International Journal of Advance Research in Engineering, Science & Technology(IJAREST),
ISSN(O):2393-9877, ISSN(P): 2394-2444,
Volume 2,Issue 12, December-2015, Impact Factor: 2.125

Effect of Axial load on deformation capacity of RC column

N. G. Patoliya[1], Prof. C. S. Sanghvi[2]

[1]Narmada, Water Resources, Water Supply and Kalpsar Department, Government of Gujarat,, patoliyanirav@yahoo.co.in

[2]Applied Mechanics Department, L. D. College of Engineering, cssanghvi@rediffmail.com

Abstract

Generally, it is believed that ultimate displacement of RC member is based on strength of the member. This definition does not reflect the true deformation capacity of member. Thus, present study is carried out to quantify the effect of axial load on deformation capacity of RC column. The study provides a clear insight into the fundamental mechanism of concrete structural deformation.

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

I. INTRODUCTION

Seismic design of member involve both strength and ductility. A popular way of defining the ductility of a structure is to choose the maximum displacement of a structure at which the structural resistance is at a certain proportion of its initial load carrying capacity under the specified load pattern. For example, the ultimate curvature of an RC member is defined as the curvature on the softening branch of the M-θ response curve at which the Moment resistant of the member drop to 80% of the maximum moment at the fourth cycle of load (Watsons, at el. 1994). This definition is based on the strength concept of the member and is widely accepted in the literature. However, this definition of ultimate displacement usually can not reflect the true deformation capacity of the member and does not provide a uniform benchmark to reflect the same degree of damage to the RC member.

This work studies the fundamental mechanism of deformation of RC members by relating the different stages of material damage of concrete to different stages of deformation of RC members. The different stages of deformation are used to define ultimate displacement of RC members and displacement ductility. The effect of axial load is then studies. These studies provide a clear insight into the various factors that affect the flexural deformation capacity of RC columns.

II. DEFINITION AND MODELING OF DEFORMATION MECHANISM IN RC MEMBERS

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.

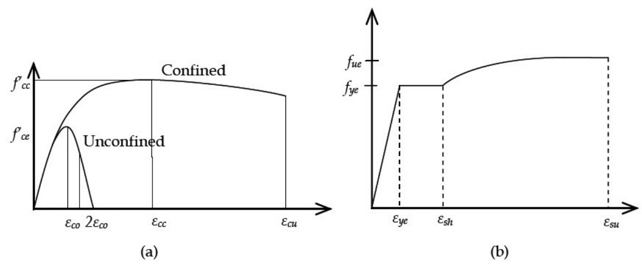

Figure 1. Stress-strain model for (a) Concrete (b) Reinforcing Steel used in the Pushover Analysis by SAP 2000 [CSI 2009]

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.

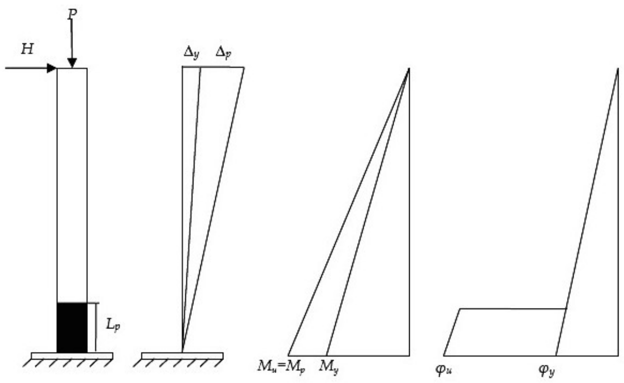

Figure 2. Lumped plasticity idealization of a cantilever and analysis model

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.

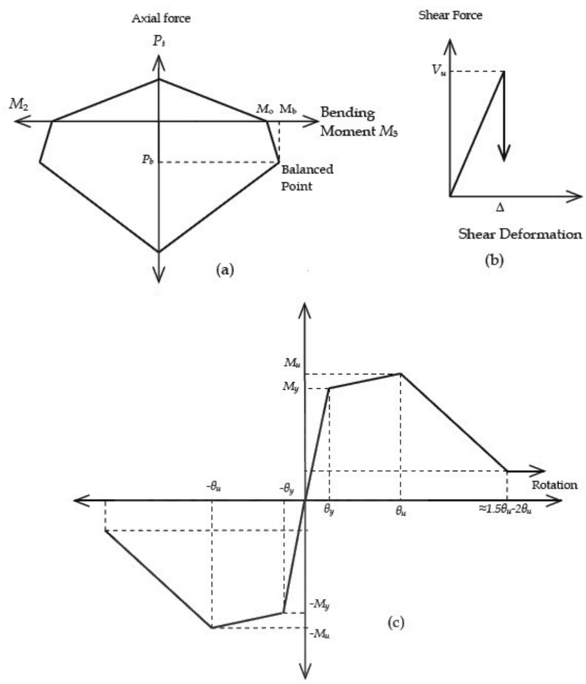

Figure 3. Typical plastic hinge propertied assigned to RC members (a) P-M (b) V-Δ, and (c) M-θ

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:

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

An initial axial load of 10%, 20% and 30% of concrete crushing strength is applied to study the effect of the axial load level on deformation capacity of rectangular section 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

An initial axial load of 10%, 20% and 30% of concrete crushing strength is applied to study the effect of the axial load level on deformation capacity of circular section 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: Effect of Axial load and amount of confinement reinforcement on Drift capacity of rectangle section

Dia. (mm)	Spacing (mm)	Axial Load level	Drift capacity (%)			
			M40	M50	M60	M70
10	50	10%	3.272	3.237	3.224	3.223
10	50	20%	3.578	3.273	2.974	2.258
10	50	30%	2.674	2.422	1.979	1.425
10	100	10%	2.844	2.746	2.305	1.730
10	100	20%	2.043	1.882	1.422	1.126
10	100	30%	1.523	1.255	0.906	0.813
10	150	10%	2.312	1.781	1.517	1.241
10	150	20%	1.633	1.096	0.912	0.799
10	150	30%	1.089	0.897	0.736	0.360
10	200	10%	1.805	1.426	1.212	0.763
10	200	20%	1.168	0.957	0.796	0.379
10	200	30%	0.866	0.511	0.183	0.148
10	250	10%	1.630	1.294	1.115	0.763
10	250	20%	0.926	0.837	0.480	0.130
10	250	30%	0.576	0.153	0.144	0.134
10	300	10%	1.378	1.214	1.086	0.700
10	300	20%	0.862	0.503	0.142	0.115
10	300	30%	0.272	0.143	0.134	0.119

Continues Table -3 over next page...

Dia. (mm)	Spacing (mm)	Axial Load level	Drift capacity (%)			
			M40	M50	M60	M70
12	50	10%	3.236	3.204	3.189	3.177
12	50	20%	3.354	3.342	3.346	3.348
12	50	30%	3.511	3.516	3.471	2.781
12	100	10%	3.285	3.249	3.241	3.291
12	100	20%	3.329	3.044	2.549	2.074
12	100	30%	2.533	2.255	1.663	1.339
12	150	10%	3.230	3.043	2.861	2.212
12	150	20%	2.516	2.140	1.743	1.291
12	150	30%	1.899	1.420	1.137	0.920
12	200	10%	2.729	2.570	1.972	1.678
12	200	20%	1.950	1.624	1.176	0.960
12	200	30%	1.436	1.076	0.849	0.532
12	250	10%	2.386	1.895	1.575	1.368
12	250	20%	1.707	1.140	0.948	0.747
12	250	30%	1.152	0.831	0.443	0.158
12	300	10%	1.965	1.708	1.304	0.758
12	300	20%	1.254	0.944	0.802	0.321
12	300	30%	0.929	0.498	0.614	0.145
16	50	10%	3.210	3.188	3.172	3.159
16	50	20%	3.326	3.309	3.293	3.295
16	50	30%	3.431	3.424	3.425	3.435
16	100	10%	3.252	3.229	3.212	3.194
16	100	20%	3.390	3.382	3.387	3.221
16	100	30%	3.554	3.158	2.802	2.077
16	150	10%	3.288	3.253	3.247	3.252
16	150	20%	3.397	3.129	2.589	1.908
16	150	30%	2.581	2.340	1.709	1.368
16	200	10%	3.335	3.363	3.194	2.918
16	200	20%	2.808	2.311	1.945	1.566
16	200	30%	2.135	1.535	1.257	1.128
16	250	10%	3.045	2.886	2.500	1.850
16	250	20%	2.197	2.022	1.502	1.211
16	250	30%	1.653	1.195	0.975	0.864
16	300	10%	2.773	2.369	1.799	1.517
16	300	20%	1.962	1.475	1.205	0.977
16	300	30%	1.471	0.969	0.866	0.704
20	50	10%	3.214	3.184	3.166	3.153
20	50	20%	3.309	3.287	3.276	3.272
20	50	30%	3.424	3.390	3.397	3.401
20	100	10%	3.230	3.207	3.195	3.182
20	100	20%	3.343	3.339	3.340	3.348
20	100	30%	3.503	3.504	3.542	3.395
20	150	10%	3.264	3.237	3.218	3.205
20	150	20%	3.387	3.397	3.477	3.370
20	150	30%	3.695	2.994	2.469	1.983
20	200	10%	3.288	3.339	3.247	3.246
20	200	20%	3.386	3.282	2.692	2.028
20	200	30%	2.679	2.414	1.778	1.420
20	250	10%	3.305	3.295	3.165	2.911
20	250	20%	3.130	2.554	2.085	1.734
20	250	30%	2.289	1.694	1.360	1.089
20	300	10%	3.419	3.247	3.039	2.309
20	300	20%	2.524	2.026	1.893	1.348
20	300	30%	2.015	1.339	1.218	0.977
25	50	10%	3.204	3.172	3.159	3.211
25	50	20%	3.300	3.270	3.259	3.405
25	50	30%	3.399	3.372	3.356	3.379
25	100	10%	3.232	3.195	3.177	3.165
25	100	20%	3.350	3.305	3.293	3.290
25	100	30%	3.462	3.418	3.420	3.418
25	150	10%	3.237	3.237	3.194	3.176
25	150	20%	3.382	3.333	3.328	3.330
25	150	30%	3.545	3.458	3.466	3.522
25	200	10%	3.274	3.224	3.212	3.199
25	200	20%	3.416	3.368	3.369	3.376
25	200	30%	3.682	3.562	3.431	2.482
25	250	10%	3.292	3.241	3.223	3.211
25	250	20%	3.498	3.402	3.404	3.295
25	250	30%	3.626	3.131	2.566	2.098
25	300	10%	3.310	3.260	3.241	3.234
25	300	20%	3.289	3.471	3.080	2.526
25	300	30%	3.066	2.736	2.051	1.621

Table 4: Effect of Axial load and amount of confinement reinforcement on Drift capacity of circular section

Dia. (mm)	Spacing (mm)	Axial Load level	Drift capacity (%)			
			M40	M50	M60	M70
10	50	10%	1.552	1.485	1.405	1.191
10	50	20%	1.306	1.112	0.935	0.865
10	50	30%	1.036	0.880	0.660	0.179
10	100	10%	1.246	1.219	1.039	0.671
10	100	20%	0.772	0.231	0.142	0.134
10	100	30%	0.171	0.153	0.141	0.128
10	150	10%	0.995	0.701	0.421	0.113
10	150	20%	0.150	0.136	0.122	0.117
10	150	30%	0.149	0.134	0.122	0.113
10	200	10%	0.640	0.377	0.113	0.106
10	200	20%	0.140	0.124	0.115	0.110
10	200	30%	0.139	0.125	0.113	0.106
10	250	10%	0.457	0.127	0.109	0.101
10	250	20%	0.134	0.119	0.110	0.106
10	250	30%	0.130	0.118	0.109	0.101
10	300	10%	0.325	0.113	0.106	0.096
10	300	20%	0.127	0.116	0.107	0.103
10	300	30%	0.126	0.114	0.106	0.095

Continues Table -4 over next page…

Dia. (mm)	Spacing (mm)	Axial Load level	Drift capacity (%)			
			M40	M50	M60	M70
12	50	10%	2.026	1.904	1.788	1.670
12	50	20%	1.710	1.576	1.317	1.100
12	50	30%	1.479	1.130	1.029	0.849
12	100	10%	1.500	1.442	1.371	1.151
12	100	20%	1.259	1.060	0.533	0.236
12	100	30%	0.770	0.179	0.162	0.150
12	150	10%	1.325	1.282	1.002	0.610
12	150	20%	0.746	0.221	0.140	0.131
12	150	30%	0.168	0.150	0.138	0.127
12	200	10%	1.145	0.824	0.567	0.227
12	200	20%	0.234	0.139	0.125	0.119
12	200	30%	0.152	0.138	0.124	0.116
12	250	10%	0.818	0.506	0.195	0.110
12	250	20%	0.144	0.128	0.119	0.113
12	250	30%	0.144	0.127	0.116	0.109
12	300	10%	0.609	0.346	0.112	0.105
12	300	20%	0.139	0.122	0.113	0.109
12	300	30%	0.138	0.122	0.113	0.105
16	50	10%	2.567	2.398	2.224	2.078
16	50	20%	2.169	1.980	1.815	1.520
16	50	30%	1.888	1.564	1.283	1.060
16	100	10%	1.796	1.713	1.612	1.488
16	100	20%	1.506	1.409	1.072	0.977
16	100	30%	1.200	0.999	0.837	0.547
16	150	10%	1.519	1.461	1.386	1.059
16	150	20%	1.274	1.091	0.923	0.584
16	150	30%	1.014	0.510	0.177	0.162
16	200	10%	1.381	1.346	1.158	0.971
16	200	20%	1.152	0.794	0.314	0.147
16	200	30%	0.552	0.170	0.153	0.144
16	250	10%	1.294	1.261	1.091	0.741
16	250	20%	0.886	0.338	0.143	0.136
16	250	30%	0.173	0.153	0.141	0.128
16	300	10%	1.231	1.157	0.690	0.481
16	300	20%	0.543	0.146	0.134	0.125
16	300	30%	0.161	0.144	0.131	0.122
20	50	10%	3.162	2.936	2.725	2.545
20	50	20%	2.681	2.438	2.230	2.056
20	50	30%	2.347	2.114	1.739	1.442
20	100	10%	2.138	2.014	1.872	1.747
20	100	20%	1.796	1.664	1.384	1.152
20	100	30%	1.567	1.183	1.081	0.889
20	150	10%	1.753	1.679	1.575	1.326
20	150	20%	1.470	1.256	1.050	0.962
20	150	30%	1.170	0.981	0.822	0.513
20	200	10%	1.558	1.494	1.417	1.089
20	200	20%	1.308	1.118	0.944	0.791
20	200	30%	1.039	0.801	0.339	0.179
20	250	10%	1.436	1.393	1.209	1.017
20	250	20%	1.201	1.039	0.799	0.443
20	250	30%	0.959	0.330	0.171	0.158
20	300	10%	1.366	1.330	1.146	0.971
20	300	20%	1.137	0.911	0.384	0.150
20	300	30%	0.592	0.173	0.156	0.146
25	50	10%	3.715	3.680	3.654	3.687
25	50	20%	3.960	3.582	3.266	2.997
25	50	30%	3.483	3.111	2.804	2.554
25	100	10%	2.970	2.751	2.565	2.380
25	100	20%	2.518	2.286	2.102	1.909
25	100	30%	2.196	1.977	1.635	1.210
25	150	10%	2.339	2.199	2.066	1.911
25	150	20%	1.974	1.817	1.527	1.258
25	150	30%	1.733	1.429	1.069	0.978
25	200	10%	2.020	1.904	1.790	1.679
25	200	20%	1.704	1.573	1.105	0.996
25	200	30%	1.344	1.130	1.026	0.767
25	250	10%	1.816	1.734	1.640	1.378
25	250	20%	1.524	1.296	1.090	0.901
25	250	30%	1.215	1.014	0.849	0.452
25	300	10%	1.690	1.610	1.520	1.164
25	300	20%	1.415	1.090	1.011	0.843
25	300	30%	1.115	0.944	0.495	0.183

V. CONCLUSIONS

It is noting based on the analysis results Referring Table: 3 and Table:4 that the axial load plays a significant role in the behavior of the column. An increase in axial load leads to increase in the flexural capacity, but decreases the ductility. A relatively high axial load makes the concrete to enter in its inelastic range before steel, so more confinement is required to achieve enough ductility to delay the failure in a seismic region. Referring to analysis results, it was found that rectangle section performs better then circular 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).