and deck acceleration from the time history analysis is tabulated in Table 8. It can be seen from the table that the benefit that may be achieved by using isolation bearing in terms of deck acceleration. A maximum reduction of 72.9% for deck acceleration has been achieved for Trinidad ground motion history. From the table, it can also be seen that the displacement of the superstructure, for the isolated bridge in all cases is more than that of the non-isolated case, which results in less force transformation and decoupling of superstructure from the substructure. It can be referred from the above study that with an increase in the time period the displacement of the LRB also increases, thus transferring less earthquake force to the bridge system, and absorbing large amount of seismic energy through bearing hysteresis. However, the relative displacement of LRB has a practical limitation, for which while designing the isolation system, a compromise should be made between transmitted earthquake forces and relative bearing displacement (i.e., permissible maximum expansion gap). As the bearing is the intermediate element between substructure and superstructure, it undergoes large

deformation if of isolated type. Hence, as expected, the bridge superstructure displacement has largely increased in isolated bridge as compared to the non-isolated bridge. In cases, seismic arresters/restrainers can also be provided to stop the bearing displacing beyond acceptable limits.

The ground motion input and deck acceleration response for one of the ground motions are plotted in Figure 14. In this plot, Loma Prieta (1989) ground motion is considered as input ground motion. It can be seen that for the isolated bridge with LRB the acceleration response is highly reduced in high frequency zone with respect to non-isolated bridge with Pot-PTFE bearing.

*Table 8.* Deck displacement and deck acceleration of both isolated and non-isolated bridge

<i>Earthquake</i>	Maximum Absolute Deck Displacement (mm)			Maximum Absolute Deck Acceleration ( $\text{mm/s}^2$ )		
	<i>Without isolation</i>	<i>With isolation</i>	<i>% Increase</i>	<i>Without isolation</i>	<i>With isolation</i>	<i>% Reduction</i>
Bhuj	16.7	75.3	349.9	9429.4	2903.0	69.2
Friuli	15.1	97.0	541.2	9062.1	3002.5	66.9
Loma-prieta	18.4	96.6	425.6	10124.1	3365.4	66.8
Trinidad	19.0	77.7	307.8	10410.2	2821.6	72.9
Kocaeli	17.0	116.6	587.7	9847.3	5593.5	43.2
Northridge	16.5	99.4	503.0	8873.6	3324.7	62.5
	Maximum increase in deck displacement = 587.7%			Maximum reduction in deck acceleration = 72.9%		
	Minimum increase in deck displacement = 307.8%			Minimum reduction in deck acceleration = 62.5%		

The time history analysis results for an increase in deck displacement and reduction in deck acceleration for different depth of fixities are plotted in Figure 15 and 16, respectively. From Figure 15 it is evident that the deck displacement increases with increase in the depth of fixity, with respect to that of the non-isolated bridge. Similarly, as the system flexibility increases due to increase in depth of fixity, even for the non-isolated bridge, the percentage reduction in deck acceleration for the isolated bridge with respect to that of non-isolated bridge decreases, as can be seen in Figure 16.

20 Dynamic behavior of multi span continuous girder bridge with isolation bearings

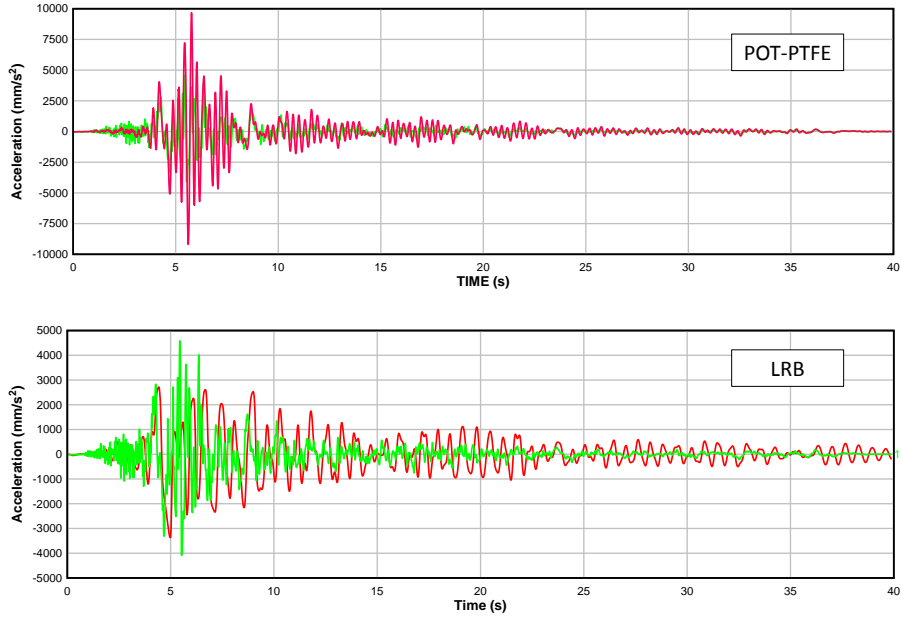


Figure 14. Deck acceleration response time history for both non-isolated and isolated bridge (Note: Green line = input response, Red line = output response)

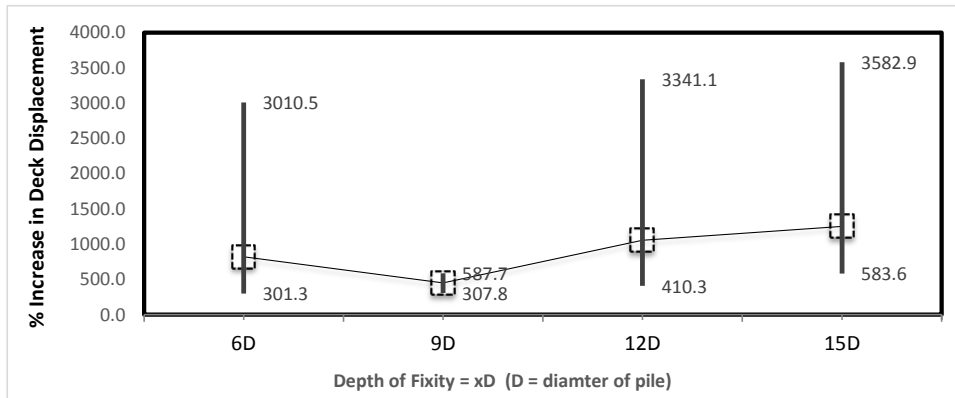


Figure 15. Increase in deck displacement for various depths of fixities

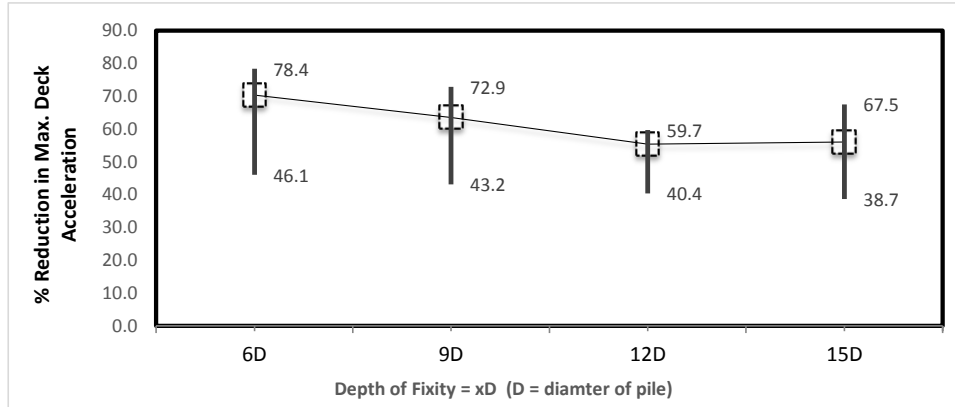


Figure 16. Reduction in maximum deck acceleration for various depths of fixities

## 6 CONCLUSION

In this paper, a method for finding out the earthquake response of a multi-span continuous pre-stressed bridge seismically isolated by LRB subjected to earthquake excitation in the longitudinal direction is presented. The modal analysis and nonlinear modal time history analysis with Pot-PTFE bearing and isolation bearings have been carried out in the present study.

For the particular model studied in this paper, the fundamental time period of the isolated bridge could increase up to 3.53 times with respect to that of the non-isolated bridge. The base shear reduction based on Indian seismic code provisions suggests a reduction of about 74% (hand calculation) can be achieved due to this elongation of the fundamental period of the bridge owing to use of isolation bearings. This base shear reduction value has been later found to be closer to the value obtained from non-linear modal time history analysis carried out for non-isolated and isolated bridge models using spectrally matched earthquake records.

The analysis also shows that the bridge response parameters greatly depend on bearing displacement which must be included in effective design of LRB. Moreover, the deck displacement also shows the importance of adequate gap between deck and dirt wall or wing wall to increase the effectiveness of this isolation system for new bridges. Adequate expansion gap may be considered in line with the maximum bearing displacement values.

For a bridge with Pot-PTFE bearing the modes are mainly governed by bridge substructure stiffness, while, for the bridge with LRB the modes are governed by the stiffness of the isolators. As far as practicable, the effective stiffness of the isolator shall be kept smaller. However, the load carrying capacity of it has to be maintained to make an economical and effective design.

For the selected six pairs of earthquake ground motion, it is observed that the responses like base shear, deck acceleration, deck displacement and pier head

displacement significantly reduces due to isolation bearing. The response is also ground motion sensitive; hence designers should use a sufficient number of spectrally matched ground motion to carry out the analysis and choose right response values added to other load effects for design. The responses do also differ for various depths of fixity scenarios, which suggest that if the soil conditions at the bridge site warrant stiffness degradation during an earthquake, the response of the bridge has to be determined for various depth of fixities and highest response to be chosen for the design.

## REFERENCES

- [1] AASTHO. Guide Specifications for Seismic Isolation Design, 3rd Edition, July 2010
- [2] Abrahamson N.A. (1992). Non-stationary spectral matching seismological research letter 63(1), 30-30.
- [3] Constantinou, M. C., Kartoum, A., Reinhorn, A. M., and Bradford, P., 1992. "Sliding isolation system for bridges: Experimental study." *Earthquake Spectra*, 8, 321–344.
- [4] FIP (2015) "Lead Rubber Bearings Series LRB" catalogue, FIP INDUSTRIALE UK Ltd. Cambridge UK.
- [5] Ghobarah A, Ali HM. Design of Base- Isolated Highway Bridges. Proceedings of Ninth World Conference on Earthquake Engineering. August 2-9, 1988, Tokyo -Kyoto, JAPAN (Vol.V)
- [6] Ghobarah A, Ali HM. Seismic performance of highway bridges, *Engineering Structures*, 10(1988) 157-66.
- [7] Haque MN, Bhuiyan AR, Alam MJ. Seismic response analysis of base isolated highway bridge: Effectiveness of using laminated rubber bearings. IABSE-JSCE Joint Conference on Advances in Bridge Engineering-II, Dhaka, 2010.
- [8] IBC, 2000, International building code. International Code Council, USA
- [9] IS 1893 (Part 1): 2002. Criteria for Earthquake Resistant Design of Structures, Bureau of Indian Standards (BIS), New Delhi
- [10] IS 1893 (Part 3): 2014. Criteria for Earthquake Resistant Design of Structures, Bureau of Indian Standards (BIS), New Delhi
- [11] IS 2911 (Part 1/Sec 2): 2010. Design and Construction of Pile Foundations — Code of Practice, Bureau of Indian Standards (BIS), New Delhi
- [12] ISO 19901-2:2004. Petroleum and natural gas industries - Specific requirements for offshore structures - Part 2: Seismic design procedures and criteria.
- [13] Jain S.K., Murty C.V.R., Dayal U., Arlekar J.N. and Chaubey S.K., (2001). "Learning from Earthquakes: A field report on structural and geotechnical damages sustained during the 26 January 2001 M7.9 Bhuj Earthquake in Western India", National Information Center of Earthquake Engineering (NICEE), IIT Kanpur.
- [14] Jangid RS. Seismic response of isolated bridges, *Journal of Bridge Engineering*, 9(2004) 156-66.
- [15] Kunnath, S. K., Larson, L., and Miranda, E. (2006). 'Modeling considerations in probabilistic performance-based seismic evaluation: Case Study of the I-880 Viaduct' *Earthquake Eng. Struct. Dyn.* 35(1), 57–75
- [16] M.N. Haque, A.R. Bhuiyan. Seismic Response of multi span Highway Bridge: Effectiveness of using isolation system. *Asian Journal of Civil Engineering (BHRC) VOL.14.NO.5* (2013). Pages 707-718
- [17] N.P. Toagaokar, R.S. Jangid. Seismic Response of Bridges with Sliding Isolation Devices. *ISSET Journal of Earthquake Technology*, Paper No. 373, Vol 35, No 1-3, March-Sept, 1998, pp 9 – 27

- [18] Robinson, W.H. (1982). "Lead-rubber hysteretic bearings suitable for protecting Structures during earthquakes", *Earthquake Engineering and Structural Dynamics*, Vol. 10, 593-604.
- [19] SAP2000, 2014, *Integrated Software for Structural Analysis and Design*. V14, Berkeley, California, USA, CSI Inc
- [20] Shome, N., Cornell, C. A., Bazzurro, P., and Carballo, J. "Earthquake, Records, and Nonlinear MDOF Responses", *Earthquake Spectra*, (1998), 14(3), 469-500.
- [21] Turkington, D. H., et al. (1989). "A design method for bridges on lead-rubber bearings." *J. Struct. Engineering.*, ASCE, 115(12), 3017-3030.
- [22] UBC (1997) "Uniform building code." *International Conference of Building Officials*, Whittier, CA