iJRASET
International Journal For Research in
Applied Science and Engineering Technology

INTERNATIONAL JOURNAL FOR RESEARCH
IN APPLIED SCIENCE & ENGINEERING TECHNOLOGY

Volume: 5 Issue: VII Month of publication: July 2017

DOI:

International Journal for Research in Applied Science & Engineering Technology (IJRASET)
ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor:6.887
Volume 5 Issue VII, July 2017- Available at www.ijraset.com

Seismic Assessment of an Existing Non Seismically Designed Major Bridge

R. Regupathi[1], R. Prakash[2]
[1]Department of Civil Engineering, Government College of Engineering, Bodi-625 582, Tamilnadu, India
[2] Department of Civil Engineering, A.C.College of Engineering and Technology,Karaikudi-630 003, Tamilnadu, India

Abstract: The extensive damages to bridges world over in earthquakes have generated considerable interest amongst the engineers and researchers on the seismic design of new bridges and retrofitting of existing ones. Prior to the development of modern codes, the bridges were designed for nominal seismic forces without provisions of ductility. Moreover, even many recently constructed bridges may become seismically deficient if the corresponding seismic codal provisions are revised to a better standards. A comprehensive study carried out to assess the seismic response of a 29 span existing bridge and its foundation system located across Palar river in Vellore district of Tamilnadu state using a refined inelastic modelling approach is summarised in this project. The bridge represents a typical case of vulnerable bridges since it was constructed in the early eighties and designed for minimal seismic requirements. A series of three dimensional dynamic response simulations of the bridge were assessed using developed Finite Element SAP2000 models. The performance based assessment study was employed by performing inelastic time history, response spectrum and pushover analysis, to identify areas of vulnerability in the 590.64m bridge at various hazard levels. The seismic demands of the interior spans significantly increases and considerably exceeds the serviceability limit states. The results clearly reflect the benefit of retrofitting different bridge components to mitigate the anticipated seismic risk. The presented assessment study contributes to improve the public safety by predicting the seismic response of deficient highway bridges, leading to reliable and cost effective retrofit strategies.
Keywords: Retrofitting, Seismic Demands, Finite Element, Dynamic Response, Time History

I. INTRODUCTION

The bridges constructed prior to 1970 were not designed for adequate seismic resistance as the ductility provisions were not incorporated in the seismic codes till then. As a result the bridges designed before 1970 significantly lack earthquake resistance and ductility and may be vulnerable to significant damages even under moderate earthquakes. The post-earthquake damage surveys of recent earthquakes have confirmed this. It has also been revealed from damage surveys that many of the damages that occurred in bridges and flyovers could be prevented by proactive measures of seismic retrofitting prior to earthquakes or by seismic assessment based retrofitting on codal revision. Many of the reinforced concrete piers designed based on the earlier codes found to have inadequate shear capacity due to lack of transverse steel confinement, inadequately lap length for longitudinal steel, and premature termination of longitudinal steel. The superstructures were vulnerable to fall down in the absence of restraining devices, bearings were deficient in accommodating large seismic displacement and bearing seat was inadequate. These deficiencies lead to an adverse impact on the performance of bridges. An existing bridge can be replaced a newly designed bridge to meet earthquake demands or upgraded in its strength by appropriate retrofitting measures. The retrofitting measures are often an economical solution than complete reconstruction.

II. DESCRIPTION OF THE PALAR BRIDGE IN VELLORE DISTRICT

To study and evaluate the seismic deficiency of a bridge system and to undertake necessary retrofitting measures so as to make the bridge system capable of resisting the seismic force as per current codal provisions, a case study bridge has been selected. The case study bridge is located across river Palar and lies at km 0/6 of old Palar bridge road in Vellore district of Tamil Nadu state. The traffic bound for Andhra Pradesh, Karnataka states and the local traffic to Katpadi and other northern villages use the Cuddalore Chittoor road, which runs parallel to the Old Palar bridge road, at approximately 300m on the downstream side of the bridge site. The bridge was aligned and cross the river Palar in perpendicular.

A. Hydraulic & Sub Soil Particulars

The hydraulic particulars pertaining to river and bridge are given below:

Discharge	:	4222.5 cumecs (or 149118 cusecs) at site.
MFL	:	RL 99.965
Bed level	:	RL 97.400
Bed width available	:	625.00 m
Bottom of deck	:	RL 101.660
Linear water way	:	590 m

The sub soil of the bridge site is consisting of sand, clayey sand, gravel for an average depth of 7 m below the sill level. Underneath, weathered rock is available for a depth of about 1 m. Below this, hard rock is available.

B. Structural Details of Existing Palar Bridge

The detailed drawing of the existing Palar bridge showing the cross section details is shown in Figure 3.1 and the details of the structural components are listed below

Type of bridge	:	High level RC Deck girder major bridge
Width of carriage way	:	7.50 m
Camber	:	2.5 %
Width of footpath	:	2.250 m on both sides (including crash barrier and kerb)
Overall width of bridge	:	12.00m
Type of super structure	:	RCC T beam cum slab super structure
Expansion joint	:	21.60 m c/c
Effective span	:	21.00 m
Number of spans	:	29
Thickness of Deck Slab	:	336mm
Depth of the I Girder	:	1830mm
Linear water way	:	590.64m
Type of foundation	:	Bored cast in situ piles of 1.2m dia and pile cap of size 5.9m X 4m in RCC M35 for abutments and piers.
Type of substructure	:	Abutment and Twin circular column piers of 1.2m diameter with capping beam of 1.50m X 1.20m size in RCC M35.
Maximum flood level	:	RL 99.965
Bottom of deck level	:	RL 101.660
Sill level	:	RL 97.000
Vertical clearance	:	1.545 m
Afflux	:	0.15 m
Depth of superstructure	:	2166 mm at the centre and 2030 mm at edges
Road level	:	RL 103.901
Wearing coat	:	75 mm uniform thick in PCC M30
Expansion joint:40 mm		
Design loading	:	One lane of class 70 R or Two lanes of Class A loading as per IRC 6 – 2000
Condition of exposure	:	Moderate
Seismic zone	:	Zone – II

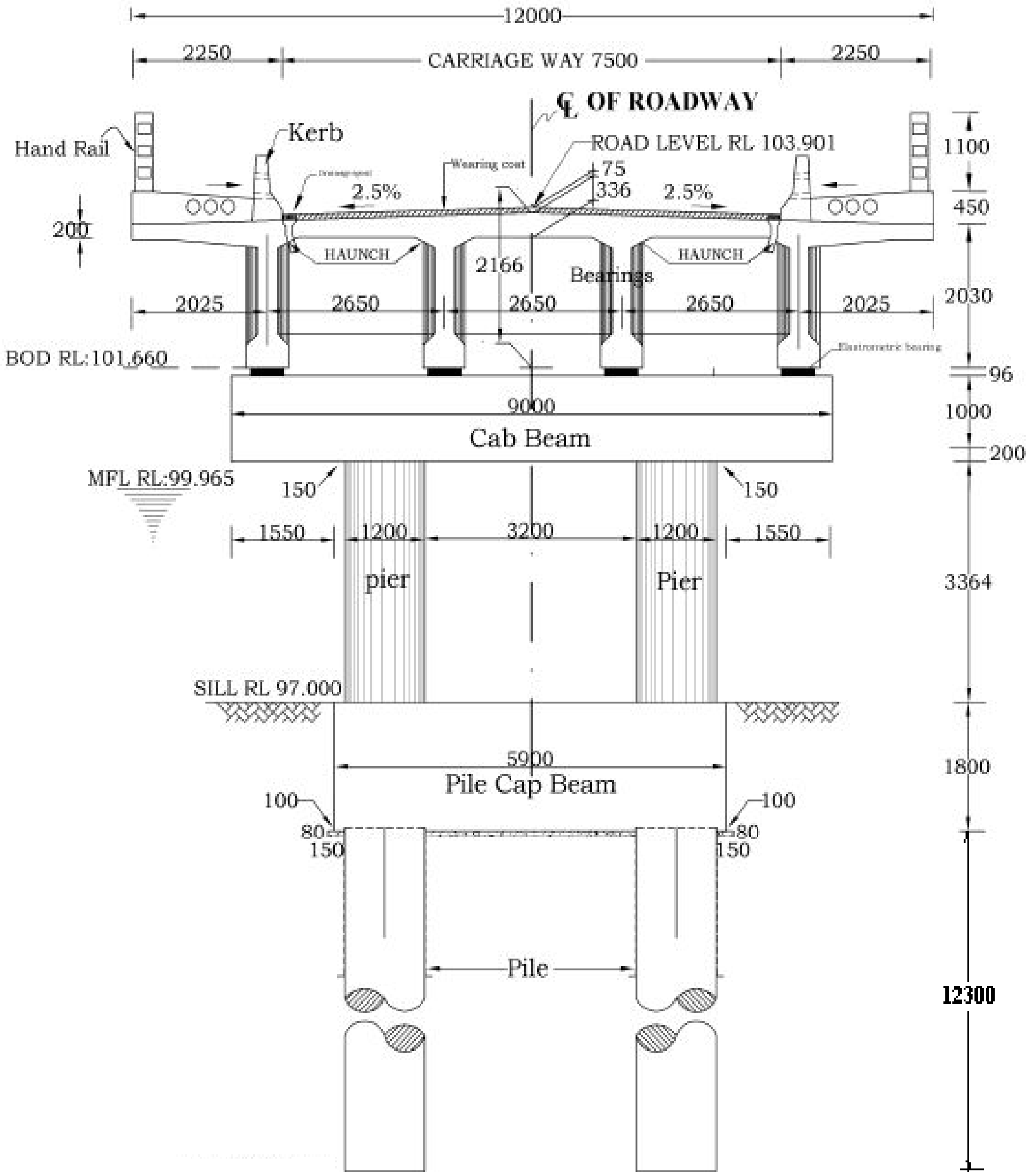


Figure 2.1 Cross Section of the Existing Palar Bridge

III. DEVELOPMENT OF SIMULATION MODELS FOR INELASTIC ANALYSIS

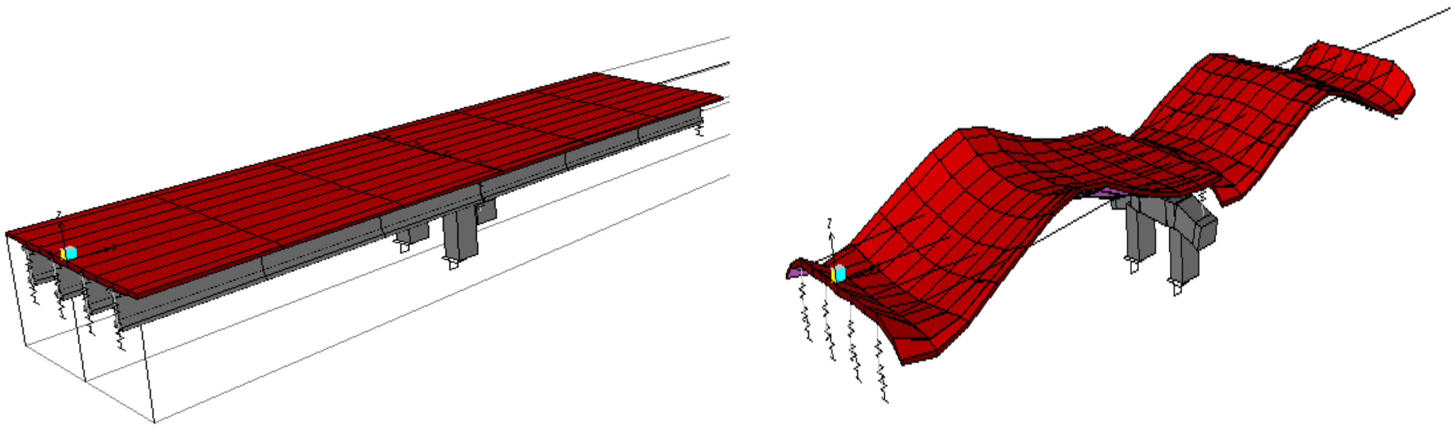
Detailed three dimensional dynamic response simulations of the bridge and its foundation system were conducted by developing refined inelastic finite element SAP2000 models. Four models were created for analysing end spans along the span next to the end spans and interior spans. Based on the available cross sections in the library of SAP2000 equivalent cross sections were adopted for modelling of the bridge members like cap beam, columns, I girders, deck sections, column bents and bearings on abutments. From material properties library of concrete and reinforcing steel the stress strain curves were specified to reflect nonlinear behaviour. The finite element analysis program of SAP2000 is employed for elastic and inelastic analysis of the structure. The spans of the bridge was discretised in to several elements and the stress-strain response at each element was monitored during the entire multi step analysis. Equivalent gravity loads and masses were distributed on the superstructure and along the piers height. The layout lines are straight, with no variation in elevation. The Model-I was developed for analyzing the end span of two span bridge. Similarly

International Journal for Research in Applied Science & Engineering Technology (IJRASET)
ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor:6.887
Volume 5 Issue VII, July 2017- Available at www.ijraset.com

Model-II, Model-III and Model-IV were developed for analysing three spans, four spans and five spans respectively as shown in Figure 4.1 to 4.4. The other details of the Models are given in the Table 4.1.

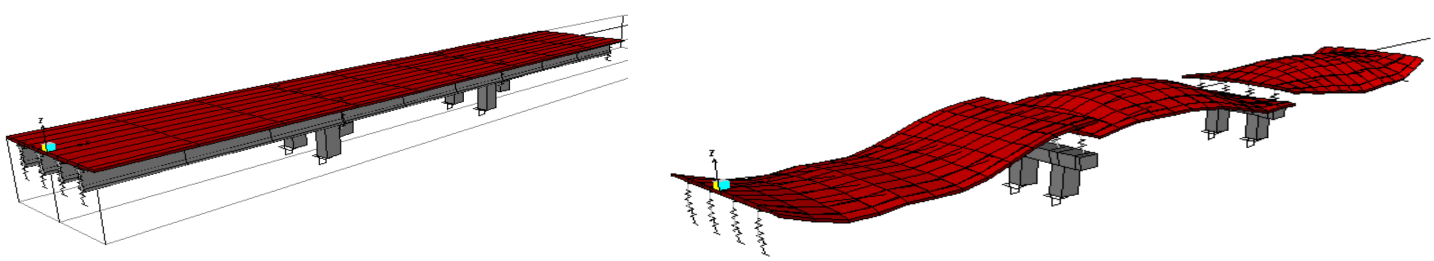
Table 3.1 Details of the Bridge Models

SL.No	Description of the Models	Model Name	No of spans	Span	
				Each bay	Total
1	Start abutment-one bent-end abutment	Model-I	2	21.6m	43.2m
2	Start abutment-two bents-end abutment	Model-II	3	21.6m	64.8m
3	Start abutment-three bents-end abutment	Model-III	4	21.6m	86.4m
4	Start abutment-four bents- end abutment	Model-IV	5	21.6m	108 m



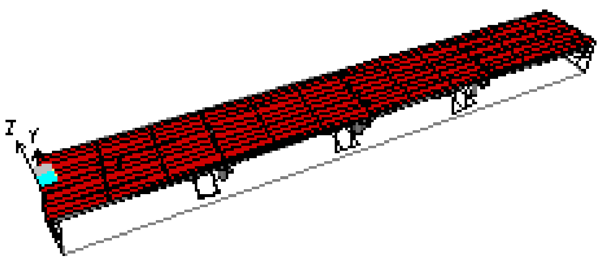
(a) Detailed SAP 2000 Model (b) Animated 12th Mode of vibration

Figure 4.1 Model –I Two Span Bridge

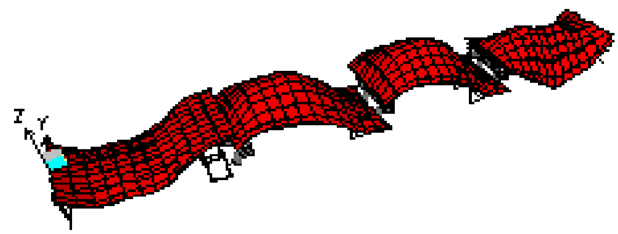


(a) Detailed SAP 2000 Model (b) Animated 12th Mode of vibration

Figure 3.2 Model –II Three Span Bridge

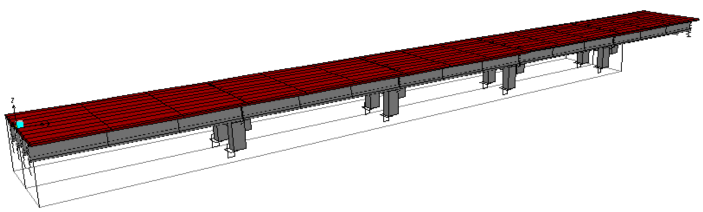


(a) Detailed SAP 2000 Model

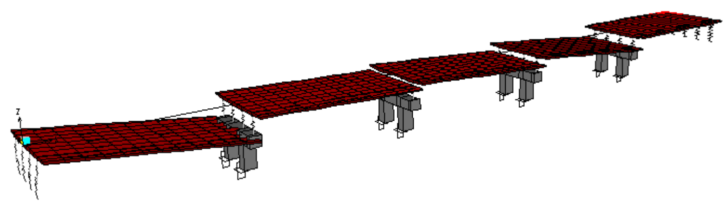


b) Animated 12th Mode vibration

Figure 3.3 Model –III Four Span Bridge



(a) Detailed SAP 2000 Mode



(b) Animated 1st Mode of vibration

Figure 3.4 Model –IV Five Span Bridge

A. *Material Property Definitions*

The ductile behaviour of a plastic hinge is significantly affected by the nonlinear material property used to define the frame member receiving the hinges. The material nonlinear properties have been defined using the Advanced Nonlinear Material Data forms. For concrete of 4000Psi (27.59N/mm^2), the nonlinear material property data form is shown in Figure 4.5. Similarly, for steel of A615Gr60, the nonlinear material data form is shown in Figure 4.6. The parametric strain data was specified, which includes the values for the strain at the onset of hardening, ultimate strain capacity, and the final slope of the stress-strain diagram.

Figure 3.5 Concrete Material Property

Figure 3.6 Non Linear Stress Strain Plot of Reinforcement Steel

International Journal for Research in Applied Science & Engineering Technology (IJRASET)
ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor:6.887
Volume 5 Issue VII, July 2017- Available at www.ijraset.com

Four frame section properties have been described to develop the models. The four types of frame elements used in the models consists of pile, bent cap beam, bent column and precast concrete I girder. A section property definitions for each of the element were defined from the material properties library of SAP 2000 as given in Table3.2. The section properties form for the precast I girder is shown in Figure 3.7.

Table 3.2 Section Property Definitions of the Bridge Elements

SL. No	Name of the element	Size of the element in mm	Reinforcement in mm	
			Main	Confined
1	Cap beam	1500 X 1200	Top-12 nos. 30 dia Bottom-12 nos. 30 dia	i). Both side face-8nos 12 dia ii). 8dia @190 c/c
2	Bent column	1200 dia	22 nos. 25 dia	8 dia @ 190 c/c
3	Piles	1200 dia	24 nos. 16 dia	8 dia @ 190 c/c (spiral type)

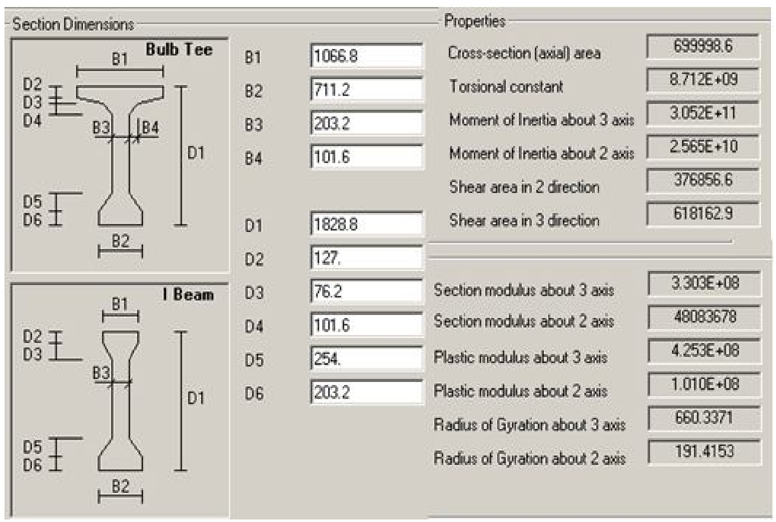


Figure3.7 Section Properties Form of the Precast Concrete I Girder

The bridge deck section is of 12m wide with a total of four I girders as shown in Figure 4.8. The parapets as well as wearing surface were not part of the bridge deck structural definition but was added to the bridge model as superimposed dead loads (SDEAD).The bents for the subject model have two columns each with a cap beam width of 9m. Since multiple columns were specified, the location, height and support condition for each column needed to be specified using the Bent Column Data form, which was accessed using the Modify/Show Column Data button. The type, location, height, angle and boundary conditions for each bent column were defined. Although the foundations can be represented as fixed, pinned, or spring-support restraints at the base of the columns, these have been explicitly modelled. Although the foundation objects were part of the bridge model, the base of the bent column must not be restrained, but instead, connected to the foundation elements. Restraining the base of the columns in the Bent Column Data form using fixed or pinned restraints would prevent the bridge loads from reaching the foundation. A foundation spring (BFSP1) having no stiffness in any direction was used as the base support data. After the foundations have been modelled and connected to the bent column bases, necessary support for the bent columns were achieved. The foundation spring data form is shown in Figure 3.9. The substructure location is critical because SAP2000/Bridge accounts for the superstructure/substructure kinematics. The ends of the bridge deck will have a tendency to rotate due to gravity loading. If the abutment bearings are restrained against translation at both ends of a bridge, outward reactions on the bearings and deck moments can be induced as a result of these restraints. The amount of outward thrust and the moment in the deck are a function of the amount of rotation and distance from the deck neutral axis to the top of abutment bearings.

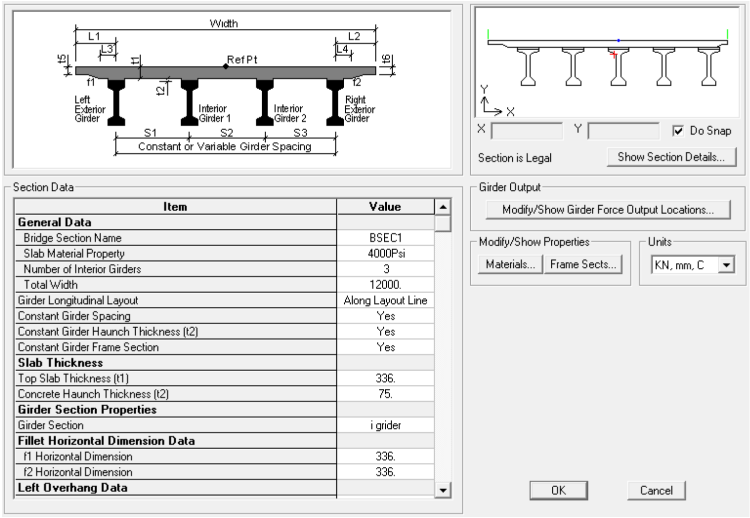


Figure 4.8 Bridge Deck Section Properties Form

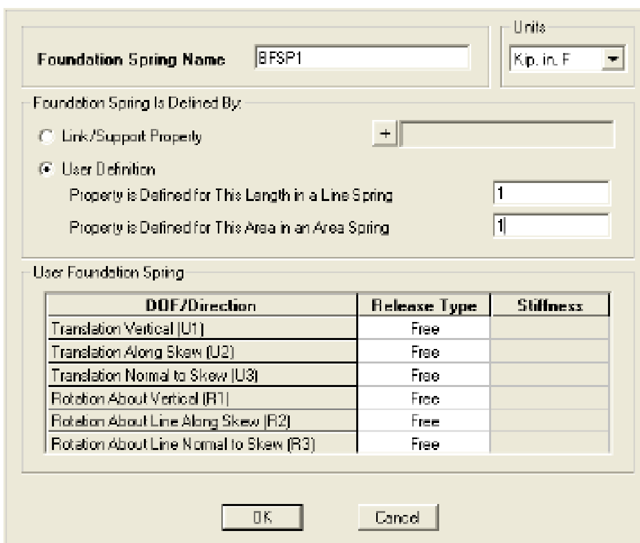


Figure3.9 Bent Column Base Restrained Definition Form

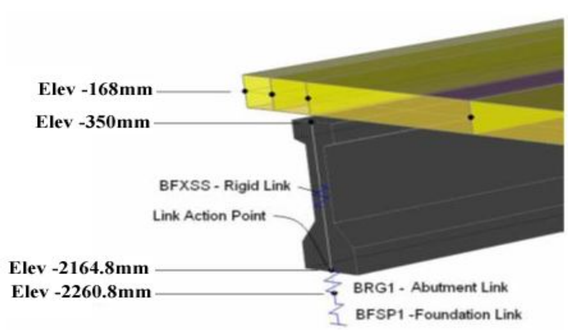


Figure 3.10. Abutment Bearing Geometry

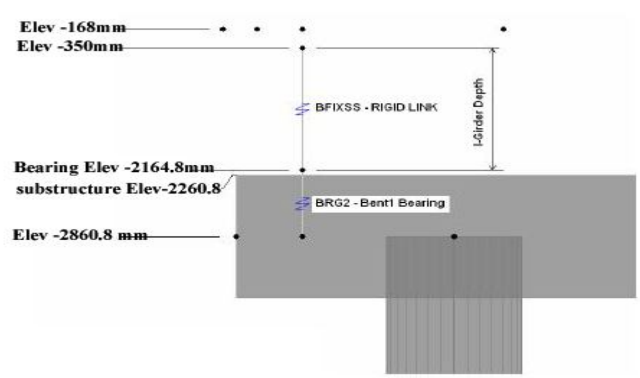


Figure3.11 Bent Support Geometry

International Journal for Research in Applied Science & Engineering Technology (IJRASET)
ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor:6.887
Volume 5 Issue VII, July 2017- Available at www.ijraset.com

IV. FOUNDATION MODELLING

Although it is not required to include explicit foundation element (foundation can be modelled as fixed, pinned, partially fixed restraints at the base of the column), these were included as part of the bridge models. Soil and foundation may have significant impact on the seismic response of bridge structures, particularly those with stiff foundation
and relatively soft deep soil. Refined inelastic simulations of the foundation and the underlying sub-strata were undertaken. The objective is to realistically estimate the soil properties required for SAP2000 soil and foundation modelling. The friction angle of the materials ranges from 28^o to 32^o. The in-situ properties of the cohesionless material at deeper layer are expected to have higher stiffness due to the large confinement. Since the shear modulus of each stratum is given as a constant value regardless of the depth of the stratum, it was assumed that the shear modulus was calculated at the mid-depth of each stratum. Vertical DOFs of side boundary are released to allow settlement due to gravity loads. All DOFs of the bottom nodes of the soil medium were restrained. To reduce the controlling nodes within the pile cap and to prevent local deformations, it was assumed that the each pile cap behaves as rigid body. All foundation profiles were thus controlled using a single node connected to eight boundary nodes, as shown in Figure 4.1. Symmetry was utilized to reduce the finite element mesh and computational demands for certain types of foundation profiles.

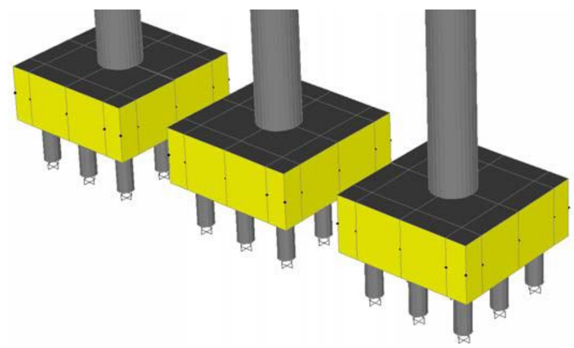


Figure 4.1 Single Node Foundation System For Inelastic Pushover Analysis

A. Equivalent Pile Formulation

Foundations can be modelled in many ways. Equivalent length piles were used with an equivalent length of 5.1 feet to model the pile surrounded by soil. The equivalent lengths were established using the equations given below for the foundation model shown in Figure 4.13.

1) Point of fixity $(EI/f)^{1/5}$
2) $K_\delta = EI/T^3$
3) $K_\theta = EI/T$
4) $K_{\delta\theta} = EI/T^2$

Where,

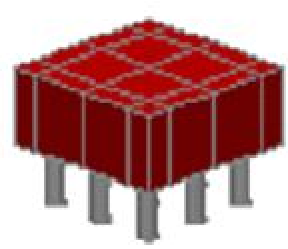


Figure 4.2 Equivalent Length of Pile

f = yield of foundation calculated from an average SPT blow count N.

T = 5.1 feet; this effective length is used in modelling the bridge foundation.
K_δ= Flexural stiffness of the pile
K_θ=Rotational stiffness of the pile
$K_{\delta\theta}$=Torsional stiffness of the pile and
EI= Flexural rigidity of the pile

The piles were defined as 1200mm diameter concrete piles with 24 nos. 16mm dia vertical bars. The outer steel casings of the pile were found to increase in the flexural stiffness of the piles by a factor of 2.353. This value was applied as a property modifier to the pile section property. The pile was added to the bridge model as "Equivalent Cantilever" piles. Using this method, the pile was replaced by a beam that has equivalent stiffness properties to that of the pile with the surrounding soil. After the lengths of the piles were known, the piles were connected to an area object representing the pile cap. The cap was meshed at the top of the pile locations. The completed pile cap is shown in Figure 4.14, which is a 3D extruded view.

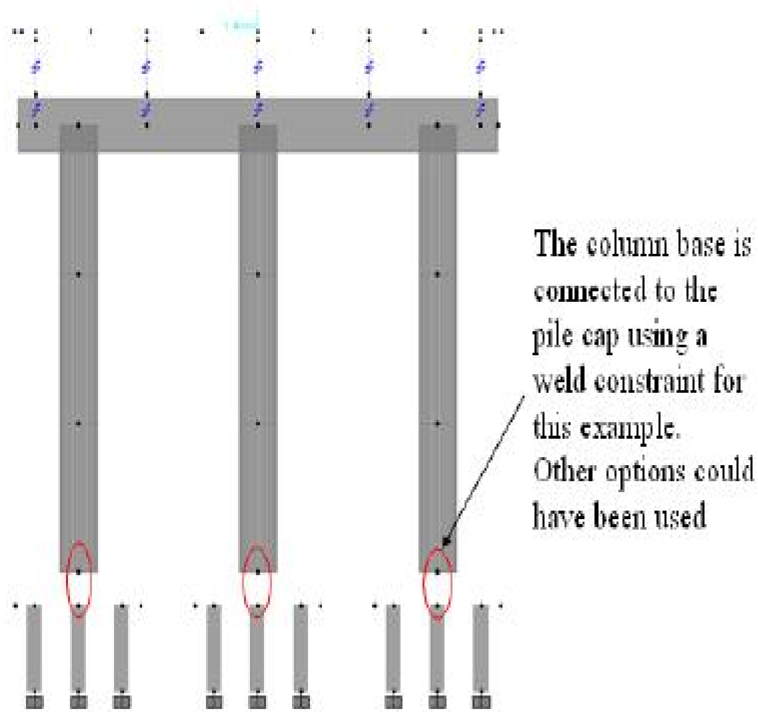


Figure 4.4 Bent Column Base Connectivity

B. Super Imposed Load Calculation

Self-weight of wearing coat = 1 X width of clear road way X thickness of wearing coat X unit weight
= 1X7.80X((0.075+.05)/2) X 25
= 12.1875kN/m

self-weight of footpath = 1X size of foot path X unit weight
= 1X2.20X((0.45+0.225)/2)X24
= 17.76 kN/m

self-weight of hand rail = No. of handrails per metre length X Volume of one handrail X unit weight
= 3X0.1008 x24
= 7.26 kN/m

self-weight of crash barrier = 1 X size of crash barrier X unit weight
= 1X(0.6+0.4)/2X0.50 x 24
= 6.00 kN/m

TOTAL LOAD = 43.21KN/M

V. PERFORMANCE CRITERIA AND ANALYSIS PROCEDURE

A. Inelastic Time History Analysis

Elcentro earthquake location time history data shown in Figure5.1 was assigned for performing inelastic time history analysis of bridge pier system. The responses from the inelastic time history analysis were monitored at the nodes of the bent cap and column. The following response plot functions were obtained from the inelastic time history analysis for the Models I to IV .

1) Force Vs time
2) Velocity Vs time
3) Acceleration Vs time
4) *Force Vs deformation*

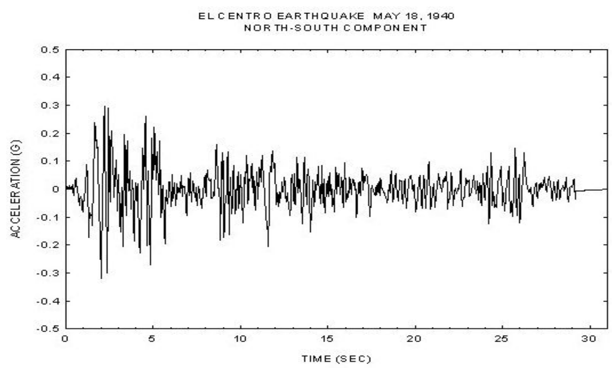


Figure 5.1 Elcentro Earthquake Time History Data

B. Response Spectrum and Seismic Design Request

The ground motion hazard (response spectrum) can be determined by SAP2000/Bridge by defining the bridge location using the latitude and longitude or the postal zone. As an alternative, any user defined response spectrum file can be used as input. The site effects (soil site amplification) also are considered and are part of the user input data. The recently adopted AASHTO Guide Specification for the LRFD Seismic Bridge Design incorporates hazard maps based on a 1000-year return period. When the the bridge location is defined by latitude and longitude, SAP2000/Bridge creates the appropriate response spectra curve. From the Response Spectrum Data form, the values for SDS and SD1 were determined by SAP2000 and reported. The SD1 value was used to determine the Seismic Design Category (SDC). The SDC was used to determine the analysis and design requirements to be applied to the bridge. For example, if the SDC is A, no capacity displacement calculation is performed. If the SDC is B or C, SAP2000 uses an implicit formula as per Section 4.8 of the AASHTO Seismic Guide Specification. If the SDC is D, SAP2000 uses a nonlinear pushover analysis to determine the capacity displacements. A flow chart that describes when an implicit or pushover analysis is used to determine capacity displacements and shown in Figure 5.

C. Dead Load Analysis And Cracked Section Properties

The results of the dead load analysis were then used to verify the analytical model followed by the determination of the cracked section properties that were then applied to the bent columns as frame section property modifiers. The reduced stiffness of the bent columns affected the response spectrum and pushover analysis. The frame section property modifiers were defined separately for each of the bent and abutment columns as a named property set. The Section Designer program was used to observe the moment-curvatures and $I_{cracked}$ properties for the various cross-sections. Auto load patterns and auto load cases are produced by the program. The load case, which has the default name, *_GRAV_SDReq1*, was automatically developed by SAP2000/Bridge as a single stage construction load case and was used to apply the cracked section property modifiers to the columns.

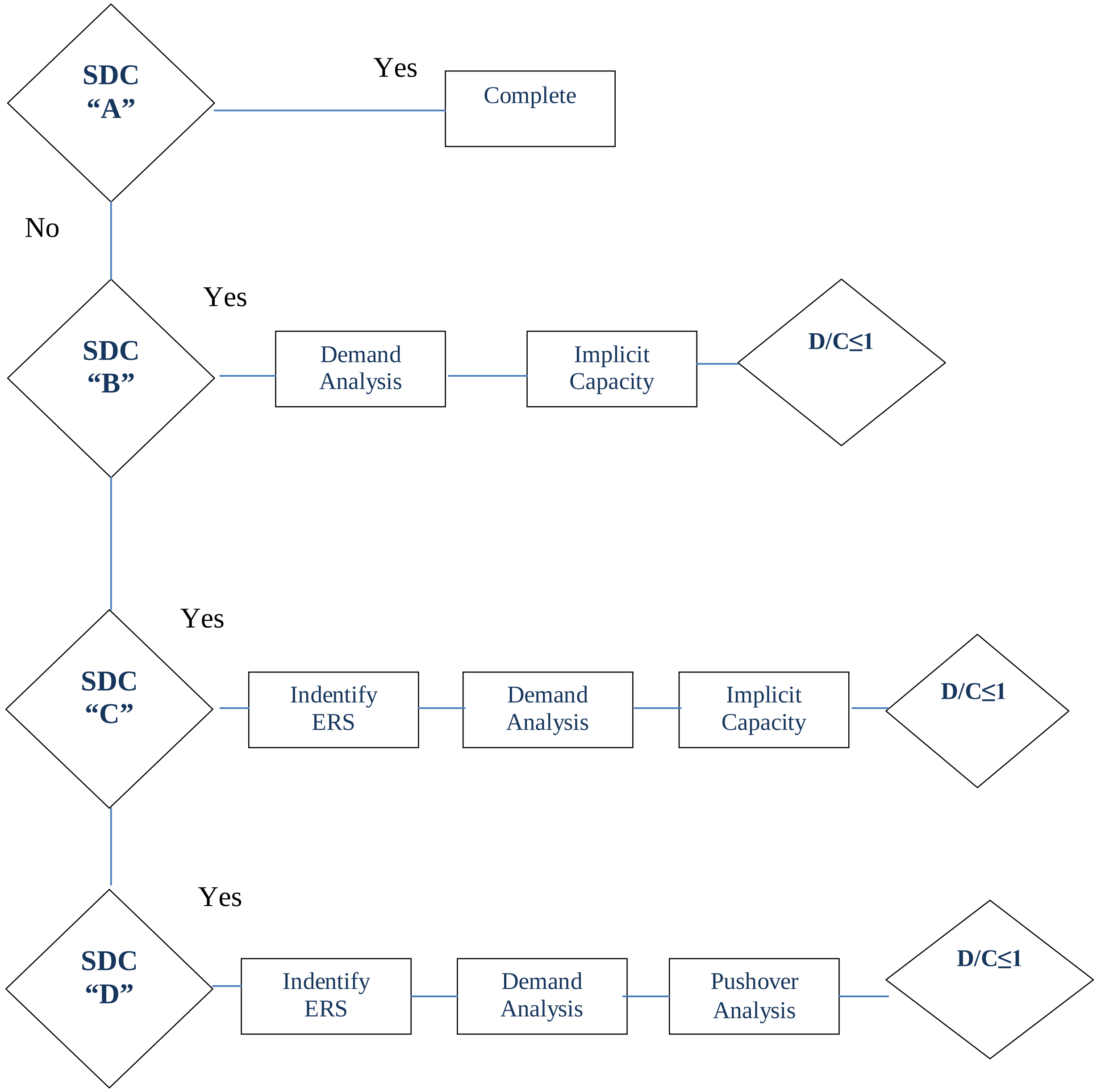


Figure 5.2 Flow Chart For Seismic Design Request

D. Response Spectrum And Demand Displacement

The seismic response of the entire bridge structure was analysed by SAP2000/ Bridge using the response spectrum function defined in section 5.3.2. The number of modes used by SAP2000/Bridge was automated and depends on the number of bridge spans. The total mass participation was ensured that an adequate number of modes were included in the modal analysis. The response spectrum displacements were used by SAP2000/Bridge as the displacement demands.Three response spectrum load cases were automatically produced by SAP2000/Bridge: _RS_X_SDReq1, _RS_Y_SDReq1 and _RS_XY_SDReq1. The first two response spectrum load

International Journal for Research in Applied Science & Engineering Technology (IJRASET)
ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor:6.887
Volume 5 Issue VII, July 2017- Available at www.ijraset.com

cases applied the dynamic loads along the U1 and U2 directions. The U1 direction is defined as the longitudinal loading direction that is chosen to be from the start abutment to the end abutment, both points located on the reference line of the bridge object. The third response spectrum load case uses a Directional Combination option of “ABS,” with an ABS scale factor of 0.3. This response spectrum load case satisfied the AASHTO Seismic Guide Specification, which requires the response spectrum loads to be combined using the 100/30 percent rule in each of the major directions. The single response spectrum load case, _RS_XY_SDReq1, envelopes the maximum response spectrum results for each of the combinations 100/30 and 30/100. The modal damping coefficient was set to 5 percent, but this value can be modified as necessary. To illustrate the ABS directional combination feature, the following BENT1 displacements are summarized for Model-III as shown in Figure 5.4.

E. Plastic Hinge Properties And Assignments

For bridge structures having a Seismic Design Category (SDC) D the AASHTO Seismic Guide Specification requires that the displacement capacity is determined using a nonlinear pushover analysis. This requires that the column plastic hinge lengths and plastic hinge properties are to be determined for each column that participates as part of the Earthquake Resisting System (ERS). In this chapter, the methodologies used to calculate the plastic hinge lengths and properties are explained. After the hinge properties have been determined, the plastic hinges are assigned to the ERS columns. The automation of the plastic hinge assignments is also explained in this chapter.

The plastic hinge lengths used in the Seismic Design Request was determined as perthe AASHTO Seismic Guide Specification as follows:

Plastic Hinge Length, $L_P = 0.08L + 0.15\, f_{ye}\, d_{bl}$,

Where,

f_{ye} = the effective yield strength of the longitudinal reinforcing, and

d_{bl} = the diameter of the longitudinal reinforcing.

The hinge length is compared to the value for the maximum hinge length valuedescribed as, LP + 0.3 $f_{ye}\, d_{bl}$, and the controlling value is used. After the hinge lengths and properties have been determined, the hinges were placed on the bent columns at each end of the column at distances from each end equal to 1/2 the hinge length, as shown below in Figure 5.5.

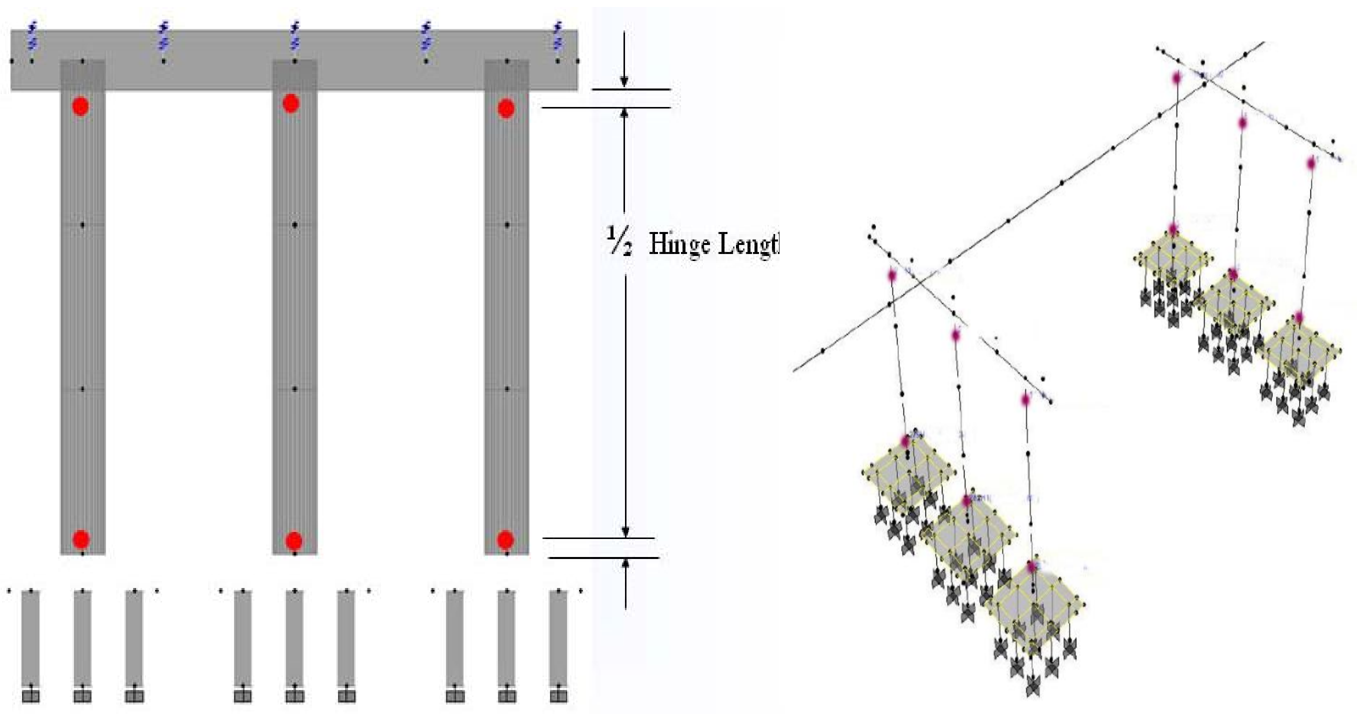


Figure 5.5 Hinge Locations of the Bents

F. Capacity Displacement And Pushover Analysis

This chapter describes the automated procedure that SAP2000/Bridge uses to determine the bridge seismic capacity displacements. The method used varies depending on the Seismic Design Category (SDC) of a particular bridge. The differences between the implicit and pushover approaches are described in the following chapters. For structures having reinforced concrete columns, the displacement capacities for SDC B and C were found using the following equations. The AASHTO Seismic Guide Specification equations are also noted.

For SDC B:

$$\Delta^{L}_{C} = 0.12H_0\ (-1.27\ l_n(x) - 0.32 \geq 0.12H_0$$

For SDC C:

$$\Delta^{L}_{C} = 0.12H_0\ (-2.32\ l_n(x) - 1.22 \geq 0.12H_0$$

In which

$$x = \lambda B_0/H_0$$

where,

H_0 = Clear height of the column (ft)

B_0 = Column diameter or width parallel to the direction of displacement under consideration (ft)

λ= Factor for the column end restraint conditions

G. Computation of D/C ratio

After the demand displacement and displacement capacity analysis have been completed, SAP2000/Bridge computed the ratio of the Demand/Capacity displacements and report these values in the Seismic Design Report. In Tables 7.14, 7.16, 7.18 and 7.20 shown, the ratio of the Demand/Capacity for all bridge models were reported, namely, the transverse and longitudinal direction for each bent. The reported Generalized Displacements displacements were used to average the top of bent displacements and to determine the relative displacements between the bent cap beam and the foundation.

VI. SEISMIC RETROFIT OF BRIDGE COLUMNS USING STEEL JACKETS

The retrofitting of bridges has received considerable attention in recent years because they need to be operational state in post-earthquake scenario for relief and rescue operations. There are two types of situations that require retrofitting in bridges, (i) the existing bridges that are deficient to meet requirements of current codes but are vulnerable to damage; these bridges have not yet experienced even moderate earthquakes, (ii) the existing bridges that are damaged in earthquakes. Providing a jacket around an existing column which has insufficient ductility and strength capacity is effective to prevent premature failure. The jacket is fabricated so that its radius is 12.5 to 25 mm larger than the column radius. After positioned over the areas to be retrofitted and are site-welded up the vertical seams to provide a continuous tube with a small annular gap around the column, the gap is grouted with epoxy resin or a pure cement grout as shown in Figure 6.1 .

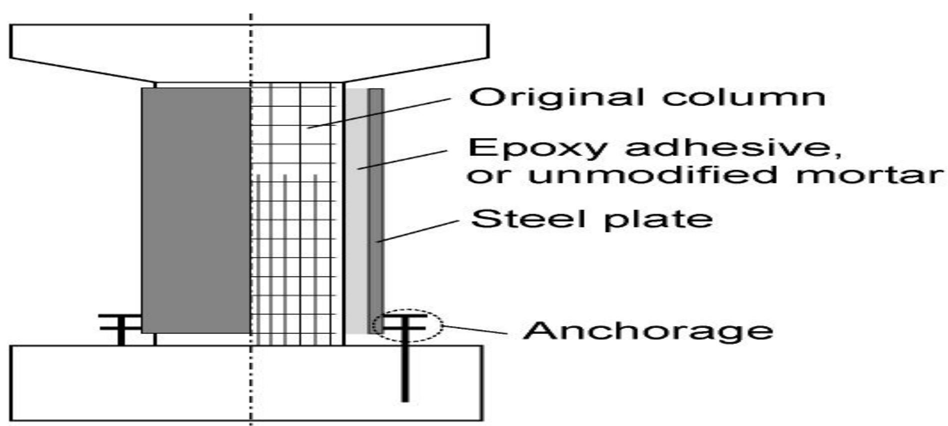


Figure 6.1 Steel Jacketing of Column

International Journal for Research in Applied Science & Engineering Technology (IJRASET)
ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor:6.887
Volume 5 Issue VII, July 2017- Available at www.ijraset.com

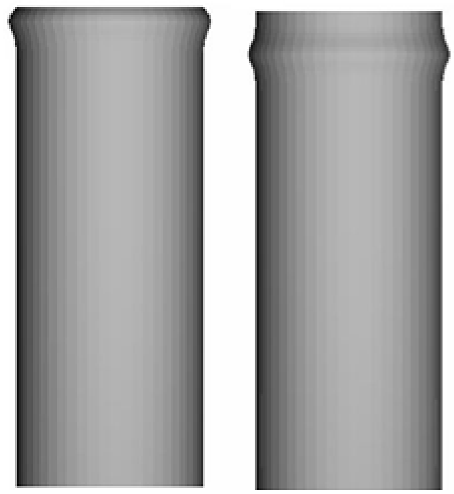


Figure 6.2 3D Extruded View of the Steel Jacketing of Column

A. Steel Jacketing Column Material And Section Property Definitions

In this bridge, circular columns were retrofitted with cylindrical steel jackets fabricated from 12mm thick f_y250 hot rolled steel. The steel jacketing bent columns were defined using the Section Designer option in the SAP2000 that can be accessed using the Define menu > Section Property > Frame Sections command. The size and quantity of both the vertical and confinement reinforcing steel were defined using the form shown in Figure 6.3.Effective confinement was provided to existing substandard circular columns by encasing the potential plastic hinge regions with a site-welded cylindrical steel sleeve or jacket. The increase in ultimate compressive strain as a result of confinement from the steel jacket was estimated 0.0541 from the equation suggested by Priestley and Seible(1991) and defined in the Concrete Strain Edit Box of the moment curvature form shown in Figure 6.4:

$\varepsilon_{cu} = 0.004 + 1.4\varepsilon_{su}\, \rho_{sj}\, (f_{yi}/f'_{cc})$

Where, ε_{cu} = ultimate compressive strain of concrete,

f_{yi}= yield strength of the steel jacket

f'_{cc}=compressive strength of confined concrete

ε_{su} =ultimate tensile strain of the steel jacket

ρ_{sj}= confining ratio of the steel jacket and is given by $4t_j/(D_j-2t_j)$

where, t_j and D_j are the out side diameter and thickness of the steel jacjet respectively.

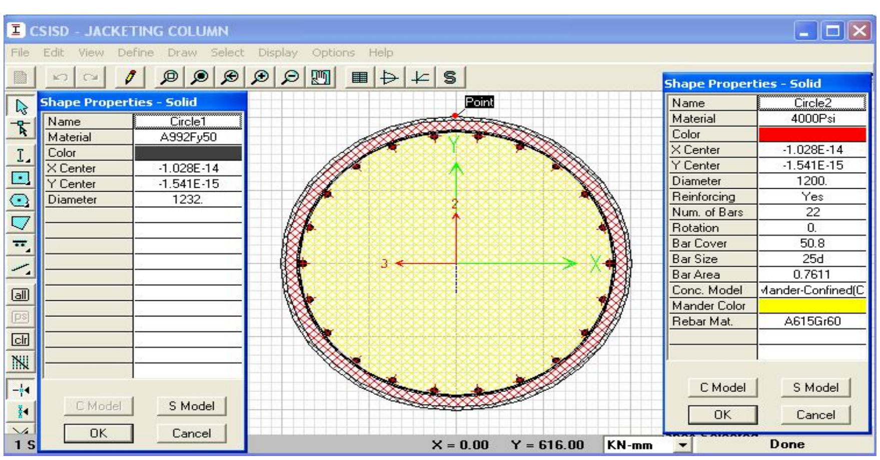


Figure 6.3 Steel Jacketing Column Definition Form

B. Demand Capacity Ratio Of Retrofit Bridge

Detailed three dimensional dynamic response simulations of the steel jacketing column bridge was conducted as in chapters 4 and 5. SAP2000/Bridge has computed the ratio of the Demand/Capacity displacements and shown in Tables 7.15, 7.17, 7.19 and 7.21 .

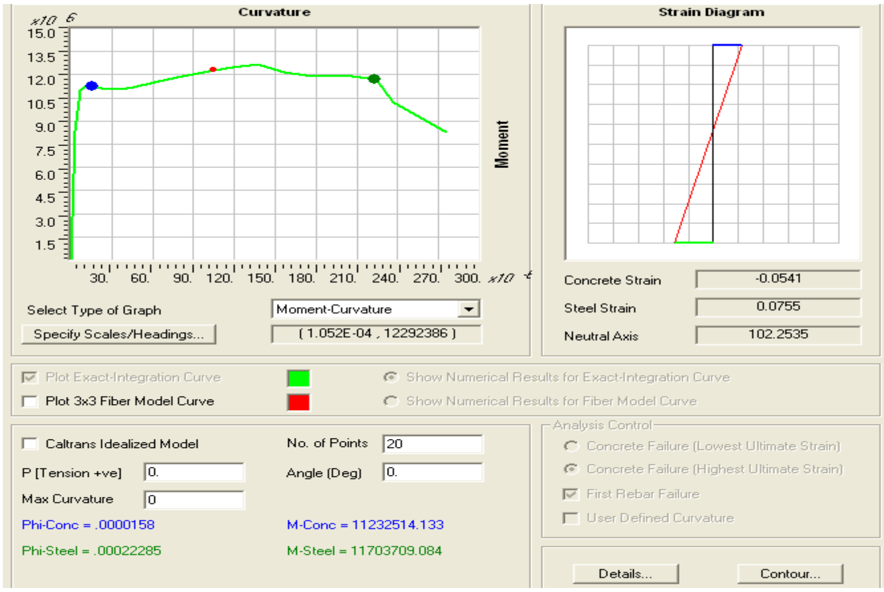


Figure 6.4 Moment Curvature and Strain Diagram Form of Steel Jacketing Section

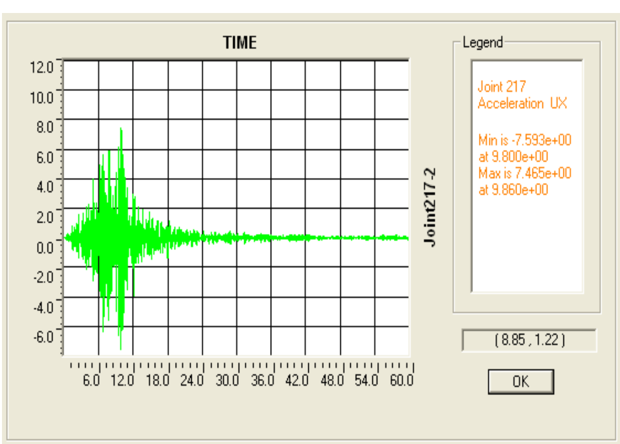
VII.RESULTS AND DISCUSSION

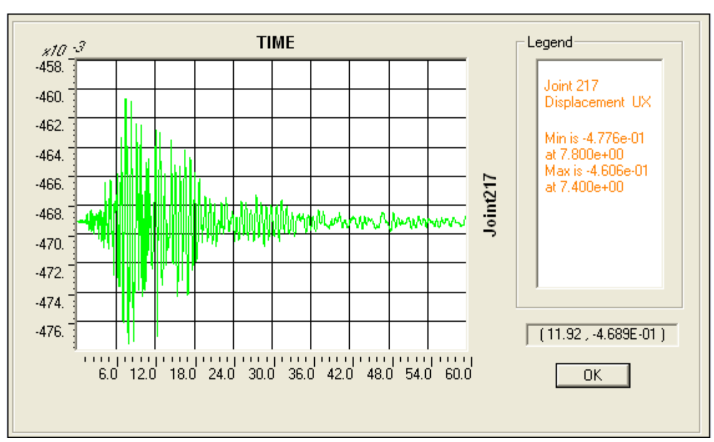
A. Inelastic Time History Analysis Response Plots

The response from the inelastic time history analysis was monitored at the level of bottom of the bearings of the bent. The peak displacement, velocity and acceleration of the Bent-I of the each model is give in Table 7.1. The response plot functions of the inelastic time history analysis for all the Models are shown in Figure 7.1 to 7.8.

Table 7.1 Peak Displacement and acceleration values of Time History Response plot

Sl.No	Name of the Model	Bent No.	Joint No.	Peak Displacement in mm	Peak Acceleration in m/sec^2
1.	Model-I	Bent-I	217	477.60	6.69
2.	Model-II	Bent-I	215	458.90	6.17
3.	Model-III	Bent-I	215	460.10	6.78
4.	Model-IV	Bent-I	217	477.60	7.46





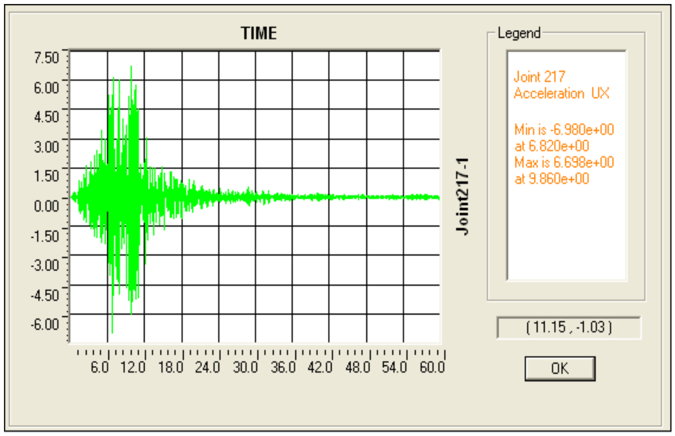


Figure 7.1 Time Vs Displacement and Time Vs Acceleration Response plots for Model-I

B. Response Spectrum And Pushover Analysis Results

1) Bent Column Demand Forces: A summary of the bent column seismic demand forces for the all models are tabulated as below.

Table 7.5 Bent column Demand Forces of the Model-IV

Sl. No.	Span Name	Station m	Location	P KN	V2 KN	V3 KN	T KNm	M2 KNm	M3 KNm
1	SPAN1	21.60	Top	4579.301	2151.21	2154.12	497.48	2680.64	3011.53
2	SPAN1	21.60	Bottom	4579.827	2162.77	2166.15	497.48	9757.97	4248.06
3	SPAN1	21.60	Top	4579.301	2151.21	2154.12	497.48	2680.64	3011.53
4	SPAN1	21.60	Bottom	4579.827	2162.77	2166.15	497.48	9757.97	4248.06
5	SPAN2	43.20	Top	2697.923	1205.24	2053.8	215.52	2552.27	1690.52
6	SPAN2	43.20	Bottom	2698.432	1216.44	2065.53	215.52	9403.33	2386.18
7	SPAN2	43.20	Top	2697.924	1205.24	2053.8	215.52	2552.27	1690.52
8	SPAN2	43.20	Bottom	2698.433	1216.44	2065.53	215.52	9403.33	2386.18
9	SPAN3	64.80	Top	2697.923	1205.24	2053.8	215.52	2552.27	1690.52
10	SPAN3	64.80	Bottom	2698.432	1216.44	2065.53	215.52	9403.33	2386.18
11	SPAN3	64.80	Top	2697.926	1205.24	2053.8	215.52	2552.27	1690.52
12	SPAN3	64.80	Bottom	2698.435	1216.44	2065.53	215.52	9403.33	2386.18
13	SPAN4	86.40	Top	4579.301	2151.21	2154.12	497.48	2680.64	3011.53
14	SPAN4	86.40	Bottom	4579.827	2162.77	2166.15	497.48	9757.97	4248.06

15	SPAN4	86.40	Top	4579.301	2151.21	2154.12	497.48	2680.64	3011.53
16	SPAN4	86.40	Bottom	4579.827	2162.77	2166.15	497.48	9757.97	4248.06

2) *Support Bearing Demand Forces:* The forces in the bearing due to the seismic loads are presented in the Tables 7.5 to 7.8 below. All bearings at the abutments and bents that are found to resist seismic forces are included in the subject table.

Table 7.9 Support bearing Demand Forces of the Model-IV

Sl. No.	Bearing Location	Station m	P KN	V2 KN	V3 KN	T KNm	M2 KNm	M3 KNm
1	Start Abutment	0	Abutment	98.624	0	0	0	0
2	Start Abutment	0	Abutment	52.115	0	0	0	0
3	Start Abutment	0	Abutment	52.115	0	0	0	0
4	Start Abutment	0	Abutment	98.6252	0	0	0	0
5	Bent @ SPAN1	21.60	Bent	800.618	736.709	687.551	0	0
6	Bent @ SPAN1	21.60	Bent	867.204	1320.24	1313.93	0	0
7	Bent @ SPAN1	21.60	Bent	867.204	1320.24	1313.93	0	0
8	Bent @ SPAN1	21.60	Bent	800.618	736.709	687.551	0	0
9	Bent @ SPAN2	43.20	Bent	523.872	392.988	617.885	0	0
10	Bent @ SPAN2	43.20	Bent	534.394	722.026	1283.91	0	0
11	Bent @ SPAN2	43.20	Bent	534.394	722.026	1283.91	0	0
12	Bent @ SPAN2	43.20	Bent	523.872	392.988	617.885	0	0
13	Bent @ SPAN3	64.80	Bent	523.871	392.988	617.885	0	0
14	Bent @ SPAN3	64.80	Bent	534.394	722.026	1283.91	0	0
15	Bent @ SPAN3	64.80	Bent	534.391	722.026	1283.91	0	0
16	Bent @ SPAN3	64.80	Bent	523.873	392.987	617.885	0	0
17	Bent @ SPAN4	86.40	Bent	800.618	736.709	687.551	0	0
18	Bent @ SPAN4	86.40	Bent	867.204	1320.24	1313.93	0	0
19	Bent @ SPAN4	86.40	Bent	867.204	1320.24	1313.93	0	0
20	Bent @ SPAN4	86.40	Bent	800.618	736.709	687.551	0	0
21	End Abutment	108.00	Abutment	98.624	0	0	0	0
22	End Abutment	108.00	Abutment	52.115	0	0	0	0
23	End Abutment	108.00	Abutment	52.115	0	0	0	0
24	End Abutment	108.00	Abutment	98.625	0	0	0	0

ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor:6.887
Volume 5 Issue VII, July 2017- Available at www.ijraset.com

3) Support Bearing Demand Displacement Results : Upon completion of the response spectra analysis, the displacements are tabulated for each bent of the all models. The displacements were calculated using "Generalized Displacements" to account for the average cap beam displacements and the relative displacement between the cap beam and foundation. The displacements for the ABS response spectrum load case also were tabulated for each of the bearing active degrees of freedom. The displacements for all bearings at the abutments and bents that resist seismic loads are tabulated in Tables 7.9 to 7.12.

Table 7.13 Support bearing Demand Displacements of the Model-IV

Sl. No.	Bearing Location	Station m	U2 mm	U3 mm	R1 Degrees	R2 Degrees	R3 Degrees
1	Start Abutment	0	51.37	86.81	1.35E-03	1.92E-04	8.28E-05
2	Start Abutment	0	51.34	85.86	1.35E-03	1.17E-04	4.14E-05
3	Start Abutment	0	51.34	85.86	0.001348	1.17E-04	4.14E-05
4	Start Abutment	0	51.37	86.81	1.35E-03	1.92E-04	8.28E-05
5	Bent @ SPAN1	21.60	0	0	2.59E-04	2.60E-02	2.78E-03
6	Bent @ SPAN1	21.60	0	0	2.46E-04	2.58E-02	4.30E-03
7	Bent @ SPAN1	21.60	0	0	2.46E-04	2.58E-02	4.30E-03
8	Bent @ SPAN1	21.60	0	0	2.59E-04	2.60E-02	2.78E-03
9	Bent @ SPAN2	43.20	0	0	1.91E-04	2.60E-02	1.48E-03
10	Bent @ SPAN2	43.20	0	0	3.58E-05	2.59E-02	2.36E-03
11	Bent @ SPAN2	43.20	0	0	3.58E-05	2.59E-02	2.36E-03
12	Bent @ SPAN2	43.20	0	0	1.91E-04	2.60E-02	1.48E-03
13	Bent @ SPAN3	64.80	0	0	1.91E-04	2.60E-02	1.48E-03
14	Bent @ SPAN3	64.80	0	0	3.58E-05	2.59E-02	2.36E-03
15	Bent @ SPAN3	64.80	0	0	3.58E-05	2.59E-02	2.36E-03
16	Bent @ SPAN3	64.80	0	0	1.91E-04	2.60E-02	1.48E-03
17	Bent @ SPAN4	86.40	0	0	2.59E-04	2.60E-02	2.78E-03
18	Bent @ SPAN4	86.40	0	0	2.46E-04	2.58E-02	4.30E-03
19	Bent @ SPAN4	86.40	0	0	2.46E-04	2.58E-02	4.30E-03
20	Bent @ SPAN4	86.40	0	0	2.59E-04	2.60E-02	2.78E-03
21	End Abutment	108.00	51.37	86.81	1.35E-03	1.92E-04	8.28E-05
22	End Abutment	108.00	51.34	85.86	1.35E-03	1.17E-04	4.14E-05
23	End Abutment	108.00	51.34	85.86	1.35E-03	1.17E-04	4.14E-05
24	End Abutment	108.00	51.37	86.81	1.35E-03	1.92E-04	8.28E-05

4) Demand capacity ratio: After the demand displacement and displacement capacity analysis have been completed, the ratio of the Demand/Capacity displacements have been computed and tabulated in the Tables 7.13 to 7.20 for all bridge and retrofitted models.

International Journal for Research in Applied Science & Engineering Technology (IJRASET)
ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor:6.887
Volume 5 Issue VII, July 2017- Available at www.ijraset.com

Table 7.20 Demand – Capacity Ratio of the Model-IV

Sl No	Span Name	Station mm	Direction	General Displacement	Demand mm	Capacity mm	DCRatio
1	SPAN1	21600	TRANS	GD_TR1_DReq1	14.38	33.64	0.427
2	SPAN1	21600	LONG	GD_LG1_DReq1	66.36	33.64	1.973
3	SPAN2	43200	TRANS	GD_TR2_DReq1	8.292	33.64	0.247
4	SPAN2	43200	LONG	GD_LG2_DReq1	66.38	33.64	1.973
5	SPAN3	64800	TRANS	GD_TR3_DReq1	8.292	33.64	0.247
6	SPAN3	64800	LONG	GD_LG3_DReq1	66.38	33.64	1.973
7	SPAN4	86400	TRANS	GD_TR4_DReq1	14.38	33.64	0.427
8	SPAN4	86400	LONG	GD_LG4_DReq1	66.36	33.64	1.973

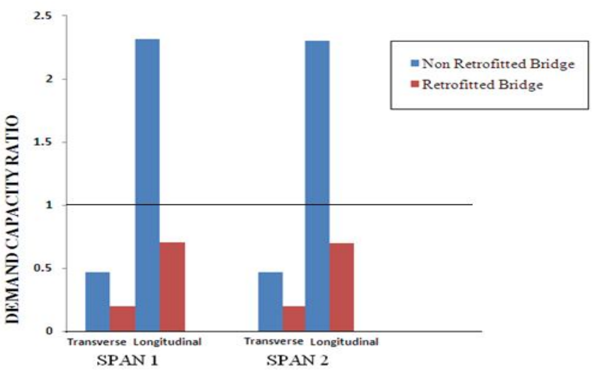
Table 7.21 Demand – Capacity Ratio of the Retrofitted Model-IV

Sl No	Span Name	Station mm	Direction	General Displacement	Demand mm	Capacity mm	DCRatio
1	SPAN1	21600	TRANS	GD_TR1_DReq1	5.879	33.64	0.175
2	SPAN1	21600	LONG	GD_LG1_DReq1	20.07	33.64	0.597
3	SPAN2	43200	TRANS	GD_TR2_DReq1	3.746	33.64	0.111
4	SPAN2	43200	LONG	GD_LG2_DReq1	20.08	33.64	0.597
5	SPAN3	64800	TRANS	GD_TR3_DReq1	3.746	33.64	0.111
6	SPAN3	64800	LONG	GD_LG3_DReq1	20.08	33.64	0.597
7	SPAN4	86400	TRANS	GD_TR4_DReq1	5.879	33.64	0.175
8	SPAN4	86400	LONG	GD_LG4_DReq1	20.07	33.64	0.597

C. Comparison of Seismic Demand Versus Capacity

The seismic design category C and D were performed for inelastic static pushover analysis in both the longitudinal and transverse directions of bridge to evaluate the lateral capacity of the bridge bents. Extensive inelastic time history analysis was also executed using Elcentro time history data to examine the response of the bridge under various seismic scenarios with increasing severity. Capacities of the foundations, bents, bearings and expansion joints are compared with seismic demands at various hazard levels. Sample results from these comprehensive analyses are discussed below to highlight the significance of simulation approaches in identifying areas of vulnerability of complex bridges.

1) Due to the length of the bridge and the non-uniform distribution of stiffness and mass, higher modes of vibrations notably contribute to the seismic response.
2) Inelastic response spectrum and time history analysis carried out in the transverse directions of the bridge indicate that the drift demands are acceptable .
3) Response spectrum corresponding to seismic Zone IV from IS1893 significantly amplifies the bent deformation demands compared with other records.
4) Higher drift demands are observed in the longitudinal direction of the bridge compared with the transverse direction due to its lower stiffness as shown in Figures 7.9 to 12.
5) High deformations are observed in the interior bent frames, which are also observed from time history analysis.
6) The high relative displacement demands in the longitudinal direction cause pounding at the two abutments and the expansion joint of bents.
7) The response of the foundation system are acceptable in both directions, while high demands are observed at the top columns of bents.
8) The shear, flexural strength and ductility capacity of the columns are enhanced by introducing steel jackets which reduces the demands of the bridge..
9) The ultimate compressive strain of concrete increases from 0.005 to 0.05 by adopting proper confinement to the columns.



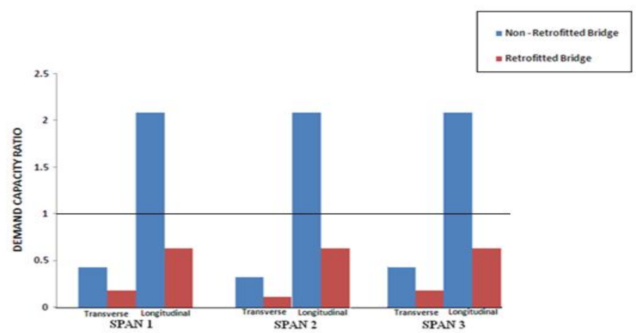


Figure 7.10 Bents Demand Versus Capacity of the Two and Three Spans Bridge

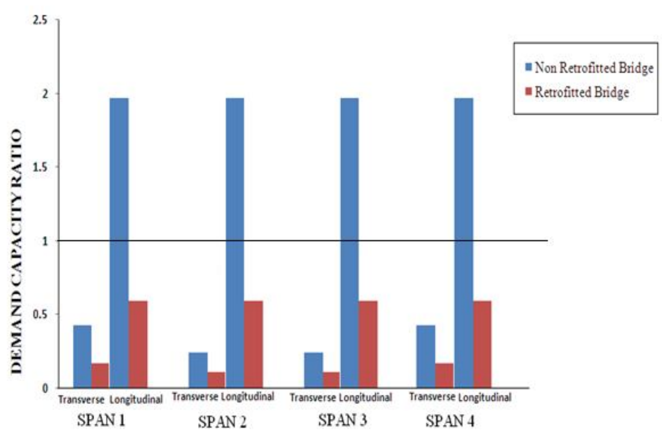


Figure 7.11 Bents Demand Versus Capacity of the Four and Five Spans Bridge

VIII. CONCLUSION

This paper highlights a work carried out to assess the seismic response of an existing 29-span bridge. The focus is on describing the methodology adopted to idealize the bridge and its foundation system. Sample results from the extensive elastic and inelastic analyses under the effect of IS1893 response spectrum input ground motions are presented. The bridge was built in the vicinity of a major source of earthquakes and includes typical deficiencies of bridges constructed without current seismic provisions. The study confirmed that simplifying modeling assumptions of different bridge components may have significant impact on the seismic response of deficient bridges. Higher modes of vibration notably contributed to the seismic response of the bridge due to the length of the bridge and the non-uniform distribution of stiffness and mass. The response of the bridge was unacceptable due to the observed yielding and damage in a number of bents and bearings. The demands corresponding to the zone III and IV ground motions almost exceeded the limit state capacity of bridge components. Indications of yielding in foundations were also observed. The suggested retrofit of piers by cylindrical steel jacketing is effective in enhancing the shear, flexural strength and ductility capacity of the piers and reducing the seismic demands of the bridge components. It is concluded that this assessment study confirmed the need to retrofit different bridge components to mitigate potential seismic risk and procedures used for this assessment are applicable to similar bridges constructed prior to IS1893-2002 current codal provisions.

REFERENCES

[1] Aman Mwafy, Oh-Sung Kwon and Amr Elnashai"Seismic assessment of an existing non-seismically designed major bridge-abutment-foundation system", Engineering Structures Vol. 32 April 2010, pp. 2192-2209.

[2] Giuseppe C. Marano and Rita Greco"A performance Reliability based Criterian for the optimum Design of Bridge isolators" ISET Journal of Earthquake Technology, Paper No. 451, Vol. 41, No. 2-4, June-December 2004, pp. 261-276.

[3] Kazuhiko Kawashima, Shigeki Unjoh " Seismic Response Control of Bridges by Variable Dampers" ASCE Journal of Structural Engineering" Paper No 9, Vol. 120, September 1994 pp. 2583 - 2600.

[4] Mahesh Tandon" Economical Design of Earthquake resistant Bridges" ISET Journal of Earthquake Technology, Paper No. 453, Vol. 42, No. 1, March 2005, pp. 13-20.

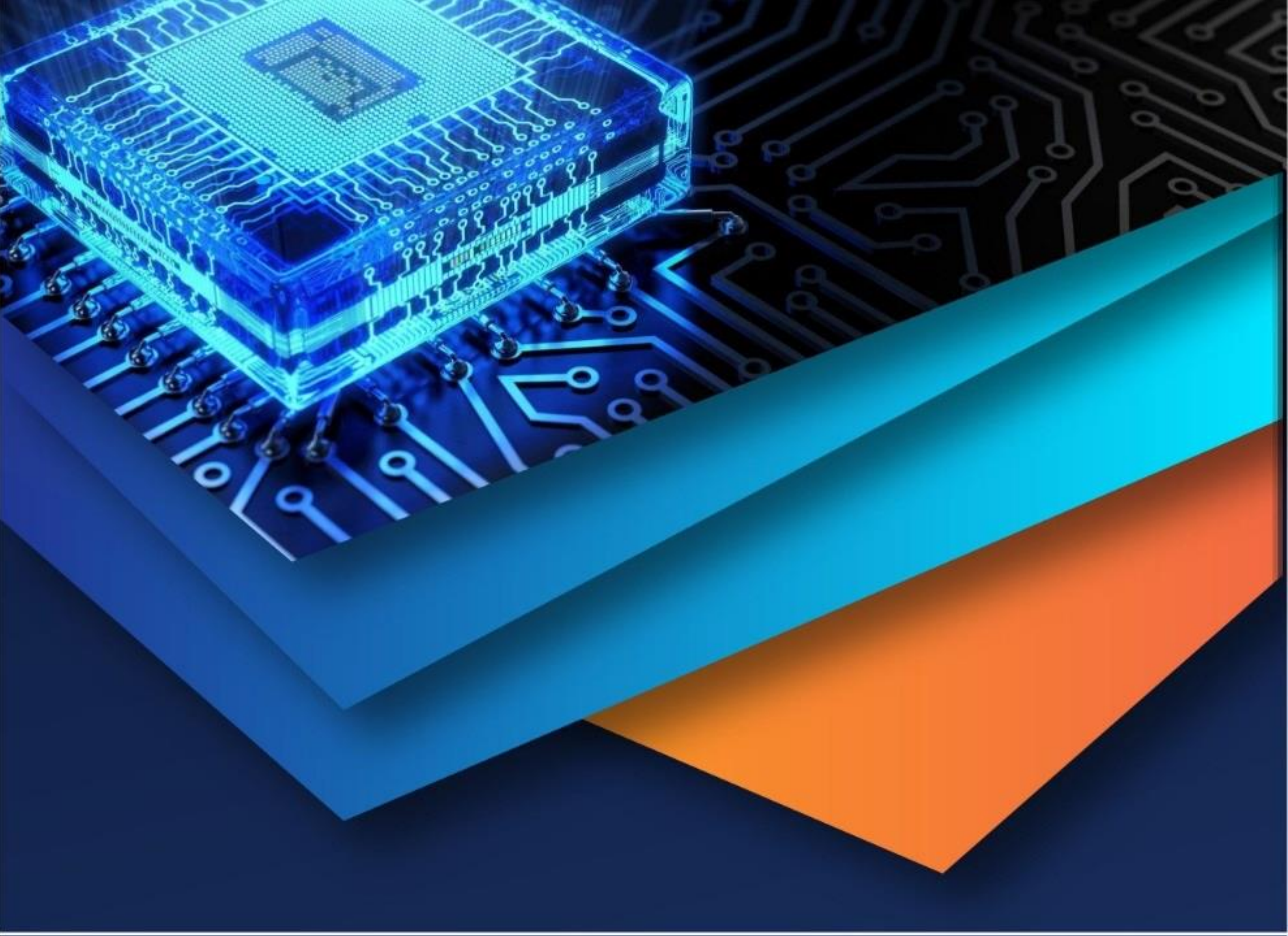
[5] Parulekar.Y.M, Reddy.G.R, Vaze.K.K, Ghosh.A.K, Kushwaha.H.S and Ramesh Babu.R" Seismic response analysis of RCC structure with yielding dampers using linearization techniques", Nuclear Engineering and Design, Paper No. 239 August 2009, Vol.23 pp. 3054–3061.

[6] Rashidi.S and Ala Saadeghvaziri.M "Seismic Modeling of Multi Span simply supported Bridge using ADINA" Computers and Structures, Vol 64, March 1997, pp.1025-1039.

[7] Rupen Goswami and C. V. R. Murty" Seismic Vulnerability of RC Bridge Piers Designed as per Current IRC Codes including Interim IRC:6-2002 Provisions" A Ph.D dissertation to Department of Civil Engineering, IIT, Kanpur, 2005.

[8] Sevasti. D, Tegou, Stergios. A. Mitoulis ,and Ioannis A. Tegos(2010)"An unconventional earthquake resistant abutment with transversely directed R/C walls" Engineering Structures, Vol. 32 September 2010, pp. 3801–3816.

[9] M.J.N. Priestley, F.Seible and Y.H.Chai (1992)" Seiemic Retrofit of Bridge Columns using Steel Jackets", University of California, San Diego, California, USA, Earthquake Engineering, Tenth world Conference 1992 Balkema, Rotterdam.



10.22214/IJRASET

INDEX COPERNICUS 45.98

IMPACT FACTOR: 7.129

IMPACT FACTOR: 7.429

INTERNATIONAL JOURNAL FOR RESEARCH

IN APPLIED SCIENCE & ENGINEERING TECHNOLOGY