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Comparative Study of confinement requirement of concrete Bridge pier using IRC, IS-1893: part-3(draft) and CALTRANs Guidline

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Abstract

Failures of the bridge piers in the past major earthquakes attracted attention towards prevalent seismic design practices for bridge piers. The present study compared design provision of RC bridge piers given in Indian and International codes of concrete member. Bridge pier confinement provision in Plastic hinge zone obtained using International code such as Caltrans (USA), was compared with those obtained using Indian codes. Seismic design scenario was considered, which reflects the differences in the RC design provisions among Indian codes, namely IS 1893 Part-3 (Draft), and IRC 21: 2000, and Caltrans (USA). From analysis of a number of pier, transverse reinforcement requirements are found to be inadequate for specific drift demand; shear capacities of section are found to be lower than the shear demand due to flexure. Increasing transverse reinforcement increase the deformability and ductility of the pier. Increase in the level of axial load reduces the ductility but increase the shear demand on the section.

Keywords-*Bridge pier behavior; Confinement reinforcement; Axial load; High strength concrete; Ductility*

I. PERFORMANCE OF BRIDGE IN PAST EARTHQUAKE

One of the most devastating earthquakes during the early part of the last century that severely affected bridges was the 1923 Kanto earthquake in Japan. Piers supported by masonry were crushed during the shaking. Based on damages to highway bridges since 1926, Seismic Coefficient Analysis Method was introduced in Japanese codes for the analysis of bridge system subjected to the lateral load caused by earthquake. In the 1995 Hyogo-Ken Nanbu (Kobe) earthquake severely affected the bridges, particularly the single column type reinforced concrete piers. The magnitude of 6.9 caused major damages about 60% of the bridges in the region. Most of concrete piers failed due to insufficient (i) transverse reinforcement for shear strength, (ii) confinement, and (iii) lateral support to longitudinal bars against buckling. Premature termination of longitudinal reinforcement caused a number of columns to develop flexure-shear failure at In USA, the 1971 San Fernando earthquake was significant from the point of view of performed better than the single type column ones. In Japan alone, since 1923 Kanto earthquake, about 3000 bridges have got significant damages (Duan, 2003).

Seismic design of bridges. Sub structure columns primarily failed in shear, both outside and within the plastic hinge region. The failure outside the plastic hinge was due to the flexural strength based shear demand exceeding the shear strength capacity and due to the lack of confinement from the inadequate transverse reinforcement. The failure inside the plastic region was due to the shear strength in the plastic hinge region being less than that in the portion outside the hinge. Piers also showed inadequate flexural ductility. Due to inadequate transverse reinforcement and, the crushing of concrete in the plastic region extended into the core of the section as soon as concrete strain reached ultimate unconfined concrete strain, and longitudinal steel buckled resulting in rapid strength degradation; eventually this lead to the inability of pier to sustain the gravity load too. This earthquake served as a major turning point in the development of seismic design criteria for United States, prior to which, specification for the seismic design of bridges were primarily based on existing lateral force requirements for buildings (Duan, 2003).

A major part of damage and collapse of bridges during the past earthquake is due to failure of piers. Bridge piers, even today, are sometimes designed primarily as axially loaded members (e.g., IRC 21: 2000). However professional documents identified that piers were vulnerable in shear in strong earthquakes. This is emphatically demonstrated by the numerous collapse of large number of reinforced concrete piers during the 1971 San Fernando earthquake. Two types of failure are mainly observed (i) flexural-shear failure, and (ii) axial compression failure. The first type is more common with slender columns with low axial loads resulting in flexural cracking, and the second type usually occurs in the stocky columns with high axial loads, which results in complete destruction of the concrete core prior from diagonal shear cracking.

Over the past two decades, India had experienced many moderate earthquakes that caused damage to highway and railway bridges. These earthquakes include the 1984 Cachar earthquake (M 6.4), the 1991 Uttarkashi earthquake (M 6.6), the 1993 Killari earthquake (M 6.4), the 1997 Jabalpur earthquake (M 6.0), the 1990 Chamoli earthquake (M 6.5) and the recent 2001 Bhuj Earthquake (M 7.7). Also during 1897-1950, India had experienced four great earthquakes (M>8), namely the 1897 Assam earthquake (M 8.7), the 1905 Kangra earthquake (M 8.6), the 1934 Bihar-Nepal earthquake (M 8.4), and the 1950 Assam-Tibet earthquake. Today, over 60% of the country lies in the higher three seismic zones III, IV and V. Earthquakes in the recent years in the country have resulted in spectacular collapse bridges (e.g. Gawana steel bridge in the Uttarkashi earthquake 1991. Thus, India has potential for strong seismic shaking, and the large number of existing bridges, and those being constructed as a part of the ongoing National Highway Development Project, as per existing design specifications can be vulnerable to future earthquakes (Murty and Jain,1997).

II. DEFINITION AND MODELING OF DEFORMATION MECHANISMD IN RC MEMBERS

For adequate seismic performance, strength and deformation capacities of a structure must be greater than the demands imposed by a design earthquake. Performance evaluation of a structure is done using several methods, for example, linear static methods specified in most of the design codes, or lately using more involved non-linear methods (i.e. Static pushover analysis, Time history analysis) Time History Analysis required more complex input quantities and highly time consuming and cumbersome if used for all structures for example, cyclic load-deformation behavior of structural element. Therefore, a simpler and effective option for most of the structure is to use approximate procedures of performance evaluation of structures, such as nonlinear static pushover analysis.

Static pushover analysis is a powerful tool to predict the lateral response of structures by considering non-linearity in material and geometry (P-Δ effects). This procedure is generally considered to be more realistic in evaluating seismic vulnerability of new or existing structures than the linear procedure. The procedure of the pushover analysis involves subjecting a structure to a monotonically increasing the prescribed lateral force or displacement which would be experience when structure subjected to ground motion. Under incrementally increasing load or displacement various structural elements would yield, consequently, at each increment, the structure experiences a lost in stiffness. In the present study, SAP2000 Advanced 14 (CSI 2009) is used for displacement-controlled pushover analysis of structure. Base shear at the base of structure plotted against corresponding displacement at the top of pier is known as Pushover Curve.

2.1. Material Modeling

In the implementation of the pushover analysis, modeling is one of the most important steps. It requires the determination of the non-linear properties of each component in structures, quantified by strength and deformation capacities, which depends upon the modeling assumptions. Stress- Strain model of confined concrete developed by Mander et. al. (1988) and stress-strain curve for the reinforcing steel developed by Park et al. (1982) as shown in Figure 1.

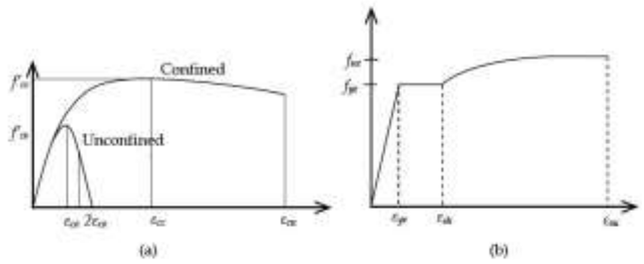

Figure 1. Stress-strain model for (a) Concrete (b) Reinforcing Steel used in the Pushover Analysis by SAP 2000 [CSI 2009]

The initial ascending curve is represented by same expression for both confined and un-confined concrete since the confining steel has no effect in this range. As the curve approaches the compressive strength of un-confined concrete, the unconfined stress begins to fall to an unconfined strain level before rapidly degrading to zero at the spalling strain ε_{sp} which is 0.005. The confining concrete model continues to ascend until the confined compressive strength f'_{cc} is reached. The ultimate compressive strain ε_{cu} is defined as the point where strain energy equilibrium is reached between concrete and the confining steel. The model is developed assuming the concrete columns under uniaxial compressive loading and confined by transverse reinforcement. The model also accounts for cyclic loading and the effect of strain rate.

The reinforcing steel is modeled with stress-strain relationship that exhibits an initial linear elastic portion, a yield plateau, and a strain hardening range in which the stress increases with strain. The length of yield plateau is a function of the steel strength and bar size. The strain hardening curve is modeled as non-linear relationship and terminates at the ultimate tensile strain, ε_{su}.

Plastic hinge length L_p is used to obtain ultimate rotation values from ultimate curvatures. Simplest form of plastic hinge length is obtained by following expression developed by the Paulay and Priestley in 1992:

$$L_p = 0.08L + 0.022\, f_{ye}\, d_{bl} \geq 0.044\, f_{ye}\, d_{bl}$$

Where, H is the section depth, L is the distance from the critical section of the plastic hinge to the point of contraflexure, and f_{ye} and d_{bl} are the expected yield strength, and diameter of longitudinal reinforcement, respectively. The plastic hinges are assumed to be form at a distance $L_p/2$ from the support.

2.2. Plastic Hinge Properties in Members

In SAP2000 (CSI 2009), non-linearity in members is not distributed along their whole length; instead, lumped plasticity is to be modeled at desired location on structural members. A two dimensional cantilever model is created in

SAP2000 (CSI 2009) to carry out non-linear static analysis. RC pier is modeled as non-linear element with lumped plasticity by defining plastic hinge at fixed support shown in Figure 2. Non-linear material properties of all the structural members are require for specifying properties for plastic hinges in pushover analysis.

In RC piers, plastic hinges that generally develop are those corresponding to axial force– bending moment (P-M hinges), bending moment–bending rotation (M-θ hinges), and shear force-shear deformation (V-Δ). Typical P-M, V-Δ, and M- θ hinge properties for RC pier are shown in Figure 3.

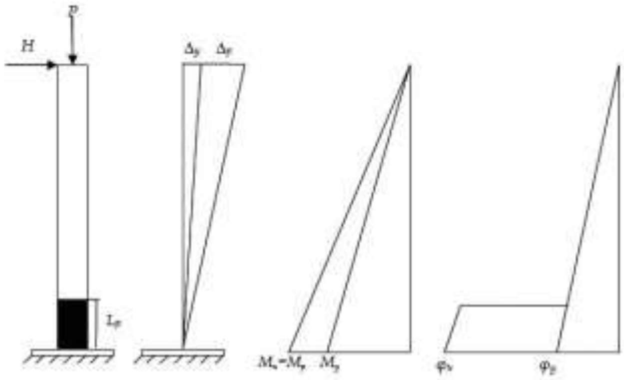

Figure 2. Lumped plasticity idealization of a cantilever and analysis model

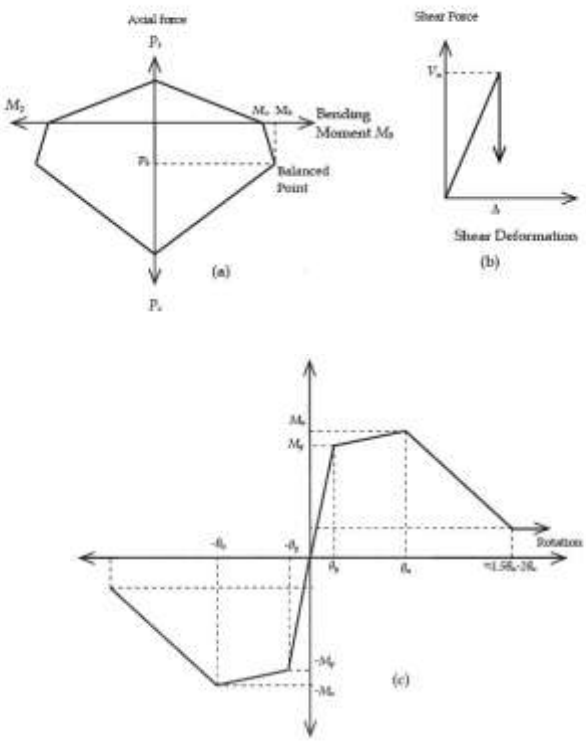

Figure 3. Typical plastic hinge propertied assigned to RC members (a) P-M (b) V-Δ, and (c) M-θ

In this study, Caltrans flexural hinge are used. The M-θ relationship for the designed sections is obtained using the moment-curvature (M-φ) relationship. The ultimate curvature φ_u at the failure limit state is defined as the concrete strain, or the confinement reinforcing steel reaching the ultimate strain. The displacement capacity Δ_{cap} of a member is on its rotation capacity, which in turn is based on its curvature capacity φ_u. The curvature capacity is determined by M-φ analysis. As per Caltrans, the plastic rotation θ_p is obtained by following Eq.:

$$\theta_p = L_p(\varphi_u - \varphi_{iy})$$

Where, φ_u and φ_{iy} are the ultimate curvature and idealized yield curvature, respectively.

The yield deflection Δ_y and plastic deflection Δ_p is obtained using Eqs.:

$$\Delta_y = \varphi_{iy} L^2/3$$

$$\Delta_p = \theta_p(L - L_p/2)$$

Where, L is the length of the member.

The total deflection capacity Δ_{cap} of section is obtained using Eq.:

$$\Delta_{cap} = \Delta_y + \Delta_p$$

The lateral load capacity obtained using M-θ relationship; it is given by following expression:

$$\text{Lateral Load Capacity} = M_p/L$$

Where, M_p is the plastic moment of the section obtained using the M-θ relationship.

The lateral load capacity (M_p/L) should be less than the shear strength V_{cap} to avoid brittle shear failure. Shear strength of the RC members were calculated using the IS 456:2000. If shear strength V_{cap} exceeds the lateral load capacity (M_p/L), then the brittle shear failure will occur, and shear hinge will be developed in the sections. Thus for no shear failure following condition should be satisfied:

$$M_p/L < V_{cap}$$

Shear failure of the members should be taken into consideration by assigning shear hinges in RC piers. Shear hinge properties are defined in such a way that when shear force in member reaches its capacity, the member fails immediately.

III. ANALYSIS PROCEDURE

Load patterns have been defined as dead load or live load, etc., and then load cases corresponding to non-linear static analysis were defined. Firstly, the Gravity Load Case is defined, which corresponds to the gravity load as well as other permanent loads acting on the structure. Secondly, in the Final Pushover Case, the stiffness of the members of structures at the end of non-linear Gravity Load Case has been considered as initial condition. More than one pushover cases are run in the same analysis. Pushover analysis cases can either be force controlled, i.e., structure is pushed at certain defined force level, or they can be displacement controlled, i.e., structure is pushed to a certain target specified displacement. In this study, Gravity Load

Case is force controlled and Final Pushover Case is displacement controlled, same is used in the present study.

Analysis model is run after necessary inputs, such as material properties, plastic hinge properties are given. SAP2000 (CSI 2009) allows increasing the maximum number of steps by modifying the non-linear parameters for the analysis. There are three methods of hinge unloading, namely, unload entire structure, apply local distribution, and restart secant stiffness. Any of three methods can complete analysis which is based on the trial and error. Unload entire structure method is used for hinge unloading to complete the analysis.

IV. PARAMETRIC STUDY AND RESULTS

Attempt has been made to study the effect of the Diameter of confinement reinforcement, Spacing of the confinement Reinforcement, Grade of Concrete, Axial load level on behavior of RC bridge pier section. It is studied with following variables.

4.1. Rectangle section

To study the effect of the confinement of concrete on the behavior of rectangular section, the diameter of confinement reinforcement varied (10mm, 12mm, 16mm, 20mm, 25mm) and spacing of the confinement ring also varied (50mm, 100mm, 150mm, 200mm, 250mm ,300mm) in following cases as shown in Table: 1

Table 1: Details of Rectangle Section

	Size of section		Grade of concrete	Long reinforcement details		Pt%
	B (mm)	D (mm)		DIA. (mm)	NO.	
Case -A	1600	2900	M40	32	60	1
Case -B	1600	2900	M50	32	60	1
Case -C	1600	2900	M60	32	60	1
Case -D	1600	2900	M70	32	60	1

4.2. Circular section

To study the effect of the confinement of concrete on the behavior of circular section, the diameter of confinement reinforcement varied (10mm, 12mm, 16mm, 20mm, 25mm) and spacing of the confinement ring also varied (50mm, 100mm, 150mm, 200mm, 250mm ,300mm) in following cases as shown in Table: 2.

Table 2: Details of circular Section

	Dia of section	Grade of concrete	Long reinforcement details		Pt%
			DIA. (mm)	NO.	
Case -A	2400	M40	32	1x57	1
Case -D	2400	M50	32	1x57	1
Case -E	2400	M60	32	1x57	1
Case -F	2400	M70	32	1x57	1

Table 3: Comparison of confinement reinforcement required in rectangle section

CLASS A WITH AXIAL LOAD 10 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	18	16	10
100	8	25	20	10
150	8	32	25	12
200	8	36	28	12
250	8	40	32	16
300	8	45	36	16

CLASS A WITH AXIAL LOAD 20 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	18	16	10
100	8	25	22	12
150	8	32	25	12
200	8	36	30	16
250	8	40	36	20
300	8	45	36	20

CLASS A WITH AXIAL LOAD 30 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	18	16	10
100	8	25	22	12
150	8	32	28	16
200	8	36	32	20
250	8	40	36	25
300	8	45	40	25

CLASS B WITH AXIAL LOAD 10 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	20	16	10
100	8	28	22	10
150	8	32	28	12
200	8	40	32	12
250	8	45	36	16
300	8	50	40	20

CLASS B WITH AXIAL LOAD 20 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	20	16	10
100	8	28	25	12
150	8	32	28	16
200	8	40	36	20
250	8	45	36	20
300	8	50	40	25

CLASS B WITH AXIAL LOAD 30 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	20	18	12
100	8	28	25	16
150	8	32	32	20
200	8	40	36	25
250	8	45	40	25
300	8	50	45	25

CLASS C WITH AXIAL LOAD 10 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	20	18	10
100	8	32	25	12
150	8	36	32	12
200	8	45	36	16
250	8	50	40	16
300	8	50	45	20

CLASS C WITH AXIAL LOAD 20 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	20	18	10
100	8	32	28	12
150	8	36	32	16
200	8	45	36	20
250	8	50	40	25
300	8	50	45	25

CLASS C WITH AXIAL LOAD 30 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	20	20	12
100	8	32	28	16
150	8	36	32	25
200	8	45	40	25
250	8	50	45	25
300	8	50	50	25

CLASS D WITH AXIAL LOAD 10 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	22	18	10
100	8	32	28	12
150	8	40	32	16
200	8	45	40	16
250	8	50	45	20
300	8	---	45	25

CLASS D WITH AXIAL LOAD 20 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	22	20	12
100	8	32	28	16
150	8	40	36	20
200	8	45	40	25
250	8	50	45	25
300	8	---	50	25

CLASS D WITH AXIAL LOAD 30 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	22	20	16
100	8	32	28	20
150	8	40	36	25
200	8	45	40	25
250	8	50	45	---
300	8	---	50	---

Table 4: Comparison of confinement reinforcement required in circular section

CLASS A WITH AXIAL LOAD 10 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	20	28	20
100	8	22	36	25
150	8	28	45	---
200	8	32	50	---
250	8	36	55	---
300	8	40	61	---

CLASS A WITH AXIAL LOAD 20 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	20	28	20
100	8	22	40	25
150	8	28	45	---
200	8	32	52	---
250	8	36	58	---
300	8	40	64	---

CLASS A WITH AXIAL LOAD 30 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	20	28	25
100	8	22	40	---
150	8	28	50	---
200	8	32	54	---
250	8	36	61	---
300	8	40	67	---

CLASS D WITH AXIAL LOAD 10 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	22	28	20
100	8	25	40	20
150	8	32	50	---
200	8	36	55	---
250	8	40	62	---
300	8	45	68	---

CLASS D WITH AXIAL LOAD 20 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	22	32	25
100	8	25	45	---
150	8	32	50	---
200	8	36	58	---
250	8	40	65	---
300	8	45	71	---

CLASS D WITH AXIAL LOAD 30 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	22	32	25
100	8	25	45	---
150	8	32	53	---
200	8	36	61	---
250	8	40	68	---
300	8	45	74	---

CLASS E WITH AXIAL LOAD 10 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	25	32	20
100	8	28	45	25
150	8	32	52	---
200	8	40	60	---
250	8	45	67	---
300	8	50	74	---

CLASS E WITH AXIAL LOAD 20 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	25	32	25
100	8	28	45	---
150	8	32	55	---
200	8	40	63	---
250	8	45	71	---
300	8	50	77	---

CLASS E WITH AXIAL LOAD 30 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	25	36	25
100	8	28	50	---
150	8	32	57	---
200	8	40	66	---
250	8	45	74	---
300	8	50	81	---

CLASS F WITH AXIAL LOAD 10 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	25	36	20
100	8	32	50	---
150	8	36	57	---
200	8	40	65	---
250	8	45	73	---
300	8	50	80	---

CLASS F WITH AXIAL LOAD 20 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	25	36	25
100	8	32	50	---
150	8	36	59	---
200	8	40	68	---
250	8	45	76	---
300	8	50	83	---

CLASS F WITH AXIAL LOAD 30 %				
Spacing	IRC	IS 1893(3)	Caltrans	DRIFT 2.5 %
50	8	25	36	25
100	8	32	50	---
150	8	36	62	---
200	8	40	71	---
250	8	45	80	---
300	8	50	87	---

V. CONCLUSIONS

It was found that the requirement of confinement reinforcement in rectangle section is Approximately 1.89 to 4.84 times higher in case of IS:1893 Part-3(Draft) and 1.54 to 4.34 times higher in case of Caltran Guidelines provisions as compared to analysis result obtained by SAP 2000, Referring Table:3.

It was found that the requirement of confinement reinforcement in circular section is Approximately, 1.25 to 3.24 times higher in case of Caltran Guidelines and 0.77 to 1.56 times higher in case of IS:1893 Part-3(Draft) provisions as compare to analysis result obtained by SAP 2000, Referring Table:4.

Spacing of confinement reinforcement is playing important role for drift capacity of section. Increasing the spacing and diameter of transverse steel and maintaining the same ratio of transverse reinforcement reduce the drift capacity. So the centre-to-centre transverse steel spacing in column should not greater than 100 mm for circular section and 150 mm for rectangle section

REFERENCES

[1] Mander, Priestly and Park “Theoretical Stress-Strain Model for Confined Concrete”, Journal of Structural Engineering, ASCE, V.114, No. 8, p. 1827-1849, (1988).

[2] Jain, S.K., Murty, C.V.R., Arlekar, J., and Jain, C.K. "Performance of Buildings During the Jabalpur Earthquake of 22 May 1997", Proceedings of the Workshop on Earthquake Disaster Preparedness, Roorkee, pp. 283 – 289, (1997).

[3] Scott, Park, Priestly “Stress-Strain Behavior of Concrete Confined by Overlapping Hoops at Law and High Strain Rates.” ACI J., 79(1), 13-27, (1982)

[4] IRC 6: 2000, “Standard Specifications and Code of Practice for Road Bridges, Section: II Loads and Stresses”, The Indian Road Congress, New Delhi, (2000).

[5] IRC 21: 2000, “Standard Specifications and Code of Practice for Road Bridges, Section: III, Cement Concrete (Plain and Reinforced)”, The Indian Road Congress, New Delhi, (2000).

[6] IS 456: 2000, “Code of Practice for Plain and Reinforced Concrete”, Bureau of Indian Standards, New Delhi, (2000).

[7] IS 1893 (Part 3) (Draft), “Criteria for Earthquake Resistant Design of Structures” (part-3) Bridges & Retaining walls.

[8] Caltrans 1998, “Seismic Design Criteria for Highway Bridges”, California Department of Transportation, California, (1998).