SEISMIC ASSESSMENT OF BRIDGES ON THE BASIS OF IS: 1893 (Part 3)

A DISSERTATION

Submitted in partial fulfilment of the

requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

EARTHQUAKE ENGINEERING

(With specialization in Structural Dynamics)

By

BALLA TARAKA MALLESWARA RAO

DEPARTMENT OF EARTHQUAKE ENGINEERING

INDIAN INSTITUTE OF TECHNOLOGY ROORKEE

ROORKEE – 247667 (INDIA)

MAY 2016

CANDIDATE'S DECLARATION

I hereby declare that work which is being presented in this dissertation report entitled, "**Seismic Assessment of Bridges on the Basis of IS: 1893 (Part 3)",** in the partial fulfilment of the requirements for the award of the degree of **MASTER OF TECHNOLOGY** in **EARTHQUAKE ENGINEERING**, with specialization in **STRUCTURAL DYNAMICS**, submitted to the Department of Earthquake Engineering**,** Indian Institute of Technology Roorkee, is an authentic record of my own work carried during the period from July 2015 to May 2016 under the supervision of **Dr. R. N. Dubey,** Assistant Professor, Department of Earthquake Engineering & **Dr. S. K. Thakkar,** Retired Professor, Department of Earthquake Engineering, IIT Roorkee.

The matter embodied in this report has not been submitted by me for the award of any other degree or diploma.

Place: Roorkee

Date: (**B. T. Malleswara Rao)**

CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of our knowledge.

Dr. R. N. Dubey
Assistant Professor
Dept. of Earthquake Engineering,
Engineering, IIT Roorke.

Dr. S. K. Thakkar
Retired Professor
Dept. of Earthquake
IIT Roorkee.

ACKNOWLEDGEMENTS

I wish to express my deep sense of gratitude and indebtedness to my learned and enlightened guide **Dr. R.N. Dubey**, Assistant Professor, Department of Earthquake Engineering, Indian Institute of Technology, Roorkee, for his instinctive and careful guidance and continued encouragement in the completion of this dissertation.

I am also thankful to **Dr. S.K. Thakkar**, Retired Professor, Department of Earthquake Engineering, IIT Roorkee, who helped me a lot by his expert guidance and giving his valuable suggestions for successful completion of my thesis.

I am extremely grateful to my family and friends for their endless support and encouragements, and for always believing that I can succeed in my endeavours.

I am highly indebted to all other members of the Department for their continuous support, valuable suggestions and encouragement throughout this report. At last but not the least, I am also thankful to all members, who directly or indirectly helped me in bringing this report in its present form.

(**Balla Taraka Malleswara Rao)**

ABSTRACT

Seismic assessment of bridges is very important to know the structural deficiencies in the existing bridges. In the recent past earthquakes, structural deficiencies in many of the existing bridges have been observed, because most of them were constructed before the advancement of seismic code. This study deals with seismic evaluation of two bridge substructures to know its seismic performance. The finite element models of two bridge substructures have been developed in SAP2000. The response spectrum analysis, nonlinear static analysis and capacity spectrum methods have been employed to determine the base shear, capacity and demand respectively. The response spectrum analysis and the capacity spectrum method have been done by using the design response spectrum given in IS: 1893-1984, IS: 1893 (Part 3)-2014. The seismic deficiency has been obtained under MCE condition as per IS: 1893 (Part 3)-2014. The reinforced concrete (RC) jacketing and carbon fiber reinforced polymer (CFRP) jacketing have been employed for improving the seismic performance of the bridge piers. The retrofitted bridge piers have been modelled in SAP2000 for evaluation of its performance by using the pushover analysis and capacity spectrum method.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 General

Bridges are the most important structures for development of transport system in the country. The construction of bridge need huge investment which, warrant that special consideration should be paid at the time of design and construction of bridge so that it gives service throughout its design life period without any major deficiencies. It should provide reliability against the natural disasters like earthquakes etc. This is of utmost important for safety of the passengers and goods. Bridges are the lifeline structures and need to remain functional after the design earthquake. Past earthquakes illustrated that bridges are vulnerable to severe damage or collapse during moderate to strong ground motions.

The history of seismic design is used to replicate the history of damaging earthquakes. It is certainly true that after each and every major damaging earthquakes, in which bridges are seriously damaged, the code of practice has to be modified accordingly. There is a strong correlation in every part of the world between occurrence of major earthquakes and advances in seismic design. Each earthquake tested the knowledge of the day and where it has been found deficient; the lessons learned have led to improvements in the state of art practice.

Earthquakes of particular significance for their impact on bridge design include Anchorage 1964, Niigata 1964, Inangahua 1968, San Fernando 1971, Guatemala 1975, Fruili 1976, Edgecumbe 1987, Loma Prieta 1989, Phillippines 1990, Costa Rica 1991, Okurshiri 1993, Nothridge 1994, Kobe 1995, and Hanshin Awaji 1995. In the last decade researchers and practitioners have been able to improve the state of art substantially and major code revisions have taken place, or are in the process of being modified, in such areas as design philosophy, performance criteria, ground motion characterization, geotechnical design, inelastic analysis and capacity design procedures for concrete and steel structures. The basic aim of engineers and experts is to estimate real amount of seismic force which the particular structure is subjected to and provide guidelines for safety and to reduce economic loss and damage. The different seismically prone countries have their own guidelines for design such as

India, Europe, Japan, USA, New Zealand etc. In our country, basically there are three codes/standards which deal with the seismic design of bridges. These are: IS 1893, IRC 6:2014 and existing Bridge Rules of Indian Railways.

The seismic performance of bridges is very important during the occurrence of earthquakes as it play a key role in rescue and relief, firefighting, medical services, transporting emergency goods to affected people. Unlike building structures, bridges have little or no redundancy and failure of one or more important structural elements namely piers and bearings may lead to their failure or collapse of bridge. The bridges constructed before 1970 were damaged in the past earthquakes occurred in India are Dharmashala earthquake in 1986, Uttarkashi earthquake in 1991, Killari earthquake in 1993, Jabalpur earthquake in 1997, Chamoli earthquake in 1999 and Bhuj earthquake in 2011. After 1970, the Indian standards for earthquake resistant design was considerably strengthened by including the ductile detailing and bridge structural behaviour has been more accurately evaluated.

The bridges constructed before the advancement of seismic code need to be checked whether they are seismically safe or not by performing the linear and non-linear static or dynamic analyses. An earthquake ground motion in the seismically active regions induces very high lateral forces in the structures and if the designer considers the entire earthquake force, it may lead to very expensive design. As the occurrence of an earthquake is a rare phenomenon, so the structures are designed to resist the less ground accelerations by considering the response reduction factors. Earthquakes identify structural weaknesses and damage is concentrated at piers and connections of the bridges. So, it is very important to know that the seismic performance of bridges after particular ground motions due to earthquakes. Thus, bridges have to be retrofitted if they are damaged during the earthquake. Retrofitting and strengthening make the structure increase its design capacity and also upgrades it to meet the requirements of current design procedures. Linear elastic procedures are sufficient for the bridges within the elastic limits even if they are subjected to ground accelerations. Linear elastic procedures cannot predict the failure mechanism of the structures beyond the elastic limits (first yielding). So the elastic procedures are insufficient to perform the seismic assessment of bridges in general. Non-linear procedures are very efficient in seismic assessment of bridges and for evaluation of retrofitting techniques.

1.2 Objectives of Dissertation

The main objectives of dissertation are

1. To develop the 3D finite element models of bridge substructure using SAP2000.
2. To carry out the response spectrum analysis, non-linear static analysis and capacity spectrum methods for determine the base shear, capacity and demand respectively.
3. The response spectrum analysis and the capacity spectrum method have been done by using the design response spectrum given in IS: 1893-1984, IS: 1893 (Part 3)-2014.
4. To compare the demand and capacity of the bridge pier to find its seismic deficiency.
5. To retrofit the bridge pier with reinforced concrete (RC) jacket and carbon fiber reinforced polymer (CFRP) jacket for seismic deficiency.

1.3 Organization of Dissertation

Chapter 1 “Introduction”

This chapter gives the outline for necessity of seismic assessment of bridges and objectives of the study.

Chapter 2 “Literature Review”

This chapter presents the literature available on seismic assessment of bridges by using various methods, and various retrofitting techniques used to improve the seismic performance.

Chapter 3 “Modelling of Bridge Substructures”

This chapter highlights the modelling of bridge pier, modelling of material properties and detailing of the pier.

Chapter 4 “Modal and Response Spectrum Analysis”

Dynamic characteristics of the bridge substructure and the results obtained after response spectrum analysis are presented in this chapter.

Chapter 5 “Non-linear Static Analysis”

Performance and seismic deficiency of the structure evaluated from the non-linear static analysis and capacity spectrum method are presented in this chapter.

Chapter 6 “Retrofitting of Bridge Piers”

This chapter briefly discuss about various retrofitting techniques used to improve the seismic performance of the bridge substructure. Modelling and performance evaluation of the retrofitted bridge pier is presented in this chapter.

Chapter 7 “Results and Discussions”

This chapter presents the detailed results of the non-linear static analysis of the existing, RC jacketed and CFRP jacketed bridge pier, and compares the performance of the existing, RC jacketed and CFRP jacketed bridge pier.

Chapter 8 “Conclusions”

This chapter summarizes the conclusion drawn from the results.

CHAPTER 2

LITERATURE REVIEW

Ballard et al. (1999) performed a three dimensional pushover analysis to estimate the capacity of the Lake Washington Ship Canal Bridge frame 10. After getting the target displacement, the extents of columns retrofits were decided based on the locations and type of failure. To validate the results obtained from the non-linear static analysis (pushover), non-linear time history analysis was conducted.

Chiorean (2003) conducted pushover analysis to study the collapse behaviour of three span prestressed reinforced concrete bridges. The bridge is built in the north-eastern of Portugal over Alva river. In that study author used line elements approach to model the bridge and plastic hinges in elements has been modelled as distributed plasticity in terms of plastic zone and concentrated plasticity in terms of plastic hinge. The pushover analysis was done by using the capacity spectrum method in accordance with EC8 provisions and displacement coefficient method in accordance with FEMA 273. The evaluation of performance levels has been done by using the performance criteria given in EC8 and FEMA 273.

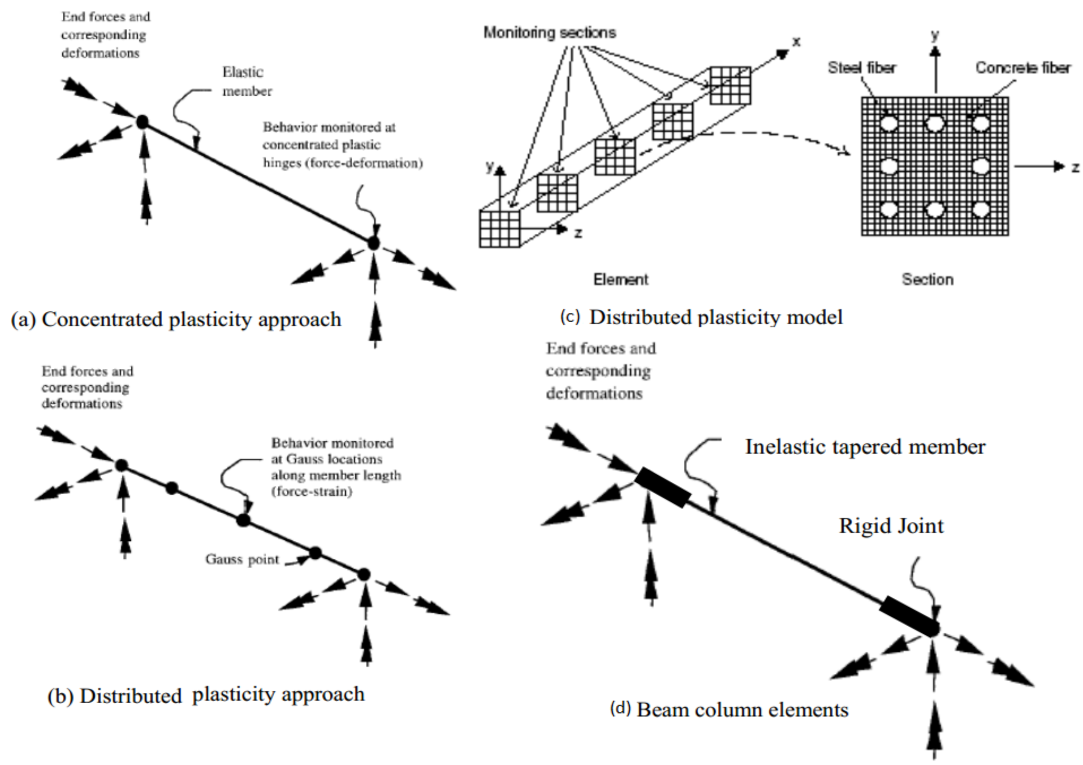


Figure 2.1 Line element and plasticity models (Chiorean, 2003)

Bignell et al. (2005) conducted pushover analysis to assess the seismic vulnerability of wall pier supporting the highway bridges on southern Illinois priority routes. There were four basic types of bridge structures existed such as multi-column supported pier, wall supported pier, culvert, and single span with the multi-column supports and wall supported bridge piers. A total of 90 wall pier bridge models were selected and three dimensional finite element models were created in ABAQUS to carry out the pushover analysis in both the longitudinal and transverse directions to assess the seismic vulnerability.

Ruiz et al. (2008) in their work in Mexico had considered few bridges of Mexico City that were constructed before 1970. Those bridges were designed with non-seismic provisions, or by using codes with seismic specifications that did not satisfy the recommendations of present-day knowledge. These conditions make it necessary to assess the seismic capacity of the existent bridges, especially for those bridges located in the more seismic active areas of the country and evaluating the seismic capacity of bridges in Mexico. Based on above analyses an assessment methodology is proposed. The procedure is divided in two phases: the first one is a screening method for the preliminary assessment of seismic vulnerability of bridges, which allows the identification of bridges in a specific region for a detailed evaluation. The second stage is a displacement based assessment procedure, applicable to the bridges in the worst conditions of vulnerability according to the screening procedure. They were using the capacity spectrum method for obtaining performance of bridges and the results are validated with the past damage observed in the few bridges during Northridge earthquake. Eventually, the above procedure was implemented on other bridges and their performance was estimated.

Rahai et al. (2010) in their study, two models of existing prestressed concrete bridges constructed by cantilever method (frame and continuity) have been selected. The first model is a three span continuous bridge supported on skew piers. The skew piers are fixed to the deck by prestressed cables and for evaluating the exact non-linear behaviour plastic hinges are assumed at top and bottom of piers. The second model is a seven span continuous box girder bridge which is supported on piers via bearings and for evaluating non-linear behaviour, the hinges are assumed at bottom. The performance evaluation of second bridge, the pier having largest span and tallest pier is considered. Then, the seismic evaluation of bridges is done by non-linear static analysis through capacity spectrum method and displacement coefficient method for the three types of hysteresis behaviour. Then they compare the results obtained from both methods and conclude that, the results obtained from the displacement coefficient method are more acceptable and conservative.

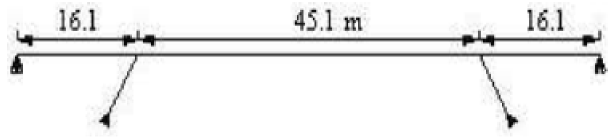


Figure 2.2 View of first model (Rahai et al., 2010)

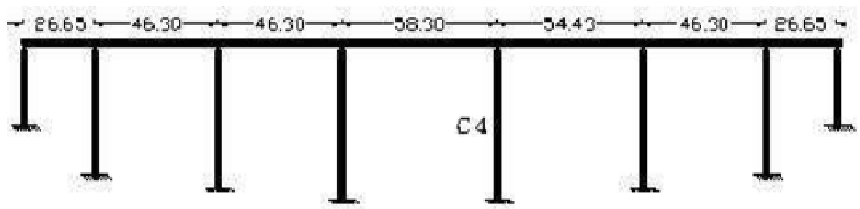


Figure 2.3 View of second model

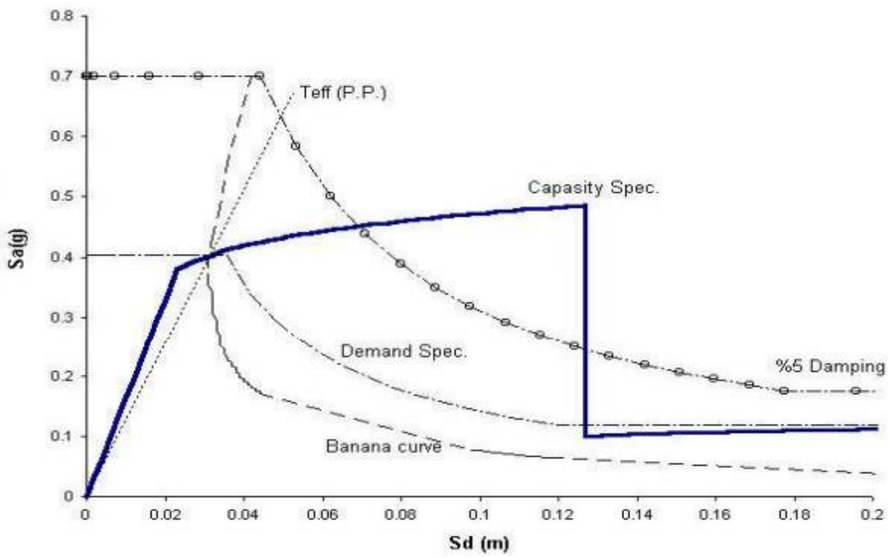


Figure 2.4 Determination of performance point of C4 column

Asae et al. (2013) investigated the accuracy of FRP confined reinforced concrete column modelled in finite element software tool, SAP2000. The results obtained after the analysis are validated with the experimental results. FRP confined concrete circular and rectangular columns are involved with the lumped plasticity models in SAP2000. Extracted results are used to compare the moment curvature and PMM interaction obtained from SAP2000 and manual calculations based on Silvia et al. The behavior of the confined fiber reinforced polymer concrete has been considered from the study of Lam and Teng (2003). The non-linear static pushover analysis is performed to know the capacity curve based on SAP2000. The study has shown that the difference between the analytical and experimental results is found to be within the acceptable criteria.

Billah et al. (2014) studied the seismic vulnerability of three column bridge bent, constructed before the 1960's, when the seismic provisions were not fully developed. The study suggested some retrofitting techniques (such as concrete jacketing, carbon fiber reinforced polymer (CFRP), steel jacketing, and engineered cementitious composite (ECC) jacketing) for improving the seismic performance. The bridge substructure considered in the study is shown below in Fig. 2.1.

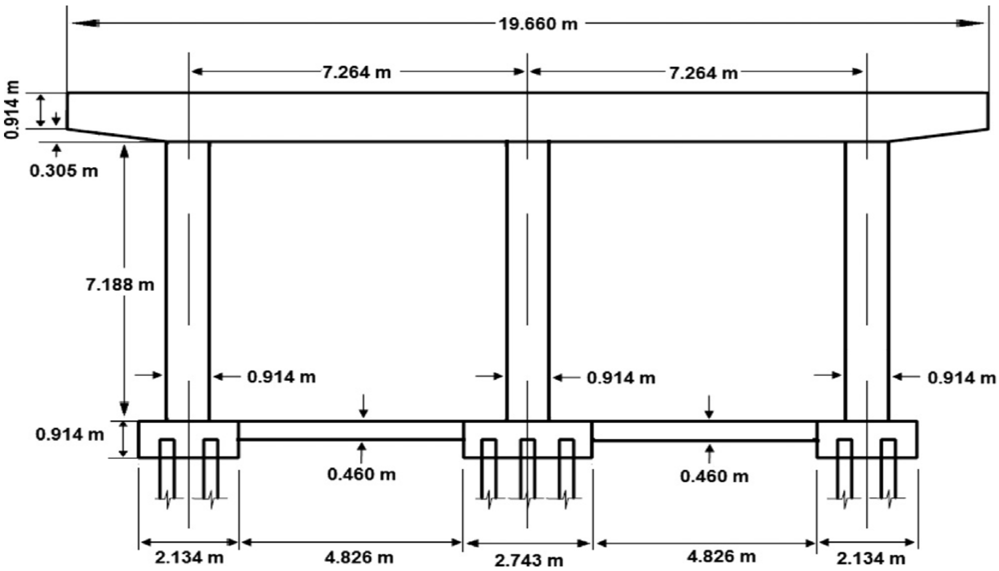
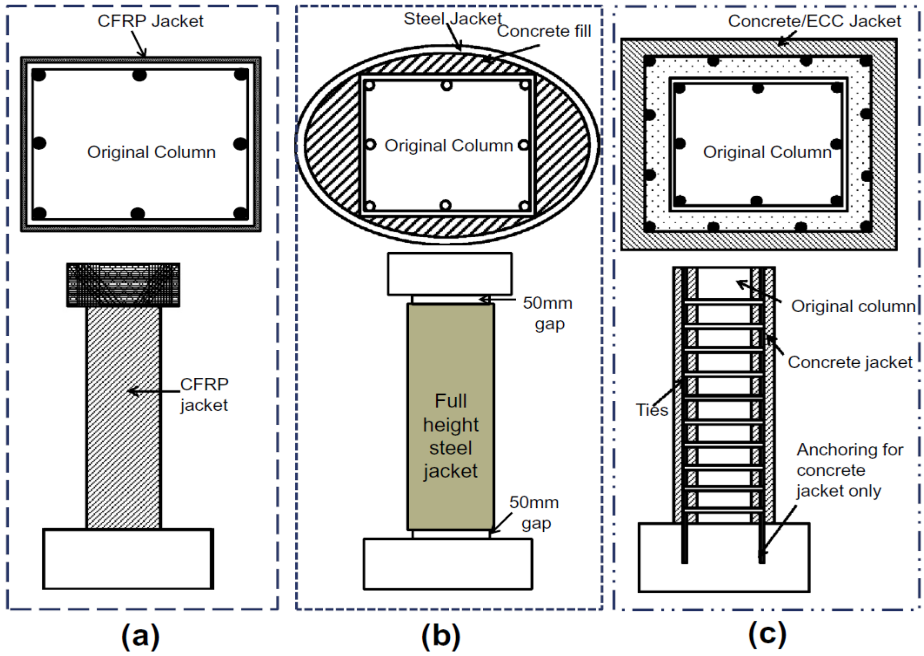


Figure 2.5 South Temple bridge bent dimensions (Pantelides et al., 2002)

It was observed that the bridge was seismically deficient according to the present code guidelines due to inadequate reinforcement. A numerical investigation has been carried out to evaluate the performance of the bridge with the suggested retrofitting techniques. The CFRP jacketing was implemented from Pantelides and Gergely (2002). The properties of the CFRP jacket used in their study were, the modulus of elasticity of 65 GPa, the ultimate strain of 0.001, the tensile strength of 628 N/mm^2 and the fiber volume fraction of 35%. The thickness of CFRP jacket was calculated to be 3.42 mm (Seible et al. 1997). The thickness of concrete jacket calculated from Priestley et al. was 120 mm. The compressive strength of concrete used in the jacket was 34 N/mm^2. The elliptical steel jacket and ECC jacket were also used. The following figures (Fig. 2.2) show the bridge bent retrofitted with different techniques.



Schematic diagram of different retrofitting techniques: (a) CFRP jacket, (b) steel jacket, and (c) concrete/ECC jacket

Figure 2.6 Bridge bent with different retrofitting techniques

The retrofitted bridge bent with the different retrofitting techniques were modelled in the SeismoStruct. Non-linear static analysis was performed to find the performance of the retrofitted bridge bent. The pushover curves of the existing bridge bent and retrofitted with different techniques are shown below in Fig. 2.3. From the pushover curves, it was observed that the desirable strength can be achieved from different retrofitting techniques.

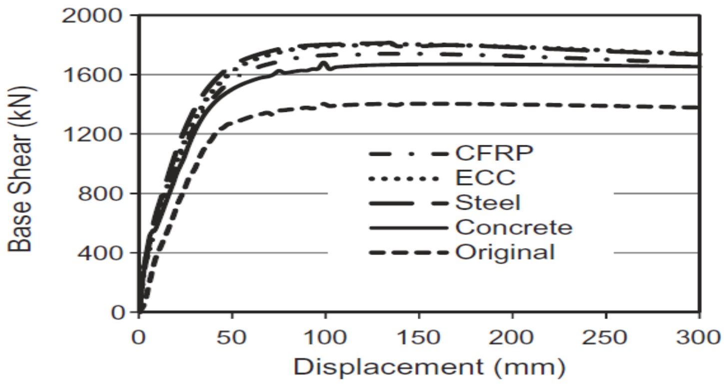


Figure 2.7 Pushover curves of different retrofitted bridge bent

CHAPTER 3

MODELLING OF BRIDGE SUBSTRUCTURES

3.1 Bridge Description

3.1.1 Ringhal Khad Bridge

The bridge is located at Ringhal on Jammu-Udhampur broad gauge link of Northern Railways (Thakkar, Dubey EQ-12, 1998). The bridge site is situated on seismic zone IV. The bridge is a 3 span continuous prestressed box girder. And the total length of the bridge is 192 m with the spans of 56 m, 80 m and 56 m.

Super structure

Type of superstructure	Continuous prestressed concrete box girder
Width of superstructure	6.9 m
Depth of box girder at supports	7.5 m
Depth at mid span	4.25 m
Width of box girder	3.5 m
Wall thickness of box girder	0.4 m

Bearings

Abutment A1	Fixed
Pier P1, P2 and A2	Free

Substructure

Height of piers	34.355 m
External diameter of hallow circular piers	6.5 m
Wall thickness of hallow circular pier	0.5 m
Thickness of pier cap	2.0 m

Foundation

Foundation rock	Shale and sand stone

Grade of Concrete

Superstructure	M 40
Substructure	M 25

3.1.2 Dhaleswari Bridge

The bridge taken for this study is Railway Bridge No 69 located on river Dhaleswari (Thakkar et al. EQ-11, 1984). The bridge site is situated in seismic zone V and site condition is soft soil. The bridge consists of seven spans of 31.926 m and two end spans of 13.10 m each.

Height of the bridge pier	18.50 m
Thickness of the pier cap	2 m
Diameter of the pier	3 m
Diameter of the pier cap	4.5 m
Grade of concrete	M 20

3.2 Modelling

In the first step three dimensional finite element model of Ringhal Khad bridge was created using SAP2000. In this modelling entire box girder deck section was modelled by using 4 node thin shell elements. The bent columns were modeled using 3D beam-column elements with each node having 6 DOF. It is assumed that the structure boundary conditions are fixed at base of the columns. The entire bridge model is used for finding out the dead load and live loads coming on to the substructure from super structure. Both the bridge pier models are developed in SAP2000 using 3D beam-column element. The dead load coming from the super structure is applied at top of piers by using lumped masses. Response spectrum approach and pushover analysis were performed to determine the base shear and performance of the bridge piers.

3.3 Assumptions in the Modelling

1. Boundary conditions at base of the both the piers are assumed to be fixed.
2. The reinforcement details assumed in the Ringhal Khad bridge piers is having longitudinal reinforcement of 105 bars of 25ϕ and transverse reinforcement of 12 ϕ @ 150 mm c/c.

3. The reinforcement details assumed in the Dhaleswari bridge piers is having longitudinal reinforcement of 32 bars of 25ϕ and transverse reinforcement of 12 ϕ @ 150 mm c/c.

The non-linearity is incorporated in two ways

1. Material non-linearity and
2. Geometrical non-linearity (P-Δ effect).

The use of nonlinear stress-stain relationships for material non-linearity is included in the modelling. The uniaxial stress-strain behaviour of confined and unconfined concrete is representing by Manders model and is shown in Fig. 3.1. Non-linearity in the structural members is incorporated by defining the hinge properties. The behaviour of non-linear plastic hinges is characterized by moment curvature relationships.

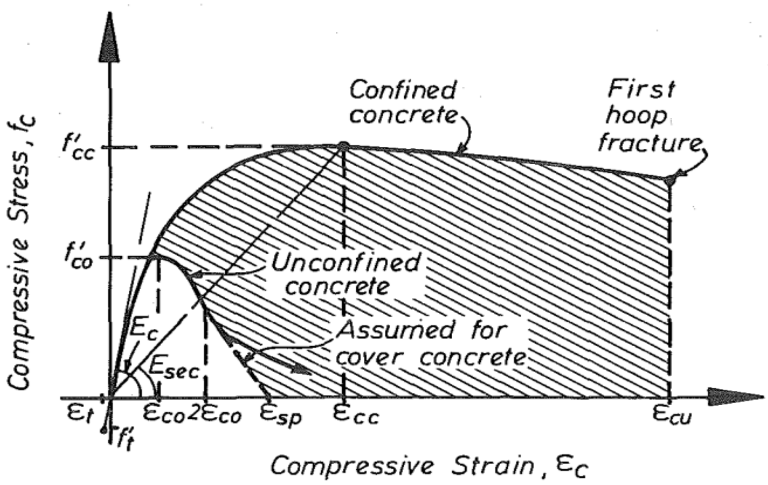


Figure 3.1 Stress-strain curve for monotonic loading of confined and unconfined concrete (Mander et al., 1988)

3.4 Dead and Live Load from Superstructure

The dead load and live load coming from the super structure are assigned as lumped masses at the top of the pier. The 50% of live load is considered in transverse direction for calculation seismic forces as per the guidelines of RDSO. The live loads on both bridges are considered as the modified broad gauge loading (MBG-1987). The dead load and live load considered on Ringhal Khad bridge pier are 17429.80 kN and 5293.30 kN respectively. The dead load and live load considered on Dhaleswari bridge pier are 5271.66 kN and 1250.86 kN respectively.

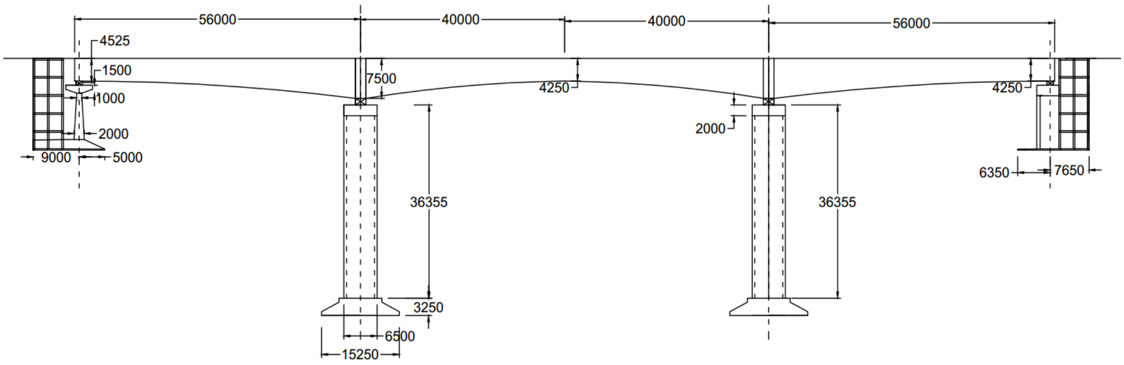


Figure 3.2 Elevation of Ringhal Khad bridge

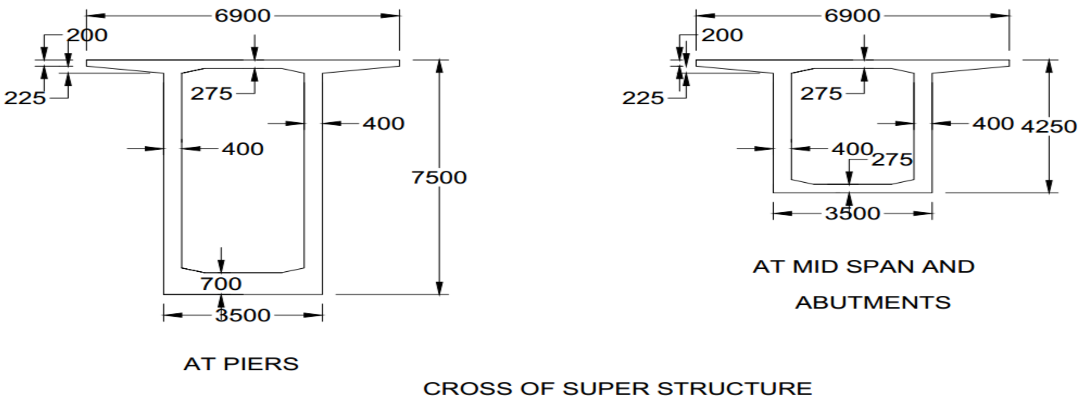


Figure 3.3 Details of box girder section of Ringhal Khad bridge

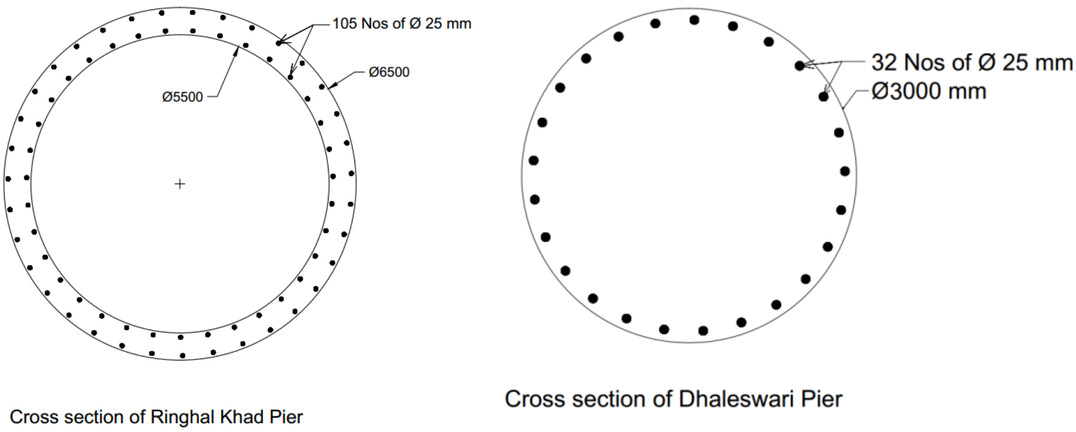


Figure 3.4 Cross sections of bridge piers

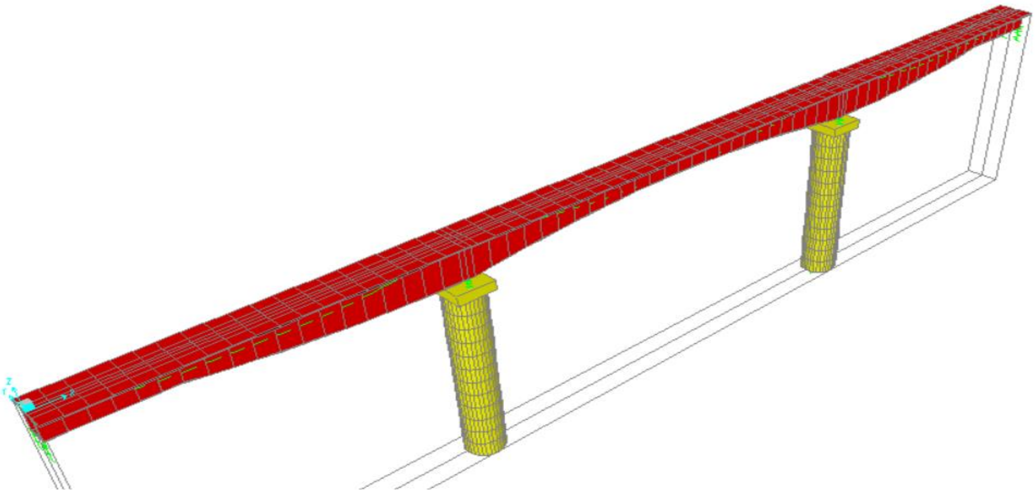


Figure 3.5 3D view of Ringhal Khad bridge

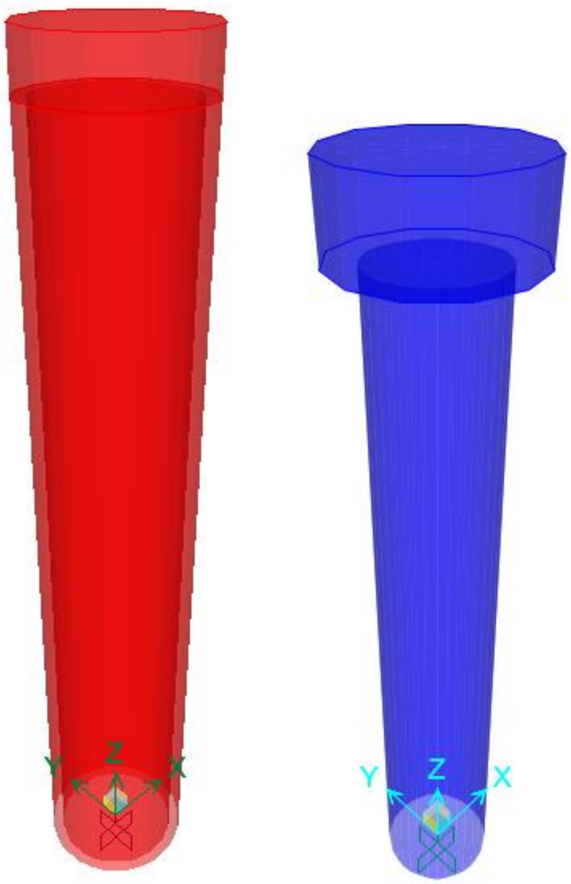


Figure 3.6 3D views of Ringhal Khad bridge pier and Dhaleswari bridge pier

CHAPTER 4

MODAL AND RESPONSE SPECTRUM ANALYSIS

4.1 Modal Analysis

Modal analysis is considered in order to determine the vibration modes of the finite element model. Dynamic characteristics of bridges are expected to be captured more accurately by the finite element models.

4.1.1 Dynamic Characteristics of Bridge Piers

For the considered bridge piers, modal analysis has been carried out to find dynamic characteristics of bridge piers. Time periods and modal mass participation factors for all the six modes have been shown below in Table 4.1 and 4.2.

Table 4.1 Time periods and modal mass participation factors for Ringhal Khad bridge pier

Step Type	Step No.	Time Period (s)	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
Mode	1	1.3144	1E-06	0.9984	0	1E-06	0.9984	0
Mode	2	1.3144	0.9984	1E-06	0	0.9985	0.9985	0
Mode	3	0.1309	0.0000	0.0000	1	0.9985	0.9985	1
Mode	4	0.0319	0.0015	0.0000	0	1.0000	0.9985	1
Mode	5	0.0319	0.0000	0.0015	0	1.0000	1.0000	1
Mode	6	0.0079	0.0000	0.0000	0	1.0000	1.0000	1

Table 4.2 Time periods and modal mass participation factors for Dhaleswari bridge pier

Step Type	Step No.	Time Period (s)	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
Mode	1	0.4736	0.5776	0.4190	0	0.5776	0.4190	0
Mode	2	0.4736	0.4190	0.5776	0	0.9966	0.9966	0
Mode	3	0.0321	0.0000	0.0000	1	0.9966	0.9966	1
Mode	4	0.0165	0.0034	4.0E-06	0	1.0000	0.9966	1
Mode	5	0.0165	4.0E-06	0.0034	0	1.0000	1.0000	1
Mode	6	0.0026	0.0000	0.0000	0	1.0000	1.0000	1

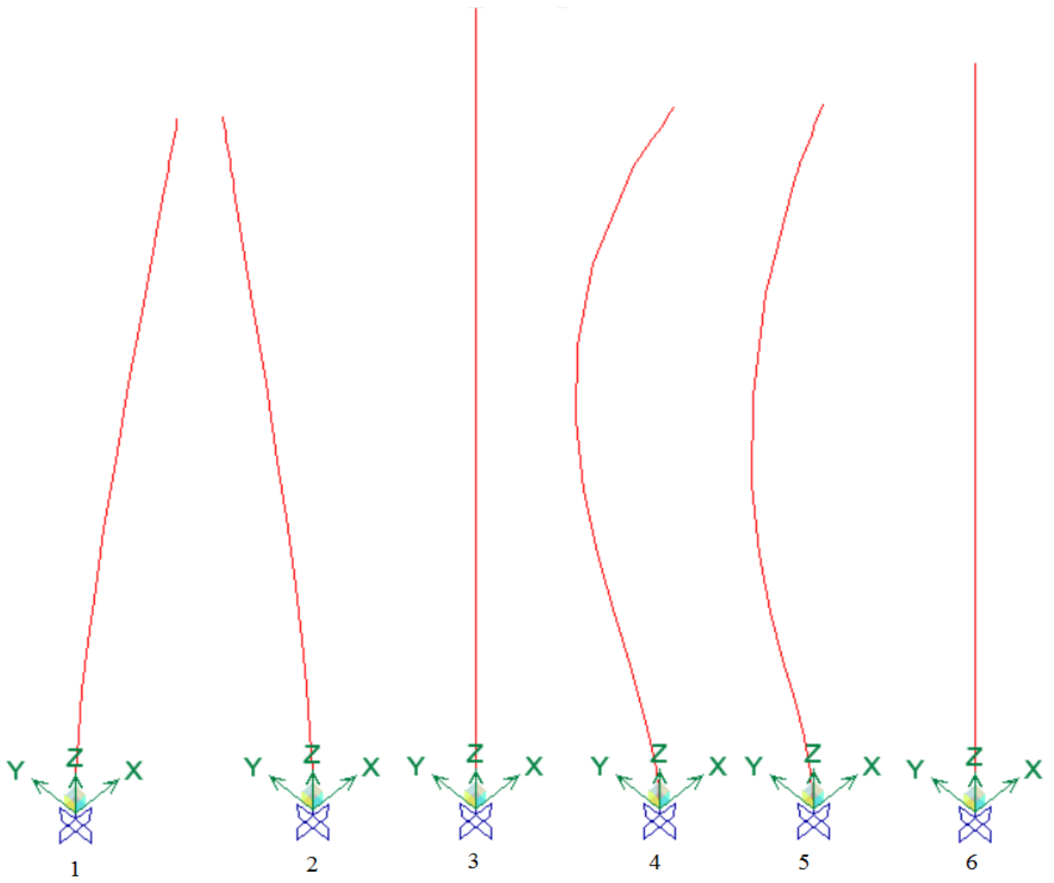


Figure 4.1 Ringhal Khad bridge pier 6 mode shapes

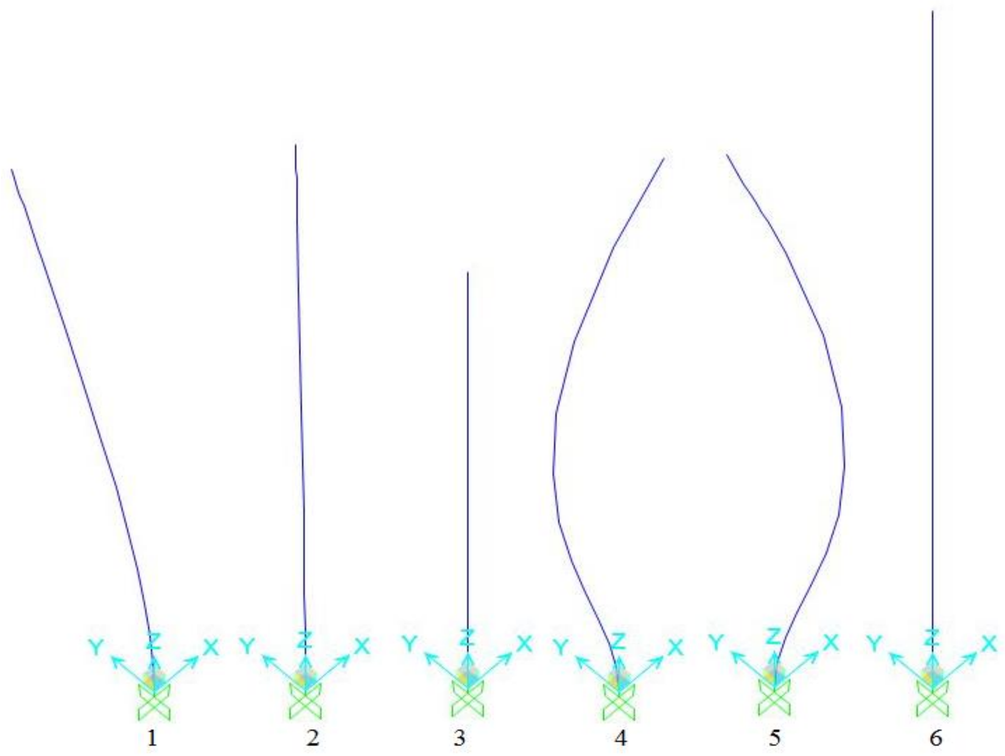


Figure 4.2 Dhaleswari bridge pier 6 mode shapes

4.2 Response Spectrum Analysis

Basically response spectrum analysis is an elastic seismic analysis procedure, which generally gives the reasonable response values for the predicted design motions, member forces and displacements in structures that are essentially expected to remain elastic under earthquake motions. The response spectrum method involves the calculation of peak values of the member forces and displacements in each mode using design spectra. It also accounts for the maximum considered earthquake (MCE) expected at the site. For each natural mode of vibration, under a set of an equivalent earthquake forces, a static analysis is conducted for the entire bridge structure. The resulting response is then multiplied by the spectral ordinates which may be displacement, pseudo-velocity, or pseudo-acceleration in order to obtain the peak modal response. Hence, this procedure converts the dynamic analysis to a series of static analysis, thus reducing the computational effort. The response spectrum analysis is still considered a dynamic analysis because this procedure involves the use of the vibration properties of the structure which includes the natural periods, the modes, and the modal damping ratios, in addition to the dynamic characteristics of the considered ground motions.

In SAP2000 response spectrum analysis is performed using mode superposition. The software automatically takes into account the elastic properties of the bridge. The non-linearities that have been defined for the geometry or materials of the structure are automatically ignored. The response spectrum curve chosen by the user reflects the damping that is present in the structure to be modelled.

For the considered models, the response spectrum analysis has been done as per IS: 1893-1984 and IS: 1893 (Part3)-2014. The soil beneath the Ringhal Khad bridge is hard rock and the site situated in zone IV. The soil beneath the Dhaleswari bridge is soft soil and the site situated in zone V. Damping considered for all modes is 5% and is constant for all modes of vibration considered.

As per IS: 1893-1984

Design horizontal seismic force is calculated by

$$F_h = \alpha_h W_m \tag{4.1}$$

where,

F_h = horizontal seismic force to be resisted

α_h = design horizontal seismic coefficient

W_m = mass of the structure excluding the buoyancy effect

The α_h is calculated by

$$\alpha_h = \beta I F_0 \left(\frac{S_a}{g} \right) \tag{4.2}$$

where,

β = a coefficient depending upon the soil-foundation system

I = importance factor depending upon the structure

F_o = seismic zone factor as shown in Table 4.3 and

(Sa/g) = average acceleration coefficient as read from Fig. 4.3 for appropriate natural period and damping of the structure

Table 4.3 Seismic zone factor, F_o

Seismic Zone	**V**	**IV**	**III**	**II**	**I**
F_o	0.40	0.25	0.20	0.10	0.05

For Ringhal Khad bridge pier β is considering as 1.0 according to their soil foundation system and F_o is considering as 0.25. For Dhaleswari bridge pier β is considering as 1.5 according to their soil foundation system and F_o is considering as 0.40. Importance factor is 1.5 for both bridge piers.

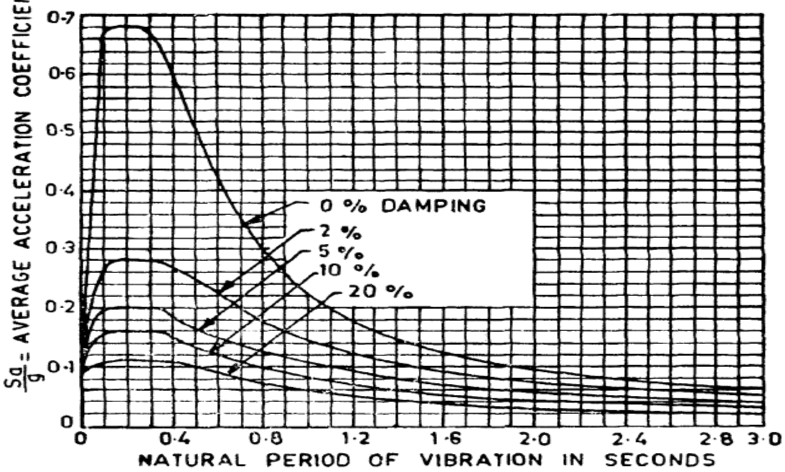


Figure 4.3 Average acceleration spectra as per IS: 1893-1984

As per IS: 1893 (Part 3) - 2014

Design horizontal seismic force is calculated by

$$F = A_h W \tag{4.3}$$

where,

W = Mass of the structure neglecting the buoyancy effect

A_h = Design horizontal seismic coefficient

$$= \frac{Z}{2}\frac{I}{R}\frac{S_a}{g} \tag{4.4}$$

where,

Z = Zone factor for MCE as given in IS: 1893 (Part 1)-2002

I = Importance factor, depending upon type of the structures

R = Response reduction factor

Sa/g = Average acceleration coefficient for rock or soil sites based on period of the structure as shown in Fig 4.4

The zone factor for Ringhal Khad bridge is 0.24 and for Dhaleswari bridge is 0.36. The response reduction factor considered for both bridge piers is taken as 3.0 (cantilever type). Importance factor for both the bridges are 1.5.

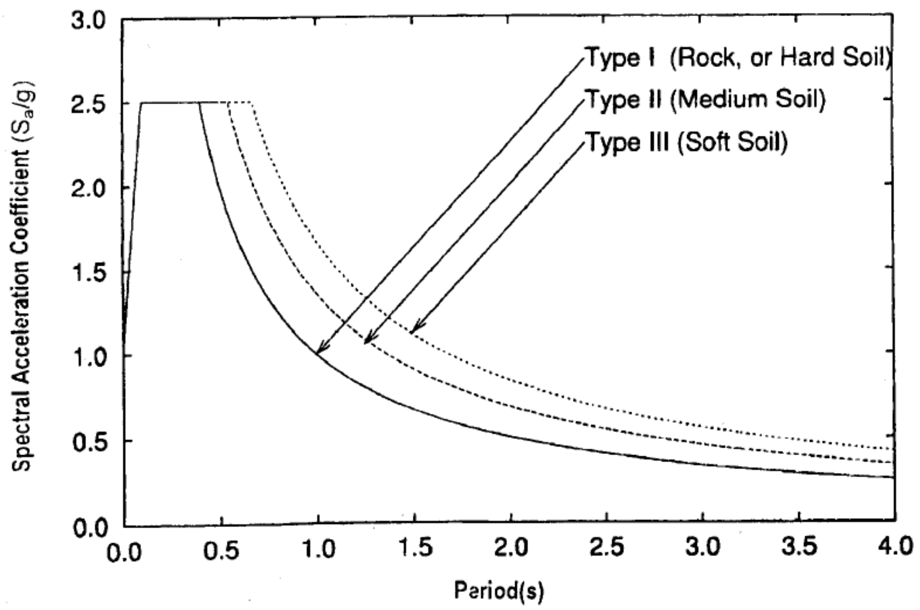


Figure 4.4 Response spectra for 5% damping as per IS: 1893 (Part 1)-2002

4.3 Response Spectrum Analysis Results

Response spectrum analysis based on IS: 1893-1984, IS: 1893 (Part 3)-2014 for DBE and MCE has been shown below for the both bridge piers.

Table 4.4 Shear force at base of the Ringhal Khad bridge pier

Seismic Force Based on Response Spectra	**Base Shear (kN)**
IS: 1893-1984	908.95
IS: 1893 (Part 3)-2014 DBE	1320.31
IS: 1893 (Part 3)-2014 MCE	2640.61

Table 4.5 Shear force at base of the Dhaleswari bridge pier

Seismic Force Based on Response Spectra	**Base Shear (kN)**
IS: 1893-1984	402.09
IS: 1893 (Part 3)-2014 DBE	542.526
IS: 1893 (Part 3)-2014 MCE	1085.05

CHAPTER 5

NON-LINEAR STATIC ANALYSIS

5.1 Introduction

Non-linear static analysis is also called as pushover analysis or inelastic collapse analysis. The non-linear static procedure in ATC (1996) and FEMA-365 is based on capacity spectrum method which was originally developed by Freeman *et al.,* (1975) and Freeman (1978). Further, pushover analysis procedure was developed, and refined by Priestly and Seible (1994), who applied this method to a number of real assessment and retrofit situations. In recent years, pushover analysis is becoming popular for performance evaluation of existing and new structures for predicting seismic demand forces and deformations. It is a relatively simple intermediate solution to the complex problems of predicting seismic demand forces and deformations imposed on a structure by a severe ground motions. The following are the different pushover analysis methods used for predicting the performance or seismic assessment of structures:

a. Capacity spectrum method
b. Displacement coefficient method
c. Equivalent linearization
d. Displacement modification method

5.2 Capacity Spectrum Method

Applied Technology Council (ATC) (1996) presented a non-linear static procedure to evaluate performance of reinforced concrete buildings subjected to seismic loadings. That procedure uses the static pushover analysis to

a. Represent the structures lateral force resisting capacity.
b. Determine the displacement demand procedure by the earthquake intensity on the structure.
c. Verify an acceptable performance level.

In general, performance is accepted when the structural capacity is larger than the demand required to satisfy a proper performance level. ATC (1996) adopts the capacity spectrum method (CSM) to determine the demand displacement, which is the maximum extreme response of the building during the ground motion. The demand displacement in the capacity spectrum method occurs at a point on the capacity curve (pushover curve) called the performance point. This performance point represents the condition for which the seismic capacity of the structure is equal to the seismic demand imposed on the structure by the specified ground motion.

ATC (1996) presented the CSM in detail and explained through step-by-step procedure to apply this method. The following are the important steps which are given in ATC

1. To find the pushover (capacity) curve, this represents the relation between the base shear and roof displacement as shown in Fig 5.1(a). Roof is the control node in case of bridges.
2. Convert the pushover curve to the capacity spectrum curve as depicted in Fig 5.1(b) by using following equations.

   Spectral displacement can be obtained by

   $$S_d = \frac{\Delta_{roof}}{\Gamma_1 \phi_{roof1}} \tag{5.1}$$

   where,

   Δ_{roof} is the roof displacement

   ϕ_{roof1} is the mode shape coefficient at roof and

   Γ_1 is the modal participation factor of mode1 and is given by

   $$\Gamma_1 = \frac{\sum_{j=1}^{N} m_j \phi_{j1}}{\sum_{j=1}^{N} m_j \phi_{j1}^2} \tag{5.2}$$

   Spectral acceleration can be obtained by

   $$S_a = \frac{V_b}{\alpha_1} \tag{5.3}$$

   where,

   V_b is the base shear and

   α_1 is the effective modal mass of mode1 is given by

$$\alpha_1 = \frac{\left[\sum_{j=1}^{N} m_j \phi_{j1} \right]^2}{\sum_{j=1}^{N} m_j \phi_{j1}^2} \tag{5.4}$$

where,

m_j is the lumped mass at j^{th} floor level

ϕ_{j1} is the mode shape coefficient at j^{th} floor of mode1 and

N is the number of floors

3. Convert the elastic response spectrum from the standard format (S_a vs T) to acceleration displacement response spectrum (ADRS) format (S_a vs S_d) as shown in Fig 5.1(c).
4. Determine the displacement demand as the intersection of the capacity spectrum curve and the demand spectrum curve, reduced from the elastic 5% damped demand spectrum as shown in Fig 5.1(d). The point of intersection represents the non-linear demand at the same spectral displacement. This step needs iterations and each iteration includes calculating updated values of natural period T_{eff} and effective damping β_{eff}. An approximately effective damping is calculated based on the shape of the capacity curve, the estimated displacement demand, and the resulting hysteretic loop.
5. Convert the displacement demand obtained in previous step back to global roof displacement.
6. Finally evaluate the deformations of individual components corresponding to demand displacement with the capacity of that component. If the demand displacement components exceed the capacity then the structure need to be retrofit to increase capacity.

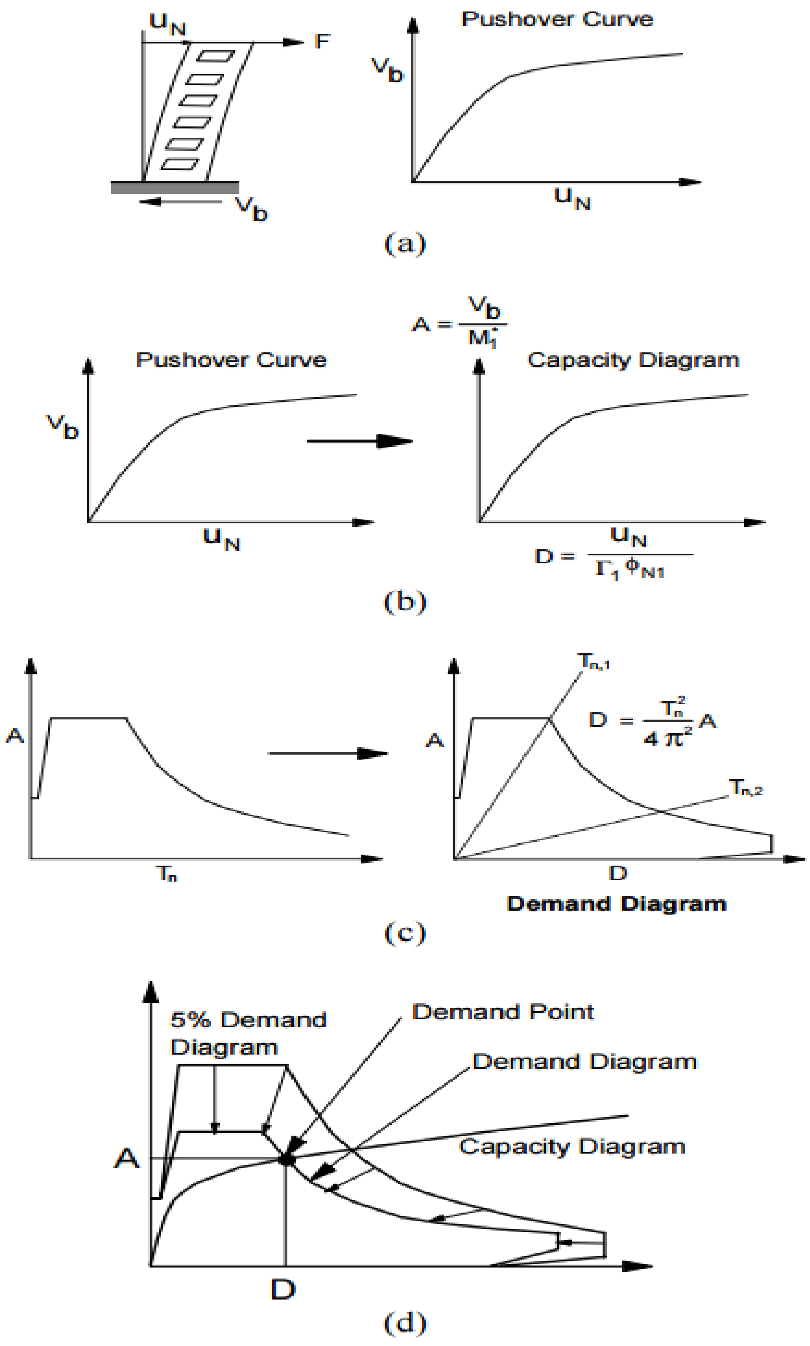


Figure 5.1 Capacity spectrum method

(a) Development of pushover curve (b) Conversion of pushover curve to capacity spectrum diagram (c) Conversion of elastic response spectrum to ADRS format (d) Determination of displacement demand (Chopra and Goel, 1999).

5.2.1 Estimation of Damping

Estimation of equivalent viscous damping is performed by representing the hysteretic damping as equivalent viscous damping. For the case where the capacity curve is replaced by a bilinear curve as shown in Fig. 5.2 equivalent viscous damping βo can be calculated as

$$\beta_0 = \frac{1}{4\pi}\frac{E_D}{E_{S0}} \tag{5.5}$$

The effective damping βeff associated with maximum displacement can be calculated as

$$\beta_{eff} = 0.05 + \beta_0 = 0.05 + \frac{0.637(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} \tag{5.6}$$

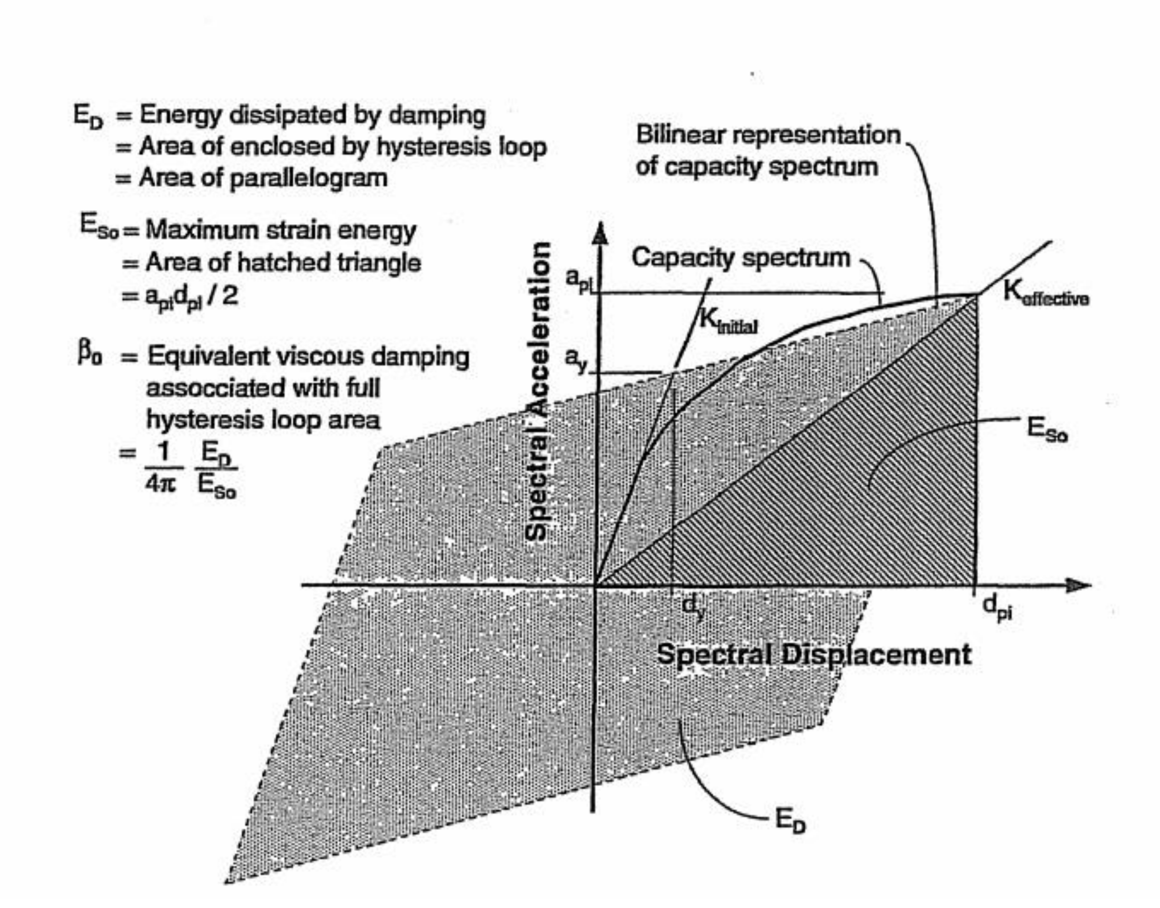


Figure 5.2 Derivation of damping for spectral reduction (ATC-40, 1996)

5.3 Pushover Analysis Results

5.3.1 Ringhal Khad Bridge Pier

For performing the non-linear static analysis, non-linearity in the concrete is considered by using the Mander model and geometrical non-linearity by using the P-Δ effect. The nonlinearity in the bridge pier is incorporated by defining the plastic hinge properties. The plastic hinge properties are manually defined as per ASCE 41-13. The hinges are assigned at the base of piers. The capacity curve of the Ringhal Khad pier obtained from the pushover analysis is shown below Fig. 5.3.

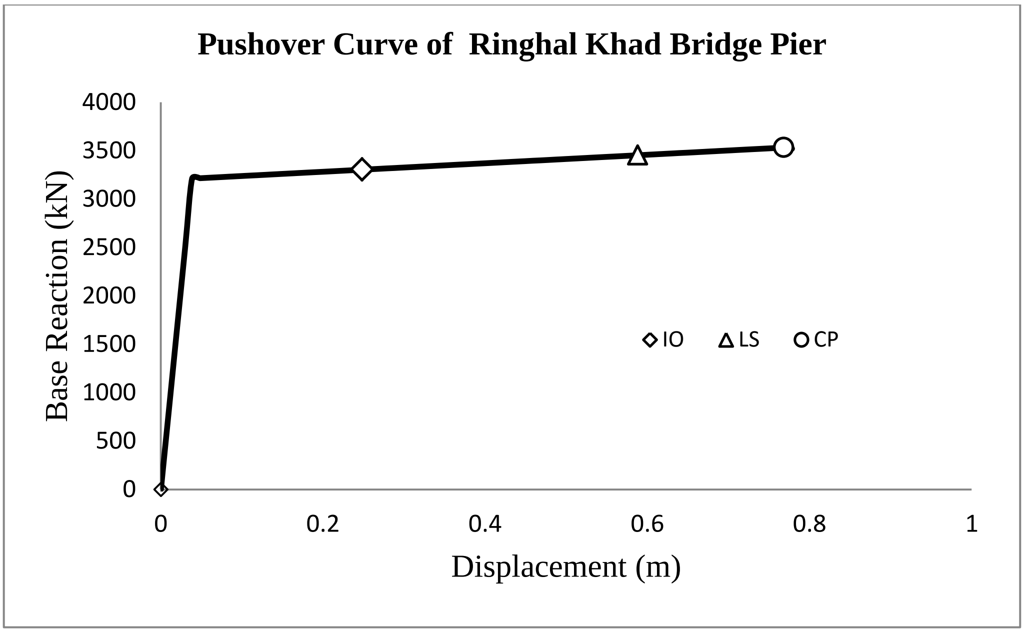


Figure 5.3 Pushover curve of Ringhal Khad bridge pier

From the capacity curve and the hinge properties the resulting value of shear force, bending moment and displacements corresponding to the various acceptance criteria is shown below in Table 5.1.

Table 5.1 Acceptance criteria of Ringhal Khad bridge bier

Acceptance Criteria	**Base Shear (kN)**	**Bending Moment (kNm)**	**Displacement (m)**
At Yield Point	3212.17	89587.47	0.0384
At Immediate Occupancy	3304.65	91916.11	0.2484
At Life Safety	3453.98	96328.10	0.5884
At Collapse Prevention	3533.05	98535.42	0.7684

The performance point of the Ringhal Khad bridge pier is obtained by using ATC 40 capacity spectrum method corresponding to the demand spectra of IS: 1893-1984, IS: 1893 (Part 3)-2014 under DBE and MCE. The results are as follows

1. Seismic demand shear force or base shear at the performance point based on IS: 1893-1984 = 1012.34 kN < 3212.17 kN yield point.
   Demand moment = 28234.16 kNm < 89587.47 kNm yield moment.
   Demand displacement based on IS: 1893-1984 = 0.012 m < 0.0384 m yield displacement.
2. Seismic demand shear force or base shear at the performance point based on IS: 1893 (Part 3)-2014 under DBE
   = 3227.754 kN > 3212.17 kN yield point
   < 3304.65 kN immediate occupancy.
   Demand moment = 90022.06 kNm > 89587.47 kNm yield moment
   < 91916.11 kNm immediate occupancy.
   Demand displacement based on IS: 1893 (Part 3)-2014 under DBE
   = 0.074 m > 0.0384 m yield displacement
   < 0.2484 m immediate occupancy.
3. Seismic demand shear force or base shear at the performance point based on IS: 1893 (Part 3)-2014 under MCE
   = 3487.486 kN > 3453.98 kN life safety point
   < 3533.05 kN collapse prevention.
   Demand moment = 97265.98 kNm > 96328.10 kNm life safety point
   < 98535.42 kNm collapse prevention.
   Demand displacement based on IS: 1893 (Part 3)-2014 under MCE
   = 0.665 m > 0.588 m life safety point
   < 0.768 m collapse prevention.

According to the guidelines of IS: 1893-1984 the bridge pier is safe, but it is outdated. As per the present guidelines of IS 1893 (Part 3)-2014 under DBE, the bridge pier performance is in between the yield point and immediate occupancy, which is acceptable. But, under MCE condition, bridge pier crosses the life safety point in the both the seismic demand considerations of force, moment and displacement. So retrofitting of bridge pier is needed to improve seismic performance under MCE condition also.

5.3.2 Dhaleswari Bridge Pier

The capacity curve of the River Dhaleswari bridge pier obtained from the pushover analysis is shown below in Fig. 5.4.

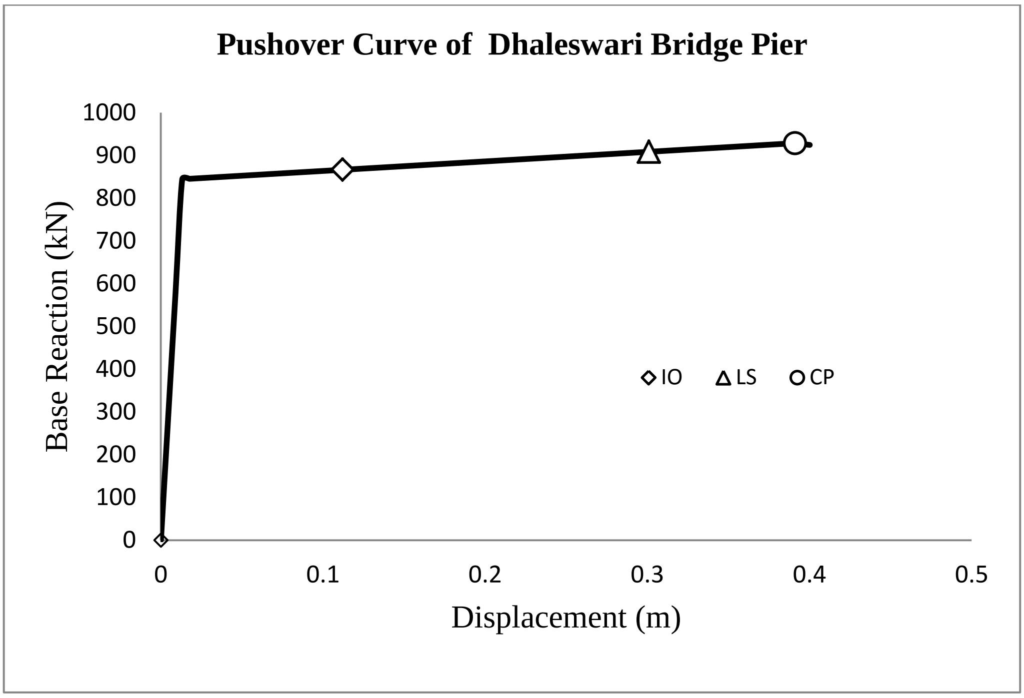


Figure 5.4 Pushover curve of Dhaleswari bridge pier

From the capacity curve and the hinge properties the resulting value of shear force, bending moment and displacements corresponding to the various acceptance criteria is shown below in Table 5.2.

Table 5.2 Acceptance criteria of Dhaleswari bridge pier

Acceptance Criteria	Base Shear (kN)	Bending Moment (kNm)	Displacement (m)
At Yield Point	844.57	8766.94	0.0131
At Immediate Occupancy	866.67	8996.30	0.1121
At Life Safety	908.85	9434.16	0.3011
At Collapse Prevention	928.94	9642.67	0.3911

The performance point of the Dhaleswari bridge pier is obtained by using ATC 40 Capacity Spectrum method corresponding to the demand spectrums of IS: 1893-1984, IS: 1893 (Part 3)-2014 under DBE and MCE. The results are as follows

1. Seismic demand shear force or base shear at the performance point based on IS: 1893-1984 = 434.56 kN < 844.57 kN Yield point, hence it is ok.
   Demand moment = 4510.73 kNm < 8766.94 kNm yield moment.
   Demand displacement based on IS: 1893-1984
   = 0.006 m < 0.0131 m yield displacement.

2. Seismic demand shear force or base shear at the performance point based on IS: 1893 (Part 3)-2014 under DBE = 845.11 kN > 844.57 kN yield point
   < 866.67 kN immediate occupancy.
   Demand moment = 8772.24 kNm > 8766.94 kNm yield moment
   < 9434.16 kNm immediate occupancy.
   Demand displacement based on IS: 1893 (Part 3)-2014 under DBE
   = 0.015 m > 0.0131 m yield displacement
   < 0.1121 m immediate occupancy.

3. Seismic demand shear force or base shear at the performance point based on IS: 1893 (Part 3)-2014 under MCE
   = 911.547 kN > 908.85 kN life safety point
   < 928.94 kN collapse prevention.
   Demand moment = 9461.86 kNm > 9434.16 kNm life safety point
   < 9642.67 kNm collapse prevention.
   Demand displacement based on IS: 1893 (Part 3)-2014 under MCE
   = 0.31 3m > 0.301 m kN life safety point
   < 0.391 m collapse prevention.

According to the guidelines of IS: 1893-1984 the bridge pier is safe, but it is outdated. As per the present guidelines of IS: 1893 (Part 3)-2014 under DBE the bridge performance is in between the yield point and immediate occupancy, which is acceptable. But, under MCE bridge crosses the life safety point in the both the seismic demand considerations of force, moment and displacement. So retrofitting of bridge pier is needed to improve seismic performance under MCE condition also.

CHAPTER 6

RETROFITTING OF BRIDGE PIERS

6.1 Introduction

Most of the bridges built before 1970 are seismically deficient. The strength and ductility of these structures can be increased by using the various retrofitting techniques to resist the required seismic forces.

Retrofitting and strengthening includes repairing the impaired and remodelling the existing structure with the current requirement. Remodelling refers to renewal of an existing structure owing to its change in occupancy and usage. Retrofitting helps in regaining the strength of a structure which is deteriorated. Strengthening makes a structure to increase its design capacity and also upgrades the structure to meet the requirements of current design procedures.

These processes of repairing a structure can be done by many methods as described below

- Ferro cement jacketing
- RC Jacketing
- Steel plate jacketing
- Stitching
- Bonding of external reinforcement
- Drilling and plugging
- Chemical grouting
- Wrapping with fiber composites
- Addition of bracings

The structure which is to be repaired should first be studied completely and then an appropriate method should be adopted. The repair technique adopted should not increase the dimensions of the member as it increases the dead load of the structure. The main complication that arises for the repairing technique is de-bonding with the substrate, and hence it has to be checked. The repair strategy should not have prolonged strength attainment, and should not make the structure brittle. The method adopted should be economical and should not hinder with the construction easiness.

The major techniques used for strengthening of reinforced concrete bridges are reinforced concrete jacket, steel jacket, fiber reinforced polymer (FRP) jacket, textile reinforced mortar and engineered cementitious composites (ECC) jacket.

6.2 Reinforced Concrete Jacketing

In the reinforced concrete jacket, the structural member or bridge pier is strengthened by using added concrete, longitudinal reinforcement and the transverse reinforcement placed around the member. This type of strengthening improves the axial load carrying capacity, shear strength and the flexural strength. To develop the interface between the old and new surfaces the old member is roughened by chipping the cover.

Guidelines for RC Jacketing as per IS: 15988-2013

i. Strength of concrete used for jacketing shall be same or greater than the existing column strength. It shall be at least 5 MPa greater than the existing strength of the concrete.
ii. Minimum thickness of the jacket should be provided is 100 mm.
iii. If there is no need to provide extra reinforcement, a minimum of ϕ12 mm bars at the four corners in case of rectangular or 6 bars in case of circular member with ties of ϕ8 mm @ 100 mm c/c should be provided with 135^{o} bends and 80 mm leg length.
iv. Minimum diameter of the tie shall not be less than 1/3rd of the diameter of the longitudinal bar or ϕ8 mm whichever is large.
v. Spacing of the ties shall not exceed 200 mm or thickness of the jacket.

Advantages of RC Jacketing

i. To increase the axial, shear and flexural strength of a member.
ii. Construction is easy.
iii. Materials are easily available.

Disadvantages

i. Increase in size of section and dead weight.
ii. Anchorage of longitudinal bars into the foundation is needed.
iii. Drilling of holes into existing members.
iv. Placement of tie bars at joints is difficult.
v. Chipping of existing members for proper bonding is required.

Figure 6.1 Reinforced concrete jacketing of bridge bent

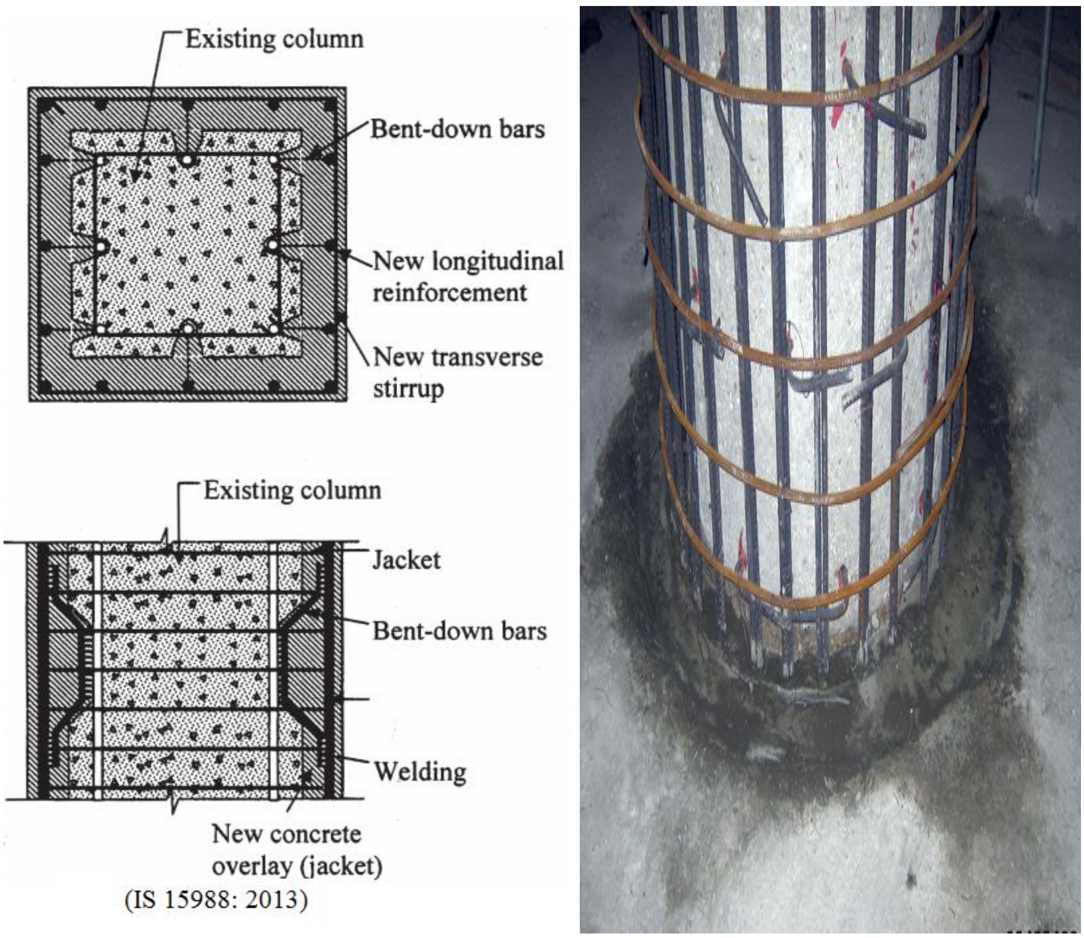


Figure 6.2 Reinforced concrete jacketing

6.2.1 Reinforced Concrete Jacketing of Ringhal Khad Bridge Pier

Grade of concrete used in the existing column is M 25.

As per the guidelines the grade of concrete used in concrete retrofitting is M 30.

Thickness of RC jacket provided = 100 mm.

Grade of steel used in RC jacket = Fe 415

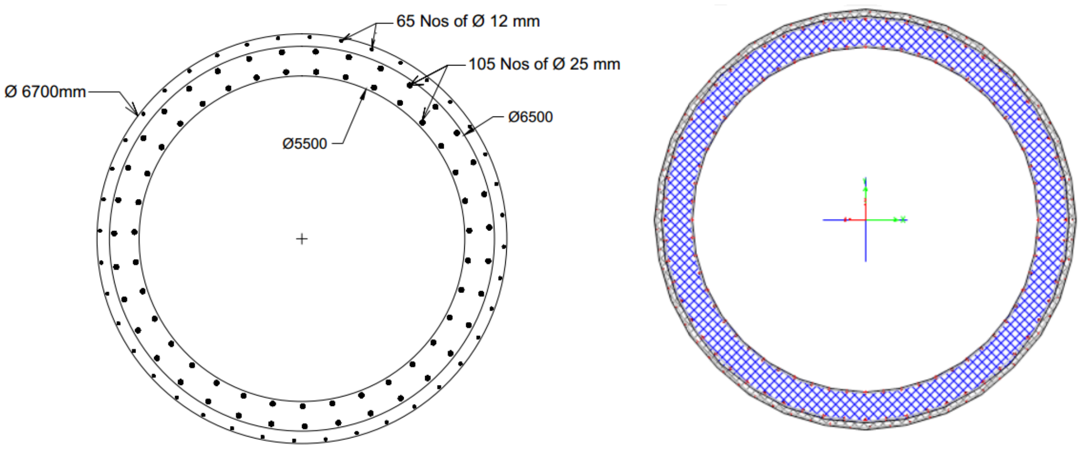
Diameter of bars used in Jacket = 12 mm

Number of bars used in concrete jacket = 65

The inner diameter of the existing pier = 5.5 m

The outer diameter of the existing pier = 6.5 m

The outer diameter of the pier after reinforced concrete jacketing = 6.7 m



Cross section of RC Jacketed Ringhal Khad Pier

Cross section defined in SAP2000 Section Designer

Figure 6.3 Cross section of the pier after the reinforced concrete jacketing

Three dimensional finite element model of the retrofitted Ringhal Khad bridge pier with the reinforced concrete jacket is prepared in SAP2000. Modal analysis has been done to find dynamic characteristics of bridges such as time periods and modal mass participation factors at different modes.

Table 6.1 Time periods and modal mass participation factors for RC jacketed Ringhal Khad bridge pier

Step Type	Step No.	Time Period (s)	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
Mode	1	1.1895	0.1194	0.8790	0.0000	0.1194	0.8790	0.0000
Mode	2	1.1895	0.8790	0.1194	0.0000	0.9984	0.9984	0.0000
Mode	3	0.1211	0.0000	0.0000	1.0000	0.9984	0.9984	1.0000
Mode	4	0.0306	3.E-06	0.0016	0.0000	0.9985	1.0000	1.0000
Mode	5	0.0306	0.0016	3.E-06	0.0000	1.0000	1.0000	1.0000
Mode	6	0.0083	0.0000	0.0000	0.00004	1.0000	1.0000	1.0000

The comparison of the time periods in all the six modes for the existing and RC jacketed Ringhal Khad bridge pier is shown below in Table 6.2.

Table 6.2 Comparison of time periods for existing and RC jacketed Ringhal Khad bridge pier

Mode No.	For Existing Pier Time Period (s)	For RC Jacketed Pier Time Period (s)
1	1.3144	1.1895
2	1.3144	1.1895
3	0.1309	0.1211
4	0.0319	0.0306
5	0.0319	0.0306
6	0.0079	0.0083

For performing the non-linear static analysis, non-linearity in the concrete is considered by using the Mander model and geometrical non-linearity considered by using the P-Δ effect. The plastic hinge is assigned at the base of piers. The capacity curve of the Ringhal Khad pier with reinforced concrete jacket obtained from the pushover analysis is shown below in Fig. 6.4.

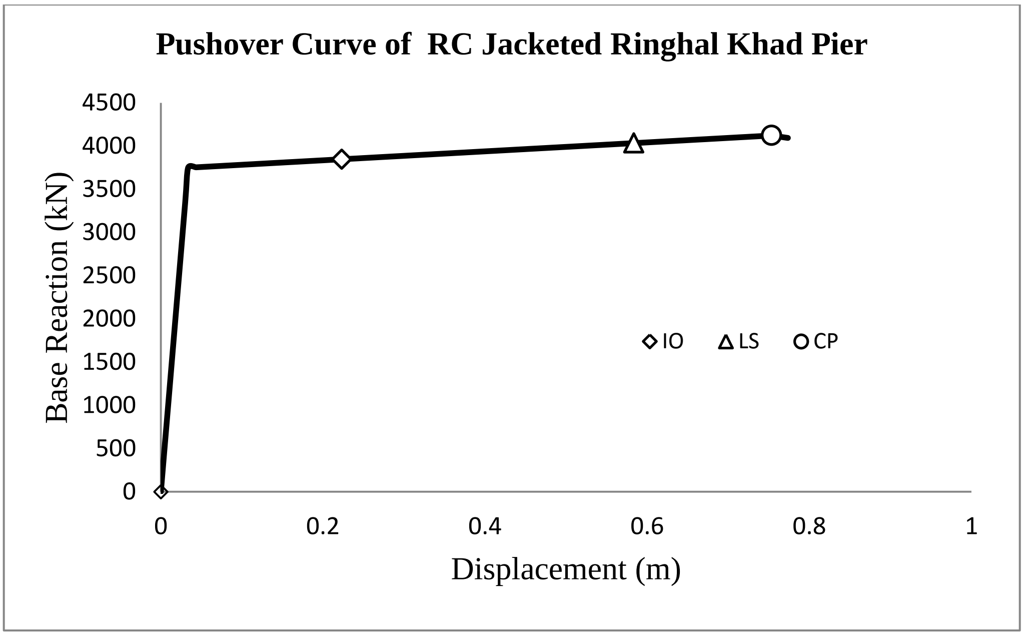


Figure 6.4 Pushover curve of RC jacketed Ringhal Khad pier

From the capacity curve and the hinge properties the resulting value of shear force, bending moment and displacements corresponding to the various acceptance criteria is shown below in Table 6.3.

Table 6.3 Capacity of RC jacketed Ringhal Khad pier at various acceptance points

Acceptance Criteria	**Base Shear (kN)**	**Bending Moment (kNm)**	**Displacement (m)**
At Yield Point	3748.45	101024.33	0.0336
At Immediate Occupancy	3846.82	103675.55	0.2236
At Life Safety	4033.21	108698.91	0.5836
At Collapse Prevention	4121.23	111071.05	0.7536

The performance point of the RC jacketed Ringhal Khad bridge pier is obtained by using ATC 40 capacity spectrum method corresponding to the demand spectrums of IS: 1893-1984, IS: 1893 (Part 3)-2014 under DBE and MCE. The results are as follows

1. Seismic demand shear force or base shear at the performance point based on IS: 1893-1984 = 1182.38 kN < 3748.45 kN yield point, hence it is ok.
   Demand moment based on IS: 1893-1984
   = 31865.14 kNm < 101024.33 kNm yield moment.

Demand displacement based on IS: 1893-1984

= 0.011 m < 0.034 m yield displacement.

2. Seismic demand shear force or base shear at the performance point based on IS: 1893 (Part 3)-2014 under DBE = 3539.68 kN < 3748.45 kN yield point.

   Demand moment based on IS: 1893 (Part 3)-2014 under DBE

   = 95394.46 kNm < 101024.33 kNm yield moment.

   Demand displacement based on IS: 1893 (Part 3)-2014 under DBE

   = 0.032 m < 0.034 m yield displacement, hence it is ok.

3. Seismic demand shear force or base shear at the performance point based on IS: 1893 (Part 3)-2014 under MCE

   = 3757.28 kN > 3748.45kN yield capacity

   < 3846.82 kN immediate occupancy.

   Demand moment based on IS: 1893 (Part 3)-2014 under MCE

   = 101258.70 kNm > 101024.33 kNm yield moment.

   < 103675.55 kNm immediate occupancy.

   Demand displacement based on IS: 1893 (Part 3)-2014 under MCE

   = 0.051m > 0.0336 m yield displacement

   < 0.223 m immediate occupancy, hence ok.

According to the guidelines of IS: 1893-1984 (at the time of construction) and IS: 1893 (Part 3)-2014 under DBE the bridge performance is within the yield point, which is acceptable. The performance of retrofitted bridge pier under MCE reaches the yield point and within immediate occupancy, which is acceptable. The required strength is enhanced by reinforced concrete jacketing.

6.2.2 Reinforced Concrete Jacketing of Dhaleswari Bridge Pier

Grade of concrete used in the existing column is M 25.

As per the guidelines the grade of concrete used in concrete retrofitting is M 30.

Thickness of RC jacket provided = 100 mm.

The diameter of the existing pier = 3.0 m

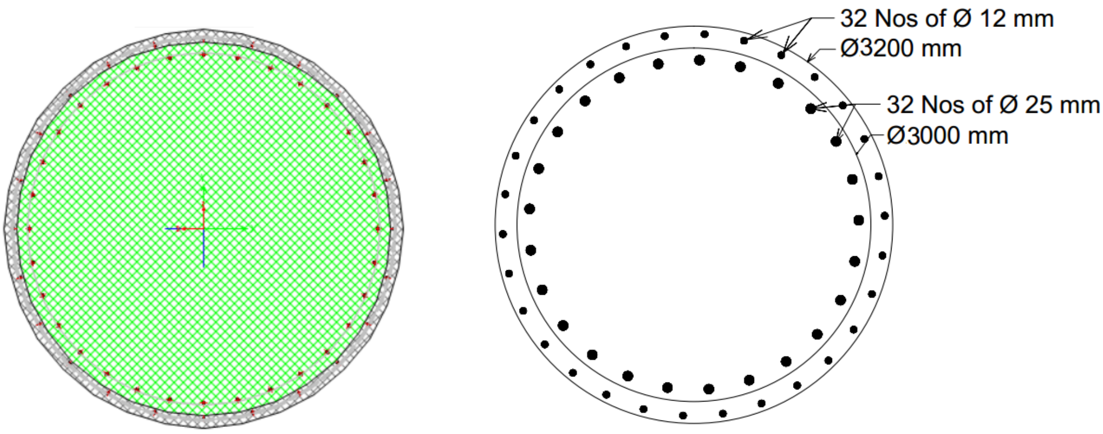
The diameter of the pier after reinforced concrete jacketing = 3.2 m

Grade of steel used in RC jacket = Fe 415

Reinforcement provided in the existing pier is 32 bars of 25 mm diameter bars.

Diameter of bars used in jacket = 12 mm

Number of bars used in concrete jacket = 32



Cross section defined in SAP2000 Section Designer

Cross section of RC Jacketed Dhaleswari Pier

Figure 6.5 Cross section of the pier after the reinforced concrete jacketing

Three dimensional finite element model of the retrofitted Dhaleswari bridge pier with the reinforced concrete jacket is done in SAP2000. Modal analysis has been done to find dynamic characteristics of bridges such as time periods and modal mass participation factors at different modes.

Table 6.4 Time periods and modal mass participation factors for RC jacketed Dhaleswari bridge pier

Step Type	Step No.	Time Period (s)	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
Mode	1	0.4298	0.0236	0.9732	0.0000	0.0236	0.9732	0.0000
Mode	2	0.4298	0.9732	0.0236	0.0000	0.9968	0.9968	0.0000
Mode	3	0.0312	0.0000	0.0000	0.9999	0.9968	0.9968	0.9999
Mode	4	0.0147	1.2E-05	0.0032	0.0000	0.9969	0.9999	0.9999
Mode	5	0.0147	0.0032	1.2E-05	0.0000	1.0000	1.0000	0.9999
Mode	6	0.0026	0.0000	0.0000	8.8E-06	1.0000	1.0000	1.0000

The comparison of the time periods in all the six modes for the existing and RC jacketed Dhaleswari bridge pier is shown below in Table 6.5.

Table 6.5 Comparison of time periods for existing and RC jacketed Dhaleswari bridge pier

Mode No.	For Existing Pier Time Period (s)	For RC Jacketed Pier Time Period (s)
1	0.4736	0.4298
2	0.4736	0.4298
3	0.0321	0.0312
4	0.0165	0.0147
5	0.0165	0.0147
6	0.0026	0.0026

For performing the non-linear static analysis, non-linearity in the concrete is considered by using the Mander model and geometrical non-linearity considered by using the P-Δ effect. The hinges are assigned at the base of piers. The capacity curve of the Dhaleswari bridge pier with reinforced concrete jacket obtained from the pushover analysis is shown in fig 6.6.

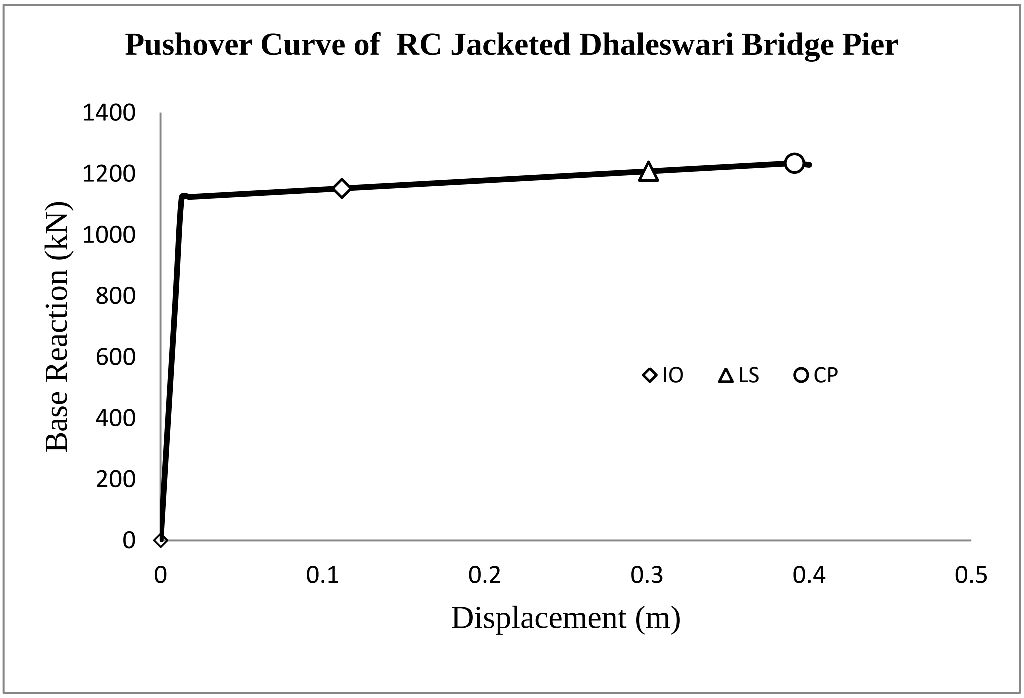


Figure 6.6 Pushover curve of RC jacketed Dhaleswari bridge pier

From the capacity curve and the hinge properties the resulting value of shear force, bending moment and displacements corresponding to the various acceptance criteria is shown below in Table 6.6.

Table 6.6 Capacity of RC jacketed Dhaleswari pier at various acceptance points

Acceptance Criteria	**Base Shear (kN)**	**Bending Moment (kNm)**	**Displacement (m)**
At Yield Point	1122.71	11444.85	0.013
At Immediate Occupancy	1152.10	11743.15	0.112
At Life Safety	1208.21	12314.96	0.301
At Collapse Prevention	1234.92	12587.31	0.391

The performance point of the RC jacketed Dhaleswari bridge pier is obtained by using ATC 40 Capacity Spectrum method corresponding to the demand spectrums of IS: 1893-1984, IS: 1893 (Part 3)-2014 under DBE and MCE. The results are as follows

1. Seismic demand shear force or base shear at the performance point based on IS: 1893-1984 = 474.87 kN < 1122.71 kN yield point, hence it is ok.
   Demand moment = 4840.21 kNm < 11444.85 kNm yield moment.
   Demand displacement based on IS 1893-1984 = 0.005 m < 0.013 m yield displacement.

2. Seismic demand shear force or base shear at the performance point based on IS: 1893 (Part 3)-2014 under DBE = 1122.86 kN $\not\sqsupset$ 1122.71 kN yield point.
   Demand moment = 11444.97 kNm $\not\sqsupset$ 11444.85 kNm yield moment.
   Demand displacement based on IS: 1893 (Part 3)-2014 under DBE = 0.013 m = 0.013 m yield displacement, which is acceptable under DBE.

3. Seismic demand shear force or base shear at the performance point based on IS: 1893 (Part 3)-2014 under MCE = 1129.35 kN > 1122.71 kN yield capacity and within immediate occupancy (1152.10 kN), which is acceptable.
   Demand moment = 11511.13 kNm > 11444.85 kNm yield moment and within immediate occupancy (11743.15 kNm), which is acceptable.
   Demand displacement based on IS: 1893 (Part 3)-2014 under MCE = 0.035 m > 0.013 m yield displacement and within immediate occupancy (0.112 m), which is acceptable.

According to the guidelines of IS: 1893-1984 (at the time of design) the bridge pier performance is within the yield point, as per IS: 1893 (Part 3)-2014 under DBE the bridge performance is at the yield point, which is acceptable. The performance of retrofitted bridge pier under MCE is in between the yield point and immediate occupancy, which is acceptable. The required strength is enhanced by reinforced concrete jacketing.

6.3 Fiber Reinforced Polymer (FRP) Jacketing

6.3.1 General

Fiber reinforced polymer wrapping or jacketing is one of the best methods with respect to effectiveness in performance and application. This method of jacketing is more efficient than the other jacketing methods such as concrete jacketing and steel jacketing as the later methods increase the dead load on the member. The advantages of using Fiber Reinforced Polymer (FRP) are as summarized below:

i. Strength of FRP is much higher and behavior is linearly elastic up to Failure.
ii. The load transfer from FRP to concrete is by adhesion of epoxy whereas in the case of reinforcing bars it is mechanical bond between bars and the concrete.
iii. The Fibers are flexible and thus can be molded into any fitting geometry, and it does not increase the dimensions of the member.
iv. Fibers exhibit high strength-to-weight ratio and are corrosion resistant.
v. Strength attainment in case of FRP is quicker as compared to other traditional methods.
vi. FRP technique can be used for any type of member such as beam, column, and slab.

Fiber Reinforced Polymer (FRP) is the term frequently used in civil engineering as high-strength composites. These composites are known for their high strength and stiffness. Retrofitting and repairing are the predominant areas were FRP is used. These composites were originally developed for the aircraft society. FRP composites are fabricated by embedding the high strength fibers in a resin matrix. This resin matrix provides the confining effect for the fibers and holds them together. Thus in a composite, the fibers provide the strength and stiffness whereas the matrix i.e. resin provides the transfer of stress and strain between the fibers. A fiber surface that is completely coated with the matrix achieves full composite action. A composite can also be made by adulterating two different types of fibers.

6.3.2 Types of FRP

The commonly used varieties of fiber reinforced polymers are as follows

a. Basalt fiber
b. Carbon fiber
c. Glass fiber

Basalt fibers: These fibers are made from the volcanic material deposits. They possess high strength, durability and thermal property. They serve as a better alternative to other high-temperature resistant fibers. Hence these fibers are typically used as hot shields, thermal and acoustic barriers. The cost of this fiber is also economical.

Carbon fibers: These fibers offer the highest modulus of all the reinforcing fibers. These fibers are manufactured from precursor sources like rayon, polyacrylonitrile and petroleum pitch. The process in which the precursor material is chemically changed into carbon fiber by the action of heat is defined as Carbonization. The disadvantage in this process of manufacture is its low yield and the problems created by the debris during manufacture.

Glass fibers: These fibers are commonly used in the engineering practices. Glass fibers are fabricated by fusing silicates with silica, potash lime and various metallic oxides. The fused molten mass is passed through micro-fine brushings and rapidly cooled to produce filaments of glass fiber. These filaments are drawn together and packed to obtain the composite. These fibers possess high tensile strength and are chemical resistant. The cost of the fiber is economical making it more practical to use.

6.3.3 FRP Confined Concrete Model

The behaviour of FRP confined concrete model is considered based on the study of Ozbakkalogu et al (2013).

The expression for peak axial compressive stress of FRP confined concrete (f'_{cc}) given by Mander et al (1988) as shown in Eq. 6.1.

$$f'_{cc} = f'_{co}(2.254\sqrt{1+\frac{7.94 f_l}{f'_{co}}} - 2\frac{f_l}{f'_{co}} - 1.254) \tag{6.1}$$

where,

f'_{co} is the peak axial compressive stress of unconfined concrete

f_l is the lateral confinement pressure provided by FRP given in Eq. 6.5.

The ultimate axial strain ε_{cc} at peak axial compressive stress of FRP confined concrete is shown below in Eq. 6.2.

$$\varepsilon_{cc} = 5\varepsilon_{co}\left(\frac{f'_{cc}}{f'_{co}} - 0.8\right) \qquad (6.2)$$

where,

ε_{co} is the axial strain of unconfined concrete at f'_{co}

The stress-strain equation of the confined concrete is given by Popovics et al (1973) energy balanced expression is shown below in Eq. 6.3.

$$f_c = \frac{f'_{cc}(\varepsilon_c / \varepsilon_{cc})r}{r - 1 + (\varepsilon_c / \varepsilon_{cc})^r} \qquad (6.3)$$

where,

f_c is the axial stress corresponding to a strain of ε_c

r is the constant and is shown below in Eq. 6.4.

$$r = \frac{E_c}{E_c - f'_{cc} / \varepsilon_{cc}} \qquad (6.4)$$

where,

Ec is the elastic modulus of concrete

The expression for lateral confinement pressure provided by FRP f_l is

$$f_l = \frac{2E_f t_f \varepsilon_f}{d} \qquad (6.5)$$

where,

E_f is the modulus of elasticity of FRP

t_f is the thickness of FRP

ε_f is the ultimate tensile strain of FRP

d is the diameter of column

According to ACI 440 recommendations, the expression for lateral confinement pressure provided by FRP f_l is

$$f_l = \frac{2E_f t_f \varepsilon_{fe}}{d} \tag{6.6}$$

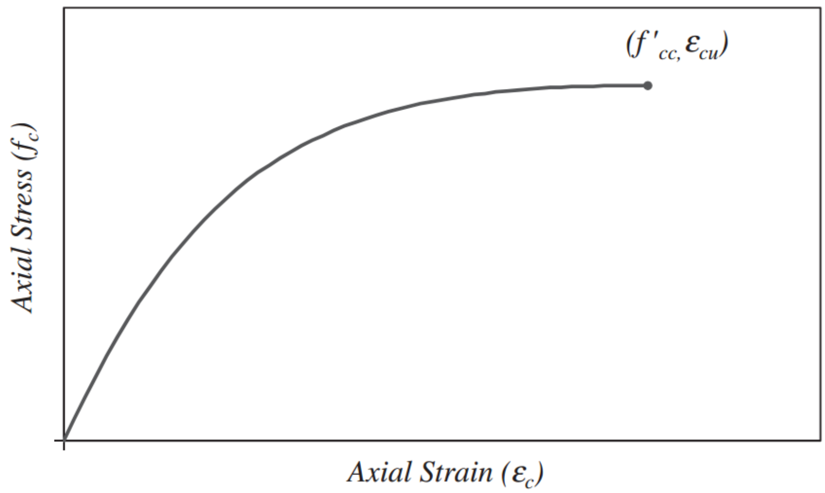
where,

ε_{fe} is the effective ultimate tensile strain of FRP and is given by

$$\varepsilon_{fe} = k_\varepsilon \varepsilon_f \leq 0.004 \tag{6.7}$$

where,

k_ε is the strain efficiency factor and for CFRP it is taken as 0.586



Stress–strain curve of FRP-confined concrete based on steel or actively confined concrete models

Figure 6.7 Stress-strain curve of FRP confined concrete (Ozbakkaloglu et al., 2013)

In this study, the bridge pier is retrofitted with carbon fiber reinforced polymer jacketing (CFRP). The properties of the CFRP have been employed from ACI 440-2R, page 60 (2008).

Figure 6.8 Carbon fiber reinforced polymer jacketing of bridge bent

6.3.4 Carbon Fiber Reinforced Polymer (CFRP) Jacketing of the Dhaleswari Bridge Pier

Grade of concrete used in the Pier is M 25.

The diameter of the pier = 3.0 m

The properties of the CFRP (ACI 440-2R, page 60)

Ultimate tensile strength = 3792 kN/mm^2

Ultimate axial strain = 0.0167 mm/mm

Initial stiffness / modulus of elasticity = 227.527 GPa

Thickness per ply = 0.33 mm

Thickness of CFRP provided = 3.3 mm

Peak compressive stress in CFRP confined concrete = 36.707 MPa

Ultimate axial strain of CFRP confined concrete = 0.00668

The 3D finite element model of the retrofit Dhaleswari bridge pier with the CFRP jacket is developed in SAP2000. The CFRP material property has been defined manually by using other material type. The CFRP confined concrete stress-strain curve has been defined manually in SAP2000 as shown in Fig. 6.9.

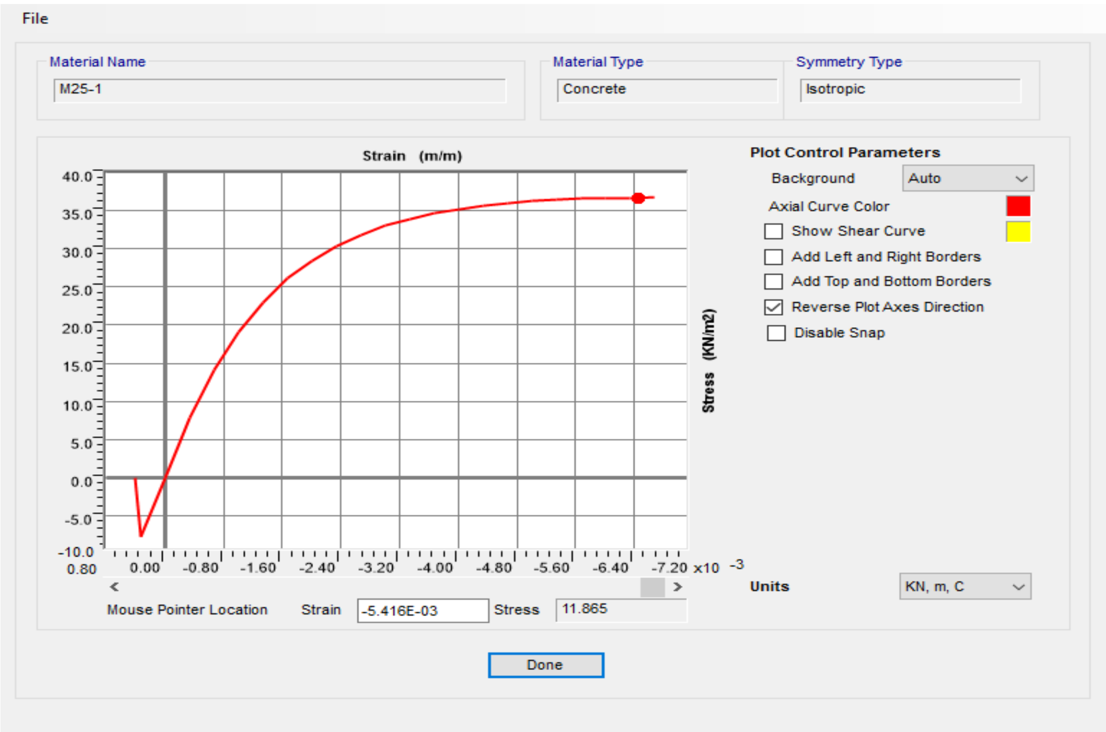


Figure 6.9 CFRP confined concrete stress-strain curve defined in SAP2000

Modal analysis has been done to find dynamic characteristics of bridges such as time periods and modal mass participation factors at different modes.

Table 6.7 Time periods and modal mass participation factors for CFRP jacketed Dhaleswari bridge pier

Step Type	**Step No.**	**Time Period (s)**	**UX**	**UY**	**UZ**	**Sum UX**	**Sum UY**	**Sum UZ**
Mode	1	0.4703	0.9963	0.0003	0.0000	0.9963	0.0003	0.0000
Mode	2	0.4703	0.0003	0.9963	0.0000	0.9966	0.9966	0.0000
Mode	3	0.0320	0.0000	0.0000	1.0000	0.9966	0.9966	0.9999
Mode	4	0.0163	0.0034	4.6E-06	0.0000	1.0000	0.9966	0.9999
Mode	5	0.0163	4.6E-06	0.0034	0.0000	1.0000	1.0000	0.9999
Mode	6	0.0026	0.0000	0.0000	8.5E-06	1.0000	1.0000	1.0000

Table 6.8 Comparison of time periods for the existing and CFRP jacketed Dhaleswari bridge pier

Mode No.	**For Existing Pier Time Period (s)**	**For CFRP Jacketed Pier Time Period (s)**
1	0.4736	0.4703
2	0.4736	0.4703
3	0.0321	0.0320
4	0.0165	0.0163
5	0.0165	0.0163
6	0.0026	0.0026

For performing the non-linear static analysis, material non-linearity and geometrical non-linearity has been considered. The hinges are assigned at the base of piers. The capacity curve of the Dhaleswari bridge pier with CFRP jacket obtained from the pushover analysis is shown below in Fig. 6.10.

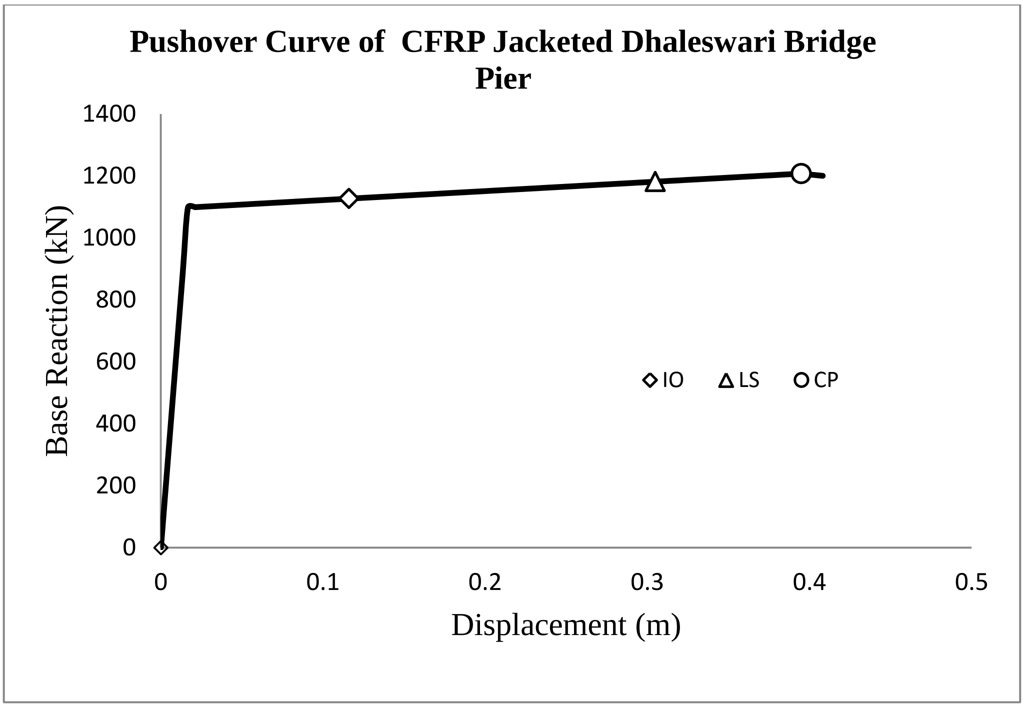


Figure 6.10 Pushover curve of CFRP jacketed Dhaleswari bridge pier

From the capacity curve and the hinge properties the resulting value of shear force, bending moment and displacements corresponding to the various acceptance criteria is shown below in Table 6.9.

Table 6.9 Capacity of CFRP jacketed Dhaleswari bridge pier at various acceptance points

Acceptance Criteria	**Base Shear (kN)**	**Bending Moment (kNm)**	**Displacement (m)**
At Yield Point	1098.04	11375.85	0.017
At Immediate Occupancy	1126.74	11673.20	0.116
At Life Safety	1181.53	12240.85	0.305
At Collapse Prevention	1207.63	12511.17	0.395

The performance point of the CFRP jacketed Dhaleswari bridge pier is obtained by using ATC 40 Capacity Spectrum method corresponding to the demand spectrums of IS: 1893-1984, IS: 1893 (Part 3)-2014 under DBE and MCE. The results are as follows:

1. Seismic demand shear force or base shear at the performance point based on IS: 1893-1984 = 438.03 kN < 1098.04 kN yield point.

Demand moment based on IS: 1893-1984 = 4537.99 kNm < 11375.85 kNm yield moment.

Demand displacement based on IS: 1893-1984 = 0.006 m < 0.017 m yield displacement.

2. Seismic demand shear force or base shear at the performance point based on IS: 1893 (Part 3)-2014 under DBE = 1094.59 kN < 1098.04 kN yield point.

   Demand moment based on IS: 1893 (Part 3)-2014 under DBE

   = 11340.00 kNm < 11375.85 kNm yield moment.

   Demand displacement based on IS: 1893 (Part 3)-2014 = 0.017 m = 0.017 m yield displacement, which is acceptable under DBE.

3. Seismic demand shear force or base shear at the performance point based on IS: 1893 (Part 3)-2014 under MCE = 1101.29 kN > 1098.04 yield capacity and within immediate occupancy point (1126.74kN) hence ok.

   Demand moment = 11409.36 kNm > 11375.85 kNm yield moment and within immediate occupancy (11673.20kNm), which is acceptable.

   Demand displacement based on IS: 1893 (Part 3)-2014 = 0.028 m > 0.017 m yield displacement and within immediate occupancy point (0.116 m) hence ok.

According to the guidelines of IS: 1893-1984 (at the time of construction) and IS: 1893 (Part 3)-2014 under DBE the bridge performance is within the yield point, hence it is ok. The performance of retrofitted bridge pier under MCE is between the yield point and immediate occupancy pint, which is acceptable. The required strength is enhanced by CFRP jacketing.

CHAPTER 7

RESULTS AND DISCUSSIONS

7.1 Ringhal Khad Bridge Pier

1. Shear force obtained from the response spectrum method based on IS: 1893(Part 3)-2014 for DBE condition is more than IS 1893-1984 by 1.45 times. And, it is based on IS: 1893(Part 3)-2014 for MCE condition is more than IS 1893-1984 by 2.90 times.

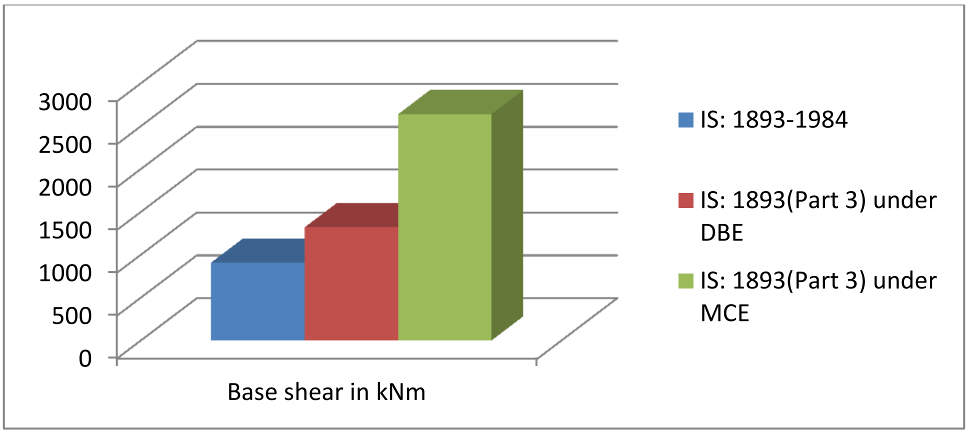


Figure 7.1 Comparison of response spectrum analysis results

2. The performance level based on IS: 1893-1984, of the bridge pier is within the yield capacity. It is based on IS: 1893(Part 3)-2014 for DBE condition is within the IO. It is based on IS: 1893(Part 3)-2014 for MCE condition is in between the LS and CP. So, the pier has to be retrofitted to improve the seismic performance for MCE condition.

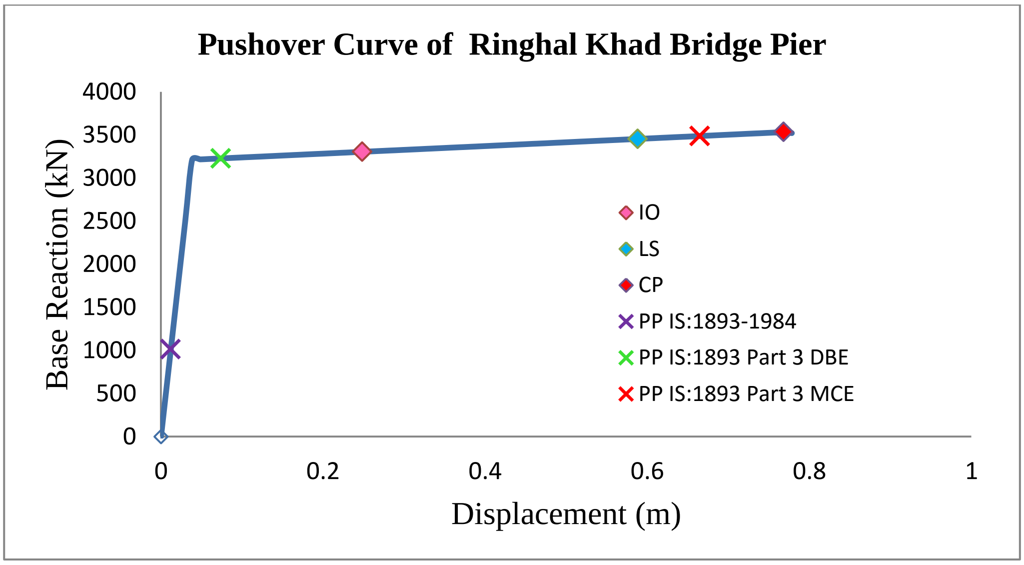


Figure 7.2 Performance levels of Ringhal Khad bridge pier

3. The performance level based on IS: 1893-1984 and IS: 1893(Part 3)-2014 for DBE condition of the RC jacketed bridge pier is within the yield capacity. Based on IS: 1893(Part 3)-2014 for MCE condition is within the immediate occupancy. The required strength is enhanced by RC jacketing.

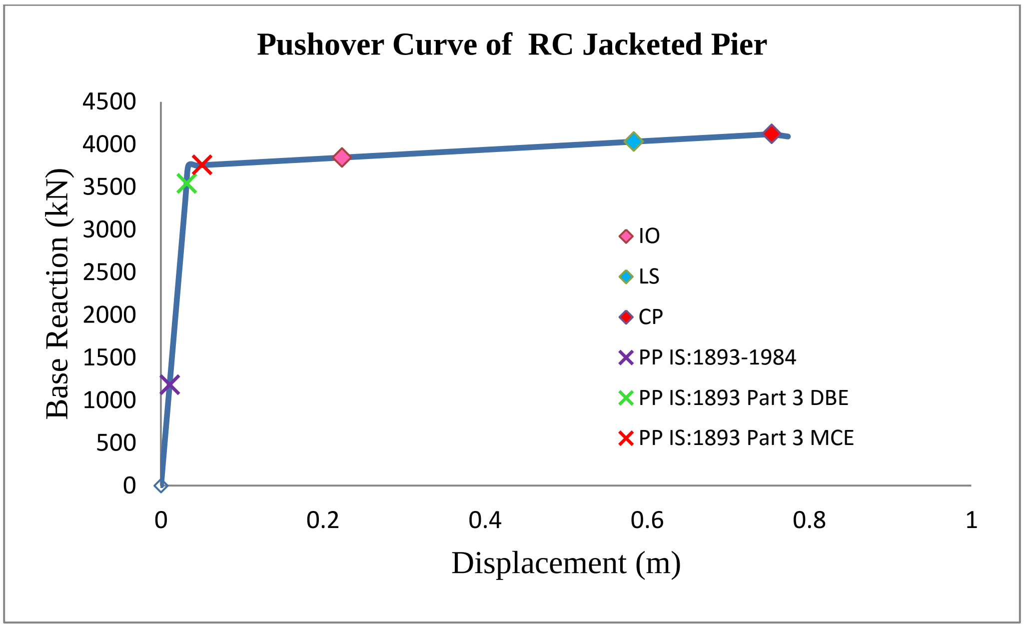


Figure 7.3 Performance levels of RC jacketed Ringhal Khad bridge pier

4. The required strength is enhanced by both the retrofitting techniques. Figure 7.5 shows the pushover curves of existing, RC jacketed and CFRP jacketed Ringhal Khad bridge pier.

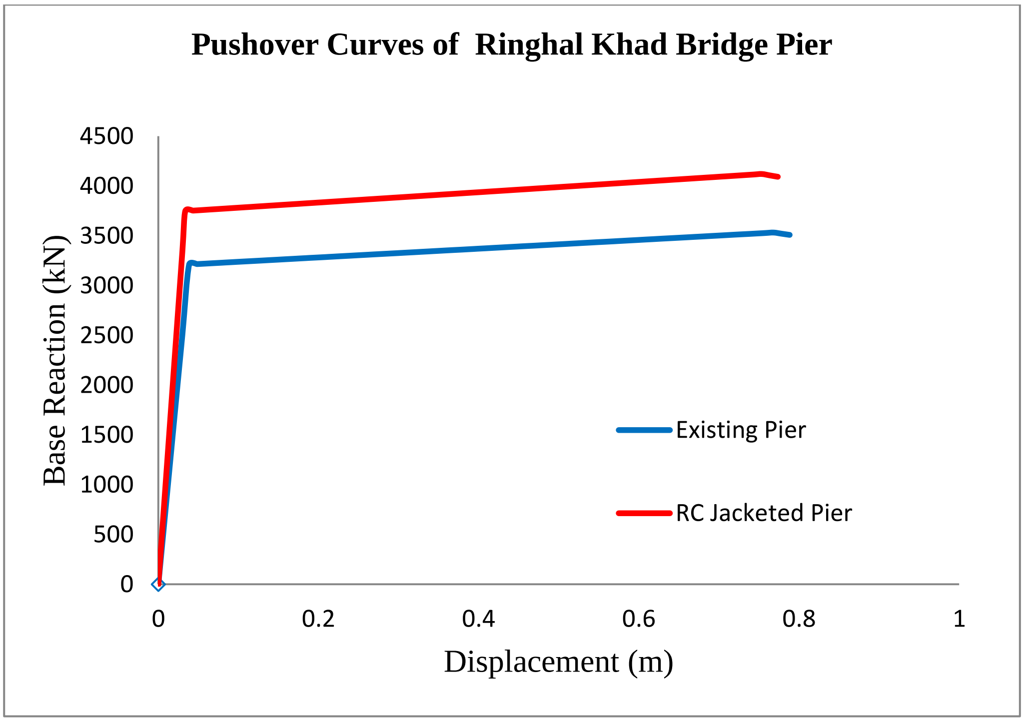


Figure 7.4 Pushover curves of retrofitted Ringhal Khad bridge pier

7.2 Dhaleswari Bridge Pier

1. Shear force obtained in response spectrum method based on IS: 1893(Part 3)-2014 for DBE condition is more than IS 1893-1984 by1.34 times. And, it is based on IS: 1893(Part 3)-2014 for MCE condition is more than IS 1893-1984 by 2.69 times.

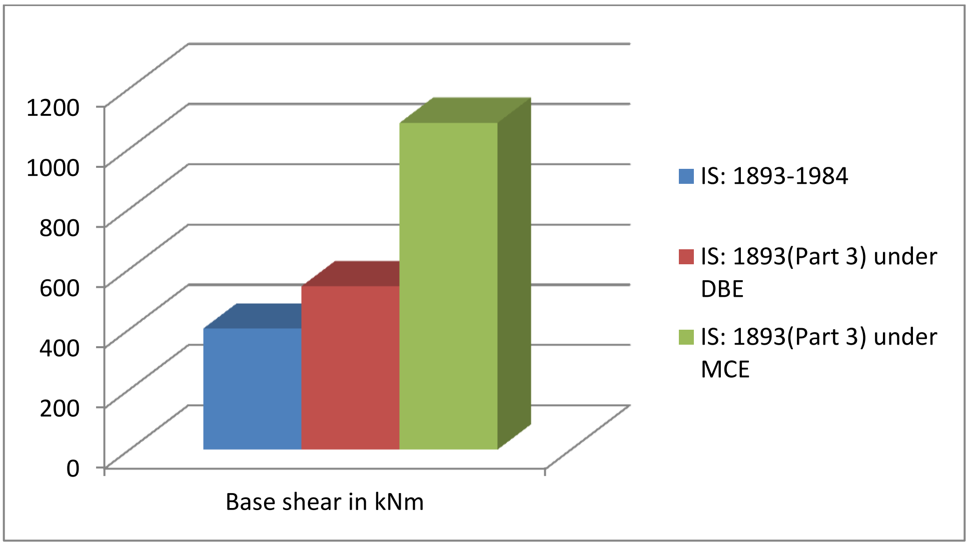


Figure 7.5 Comparison of response spectrum analysis results

2. The performance level based on IS: 1893-1984, of the bridge pier is within the yield capacity. It is based on IS: 1893(Part 3)-2014 for DBE condition is within the IO. It is based on IS: 1893(Part 3)-2014 for MCE condition is in between the LS and CP. So, the pier has to be retrofitted to improve the seismic performance for MCE condition.

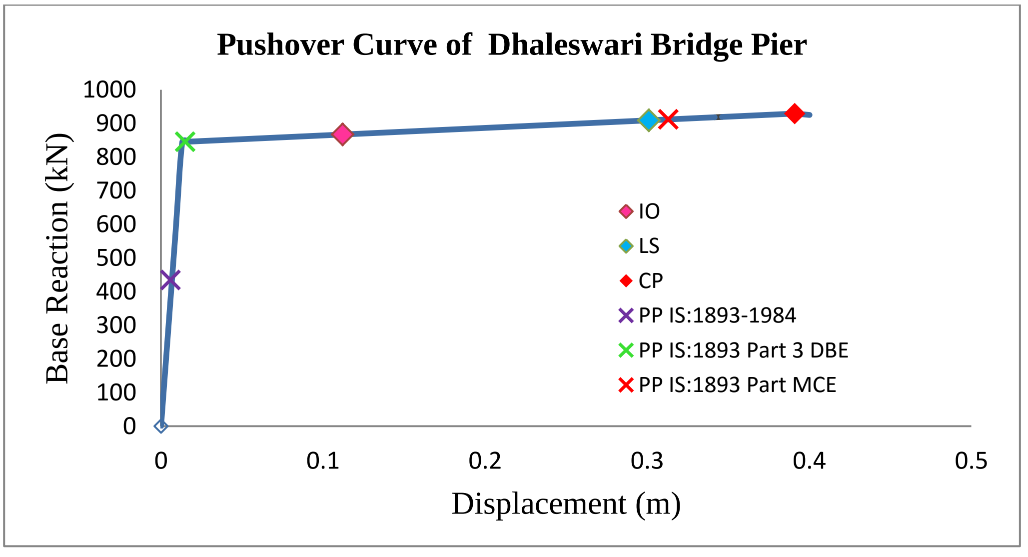


Figure 7.6 Performance levels of Dhaleswari bridge pier

3. The performance level based on IS: 1893-1984 of the RC jacketed bridge pier is within the yield capacity. Based on IS: 1893(Part 3)-2014 for DBE condition it is at the yield capacity. And, based on IS: 1893(Part 3)-2014 for MCE condition is within the immediate occupancy. The required strength is enhanced by reinforced concrete jacketing.

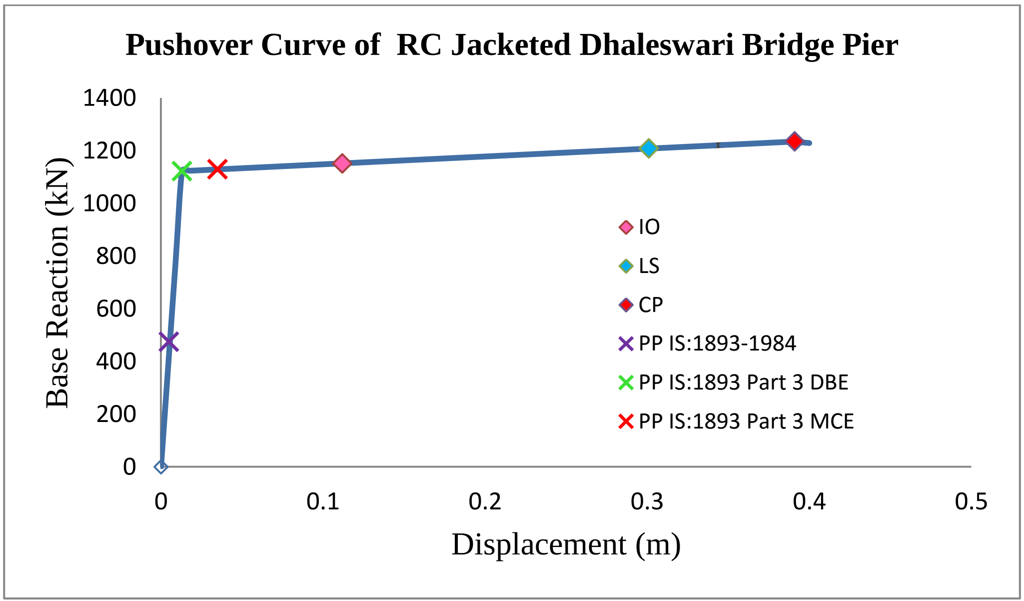


Figure 7.7 Performance levels of RC jacketed Dhaleswari bridge pier

4. The performance level based on IS: 1893-1984 and IS: 1893(Part 3)-2014 for DBE condition of the CFRP jacketed bridge pier is within the yield capacity. Based on IS: 1893(Part 3)-2014 for MCE condition is within the immediate occupancy. The required strength is enhanced by CFRP jacketing.

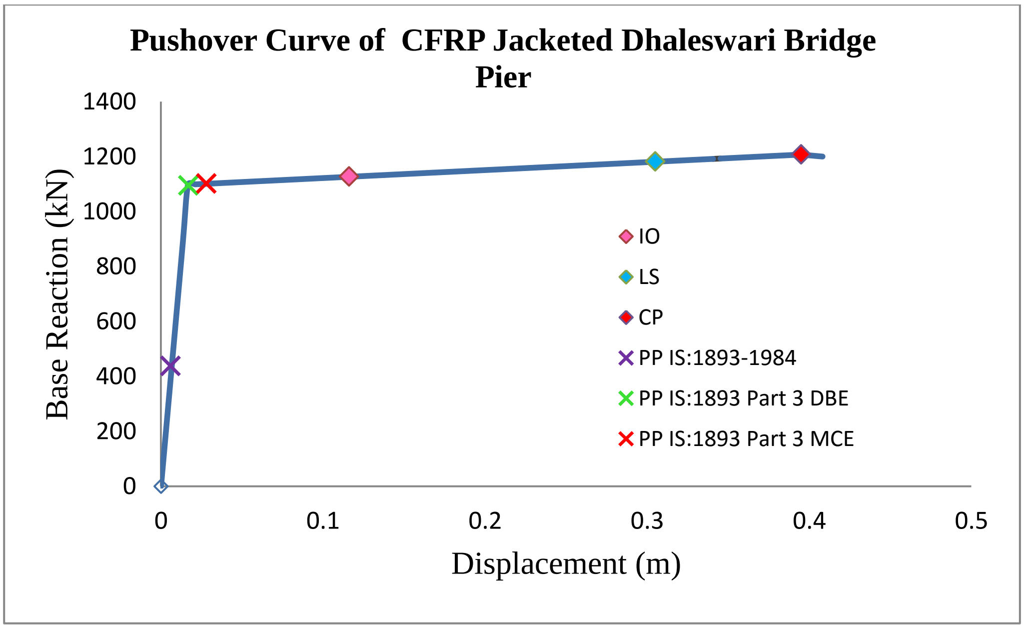


Figure 7.8 Performance levels of CFRP jacketed Dhaleswari bridge pier

5. The required strength is enhanced by both the retrofitting techniques. Figure 7.5 shows the pushover curves of existing, RC jacketed and CFRP jacketed Dhaleswari bridge pier.

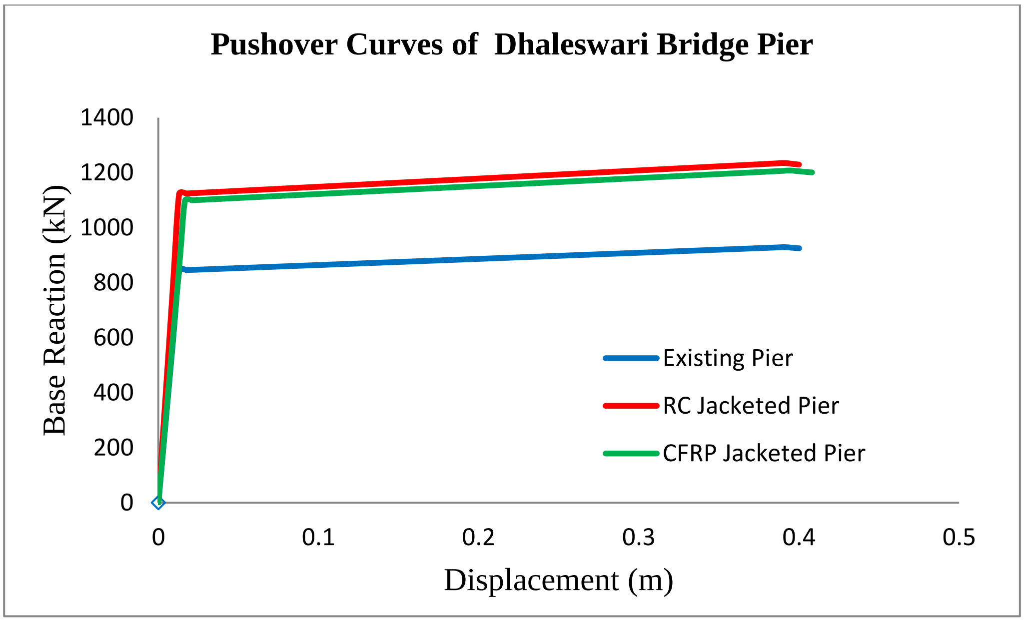


Figure 7.9 Pushover curves of retrofitted Dhaleswari bridge pier

CHAPTER 8

CONCLUSIONS

Two bridge substructures have taken for the present study. The first bridge which lies in seismic zone IV has hollow circular pier, while the second bridge falls in seismic zone V having solid circular pier. The finite element models of two bridge substructures have been created in SAP2000. The response spectrum analysis, nonlinear static analysis and capacity spectrum methods have been employed to determine the base shear, capacity and demand respectively. Seismic deficiency has been observed in the bridge piers by comparing the seismic demand and capacity. Seismic performance of the bridge piers have been improved by retrofitting techniques. Based on the foregoing study following conclusions are drawn.

1. It has been observed that the base shear evaluated for the Ringhal Khad and Dhaleswari bridge piers, from response spectrum analysis based on IS: 1893 (Part 3)-2014 under MCE is 2.9 and 2.69 times more than IS: 1893-1984, respectively.
2. The performance level based on IS: 1893-1984, of both the bridges are observed within the yield point.
3. The performance level based on IS: 1893 (Part 3)-2014 for DBE condition, of both the bridges are observed within the limits of IO.
4. The performance level based on IS: 1893 (Part 3)-2014 for MCE condition, of both the bridges are in between LS and CP.
5. Ringhal Khad bridge pier is RC jacketed with increase in diameter from 6.5 m to 6.7 m. Because of increase in stiffness, the time period in the fundamental mode is reduced from 1.314 sec to 1.189 sec.
6. Dhaleswari bridge pier is RC jacketed with increase in diameter from 3.0 m to 3.2 m. Because of increase in stiffness, the time period in the fundamental mode is reduced from 0.473 sec to 0.423 sec.
7. In CFRP jacketing, since there is no change in cross section of the Dhaleswari bridge pier, there is hardly any difference in time periods.
8. In the Ringhal Khad and Dhaleswari bridge piers, due to the RC jacketing the flexural strength is enhanced by 17% and 32 % respectively.

9. In the Dhaleswari bridge pier, due to the CFRP jacketing the flexural strength is enhanced by 30%.
10. It has been observed that the retrofitting enhances the performance levels based on IS: 1893 (Part 3)-2014 for MCE condition of both the bridges (within IO).

REFERENCES

Asaei S. M. S, Lau T. L, Bunnori N.M, "Modelling FRP-confined RC Columns using SAP2000", *World Applied Sciences Journal* 27 (12): 1717-1763, 2013.

ASCE/SEI 41-13, "Seismic Evaluation and Retrofit of Existing Buildings", *American Society of Civil Engineers (ASCE),* Virginia, 2015.

ATC-40, "Seismic Evaluation and Retrofit of Concrete Buildings", *Applied Technology Council (ATC),* California, 1996.

Ballard T. A, Sedarat H, "SR5 Lake Washington Ship Canal Bridge Pushover analysis", *Computers and structures* 72(1999) 63-80, 1999.

Bignell J. L, Lafave J. L, Hawkins N. M, "Seismic Vulnerability Assessment of Wall Pier supported highway bridges using Nonlinear Pushover Analysis", *Engineering structures* 27(2005) 2044-2063, 2005.

Billah A. H. M. M, Alam M. S, "Seismic performance evaluation of multi-column bridge bents retrofitted with different alternatives using incremental dynamic analysis", *Engineering structures* 62-63 (2014) 105-117, 2014.

Chiorean C. G, "Application of Pushover Analysis on Reinforced Concrete Bridge model", *Project*/36019/99, July 2003.

Chopra A. K, Goel R. K, "Capacity-Demand-Diagram methods for estimating seismic deformation of Inelastic Structures", *Pacific Earthquake Engineering Research Center,* Report No. PEER-02, 1999.

FEMA 356, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings", *Federal Emergency Management Agency,* Washington, D. C., 2000.

IS 15988: 2013, "Seismic evaluation and strengthening of existing reinforced concrete buildings-guidelines", *Bureau of Indian Standards,* New Delhi, 2013.

IS: 1893-1984, "Criteria for Earthquake Resistant Design of Structures (Fourth Revision)", *Bureau of Indian Standards,* New Delhi, 2003.

IS 1893 (Part1):2002, "Criteria for Earthquake Resistant Design of Structures-Part1, General Provisions and Buildings (fifth revision)", *Bureau of Indian Standards,* New Delhi, 2002.

IS: 1893 (Part3)-2014, "Criteria for Earthquake Resistant Design of Structures-Part 3, Bridges and Retaining Walls", *Bureau of Indian Standards,* New Delhi, 2014.

Lam L, Teng J, “Design-oriented stress-strain model for FRP-confined concrete in rectangular columns”, *Journal of Reinforced Plastics and Composites,* 22 (13): 1149-1186, 2003.

Mander J. B, Priestley M. J. N, Park R, “Theoretical stress-strain model for confined concrete”, *Journal of structural engineering*, 114(8): 1804-1826, 1988.

Ozbakkaloglu T, Lim J. C, Vincent T, “FRP-confined concrete in circular sections: Review and assessment of stress–strain models”, *Engineering structures*, 49 (2013) 1068-1088, 2013.

Pantelides C. P, Gergely J, “Carbon-fiber-reinforced polymer seismic retrofit of RC bridge bent: design and in situ validation”, *Journal of Composites for Construction*, 6(1):52–60, 2002.

Rahai A, Gorji B. A, Hatami F, “Seismic Evaluation of Prestressed Concrete Bridges by using DCM, CSM and Spectrum Analysis”, *World Applied Sciences Journal* 8(2): 210-216, 2010.

Ruiz L. L, Diaz M. J, Guerrero J. M. J, Barrios H. H, “Seismic Assessment Analysis of bridges in Mexico”, *14th world conference on Earthquake Engineering*, Beijing, China, 2008.

Seible, F., Priestley, M. J. N., Hegemier, G., and Innamorato, D, ‘‘Seismic retrofitting of RC columns with continuous carbon fiber jackets.’’, *Journal of Composites for Construction*, 1(2), 52–62, 1997.

Thakkar S. K, Dubey R. N, “Seismic Analysis of Bridge at Ringhal Khad on Jammu-Udhampur Rail-Link”, *Project Report, Department of Earthquake Engineering, University of Roorkee,* Roorkee, EQ 98-12, 1998.

Thakkar S. K, Gupta M. K, “Dynamic Analysis of Railway Bridge No. 69 at Dhaleswari”, *Project Report, Department of Earthquake Engineering, University of Roorkee,* Roorkee, EQ 84-11, 1984.