Non-Linear Static Analysis of Reinforced Concrete Bridge

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Abstract - *Seismic vulnerability assessment to being structure has gained nation-wide attention, particularly after 2001 Gujarat Earthquake and 2005 Kashmir Earthquake. There isn't important trouble available in literature for seismic evaluation of existing bridges although bridge is a very important structure in any country. In order to evaluate existing bridges and to suggest design of retrofit schemes performance based nonlinear pushover analysis is applied in some international codes but no such inclusion is found in Indian Codes. In order to draw comparison between pushover analysis schemes with Indian method of Seismic analysis, the present project aimed to carry out a seismic evaluation of RC Bridges using nonlinear static (pushover) analysis. The two series of model bridges are analyzed using displacement coefficient method (FEMA 356), capacity spectrum method (ATC40), displacement modification method (FEMA440) and equivalent linearization method (FEMA 440). Each series consist of five bridges one with varying span and other with varying pier height. Few parameters that are used for bridge analysis are modified in this study. The evaluation results presented here shows that the modeled bridges designed as per IS codes falls short to meet the desired performance level as per non-linear pushover scheme.*

Key Words: Nonlinear Pushover Analysis, FEMA, ATC 40

1. INTRODUCTION

India has had a number of the world's greatest earthquake since the last century. The Himalayan region of India is more prone to earthquake where the seismic magnitude is about of 8.0 richer scale and leads to drastically high damage. After 2001 Gujarat Earthquake and 2005 Kashmir Earthquake, the resignation-wide attention to the seismic vulnerability assessment of old structure. The IS Code for seismic design is also revised because magnitude of earthquake changes its effect was dangerous after studying past effect over seismic prone region. Many structural engineers gave their effort for its revision so that the building that they make should be safe against seismic effect. The magnitudes of the design seismic forces have been considerably enhanced in general, and the seismic zone category of some regions has also been upgraded. Data present in various literature like IITM-SERC Manual, 2005 available that presents step-by-step procedures to evaluate multi-storey buildings. For seismic analysis we follow static nonlinear analysis also called pushover analysis with the help of code FEMA356

The attention for existing bridges is comparatively less. Bridge provides a better transportation system. But, a large number of bridges were designed and constructed without considering seismic forces. It is must to design and check the bridges in seismic condition. For retrofitting of existing bridges , buildings or any type of structure currently there are no guidelines that will assist engineer in maintaining structure life. In order to address this problem, the present work aims to carry out a seismic evaluation of RC bridges using nonlinear static (pushover) analysis. Nonlinear static (pushover) analysis as per FEMA 356 is not compatible for bridge structure. Bridges are structurally very different from a multi-storey building.

1.1Pushover Analysis

It is used to find strength, drift of building already exists, effect on earthquake loading and its demand curve, its failure chances etc. For working in pushover ATC 40 and FEMA 356, euro codes PCM 3274 are used. The buildings are converted to mathematical models using nonlinear load deformation characteristics of every element and monotonically loads are applied. The target displacement and lateral loads are implemented where target displacement is the maximum displacement building can bear during seismic forces.

The seismic demand parameters are global displacements (at roof for any other reference point), storey drifts, storey forces, component deformation and forces.

Response characteristics that can be obtained from the pushover analysis are summarized as follows:

1. Force and displacement capacities curves are estimated for the structure.

2. We will get axial forces, shear values, bending moments of elements that are brittle in nature and for ductile materials deformation demand is estimated.

3. Elements which fails and their effect on stability.

4. Identification of the critical regions, where the inelastic deformations are expected to be high and identification of strength irregularities (in plan or in elevation) of the building.

1.2 Pushover Analysis Procedure

In this lateral load is monotonically increased according to predefined distribution pattern along building height. (Fig1.1a). the displacement to building is maintained until control nodes reaches the target displacement or at a state of building collapse. Cracking pattern, plastic hinge generation and failure is observed. Plot between nodal displacement and base shear is drawn in graph.

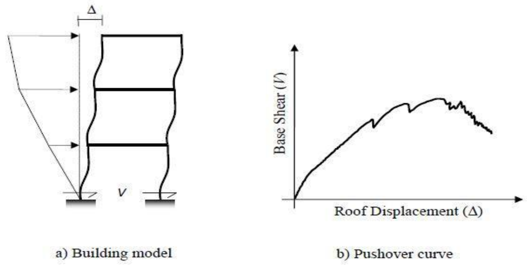

Fig 1.1: Schematic representation of pushover analysis procedure

Generation of base shear – control node displacement curve is single most important part of push over analysis. The curve obtained is also called pushover or capacity curve and gives target displacement. So the pushover analysis may be carried out twice: (a) first time till the collapse of the building to estimate target displacement and (b) next time till the target displacement to estimate the seismic demand. The values of lateral forces, storey drift, target displacements are calculated using pushover at various level of buildings. These values are then compared with the values of predefined limits of same structural capacity.

Orthogonal axis is considered for individual analysis unless and until it is not needed we didn't perform it using bi-directional axis. Where results that are obtained depend on control nodes and load pattern. The lateral load generally applied in both positive and negative directions along with gravity load (dead load and a portion of live load) to study the actual behavior.

3. OBJECTIVE

The objectives of study are as follows:-

1.To study the standard pushover analysis procedures and other improvement in pushover methodology available in literature.

2.To carry out a detailed exhaustive study of pushover analysis for a number of reinforced concrete bridges using standard pushover analysis and other improved pushover method.

3.To compare seismic analysis results performed as per Indian standards with the results of pushover analysis for bridges.

3. RESEARCH METHODOLOGY

1. Although literature review on application of Adaptive pushover analysis for RCC Bridge and seismic performance of bridge piers.

2. Carryout bridge modelling in suitable software and design the bridge as per design code IRC 21-2000 &IRC 6-2002 and perform pushover analysis.

3. We will perform pushover on bridge models with different spans

4. Repeat the Bridge modelling and pushover analysis with varying pier heights.

5. Compare the result of non static linear pushover demand with design demand based on Indian codes and arrive at a conclusion.

6. The performance of different models are compared to each other.

Accurate modeling of the non linear properties of various structural elements is very important in nonlinear analysis. In the present study, piers were modeled with inelastic flexural deformations using point plastic model. We should provide Mass distributions, stiffness; strength should be represented by the model. Modeling of the material properties and structural elements used in the present study are discussed here.

3.1 Structural Elements - We have modeled the 3D elements of Piers, Cap, Girders decks for performing pushover. The girder-pier joints are modeled by giving end-offsets to the frame elements, to obtain the bending moments and forces at the beam and column faces. The pier-cap joints are assumed to be rigid (Fig. 3.1).The pier end at foundation was considered as fixed. Moment releases are applied at both ends of all the girders. This is done to obtain simply supported condition as per actual structure. All the pier elements are modeled with nonlinear properties at the possible yield locations. Deck is not modeled physically. Weight and mass of deck is also considered in the dead load of structure.

3.2 Bridge Geometry-In this study two set of bridges one with fixed span and varying pier height and the other with fixed pier height and varying span are modeled.

3.3 Fixed Span Bridges -The bridge considered consists of two spans each of 30m. The bridge deck is supported by single-span concrete girders. Girders are placed on the concrete pier-caps through the bearing and locked in the transverse direction. The supporting piers heights are same for single bridge and are varied to obtain the desired series. Bridge mode in WBRH5M, NWBR H10M. NWBR H15M, NWBR H20M & NWBR H25M with pier heights of 5m, 10m, 15m, 20m and 25m are used in the study. The width of the bridge is 10.5m

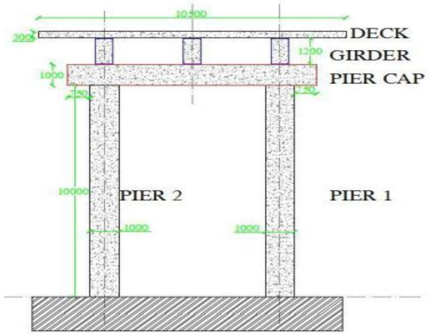

Fig. 3.1: Typical Cross-sectional details of the bridge

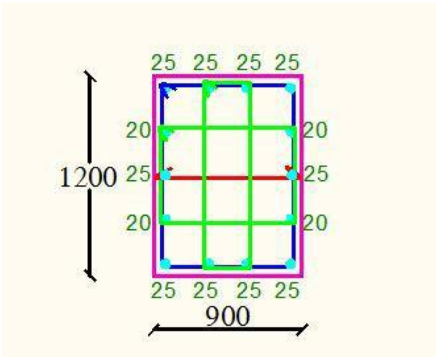

Fig. 3.2: TypicalDetails of the pier section

3.4Fixed Pier Height Bridges -The Bridge considered consists of two spans of same length. The bridge deck is supported by single-span concrete girders. Girders are placed on the concrete pier-caps through the bearing and locked in the transverse direction. The supporting piers height is 15 m and same for all bridges and span length are varied to obtain the desired series. Bridge models NWBR S20M, NWBR S30M. NWBR S40M, NWBR S50M & NWBR S60M with span of 20m, 30m, 40 m, 50m and 60m are adopted for the study. The width of the bridge is 10.5m Fig. 3.1 presents a section view of the bridge in Y-Z plane that shows the pier and deck arrangement and dimensions. Pier cross-section is of rectangular size as shown in Fig. 3.2 The Bridge is modeled using commercial software SAP 2000 V18.0.1.Ultimate.A3Dcomputer model is shown in below.

3.5 Modeling of flexural plastic hinges. –The development of sound model to explicitly define then on linear behavior of the structural elements is integral in the implementation of pushover analysis. In the present study, a point-plasticity approach is adopted for modeling nonlinearity of RCC elements, plastic hinges are assumed to be applied in order to concentrate the load at specific point in frames. Piers in this study are modeled with flexure (P-M2-M3) hinges at possible plastic regions under lateral load (i.e., both ends of the beams and columns).

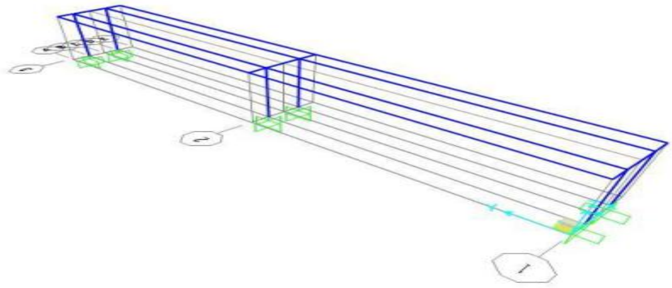

Fig 3.3: 3D model of the bridge

Plastic hinges are assumed at an offset of .05L from both ends. Lateral load are applied with flexural hinges so that actual response is seen in RCC component. In practical use, most often the default properties provided in the FEMA-356 and ATC-40 documents are preferred due to convenience and simplicity .SAP-2000 performs nonlinear static pushover analysis in corporate with the implementation of default flexural hinge properties based on FEMA-356andATC-40.Italsoallows modifying the default properties. In this study the concept of generated properties is used in SAP2000, when generated properties are used, the program which combines its built-in criteria (FEMA-356andATC-40) with the defined section properties for each object to generate the final hinge properties. Moment curvature analysis is carried out in order to get hinge properties which depend on reinforcement and area of element. The relation between concrete, rebars, Hinges its length is needed for calculation. Only axial forces are only required for pier flexural hinges, where rotation are needed along with gravity load conditions.

3.6Stress-Strain Characteristics for Concrete-The stress-strain curve of concrete in compression forms the basis for analysis of any reinforced concrete section. The characteristic and design stress-strain curves specified in most of design codes (IS 456: 2000, BS 8110) do not truly reflect the actual stress-strain behavior in the post-peak region, as (for convenience in calculations) it assumes a constant stress in this region (strains between 0.002 and 0.0035). Micro cracks and softening are seen in experimental testing. Also, models as per these codes do not account for strength enhancement and ductility due to confinement. However, the stress – strain relation specified in ACI318 M-02 consider some of the important features from actual behavior. A previous study (Chugh,2004) on stress- strain relation of reinforced concrete section concludes that the model proposed by Panagio tacos and Fardis (2001) represents the actual behavior best for normal-strength concrete. Accordingly, this model has been selected in the present study for calculating the hinge properties. This model is a modified version of Mander's model (Mander *et. al.*, 1988) where a single equation can generate the stress *fc* corresponding to any given strain ε_c:

$f_c = f'_{cc} x\, r / (r-1+x^n)$ where, $x=\varepsilon_c/\varepsilon_{cc}$; $r=E_c/(E_c-E_{sec})$; $E_c=5000 f'^{\,-1}_{co}$; $E_{sec}=f'_{cc}/\varepsilon_{cc}$ and f'_{cc} is the peak strength expressed as follows:

$$\varepsilon_{cc}=\varepsilon_{co}\left[1+5\left(\frac{f'_{cc}}{f'_{co}}-1\right)\right] \qquad \varepsilon_{cu}=0.004+\frac{0.6\rho_s f_{yh}\varepsilon_{sm}}{f'_{cc}}$$

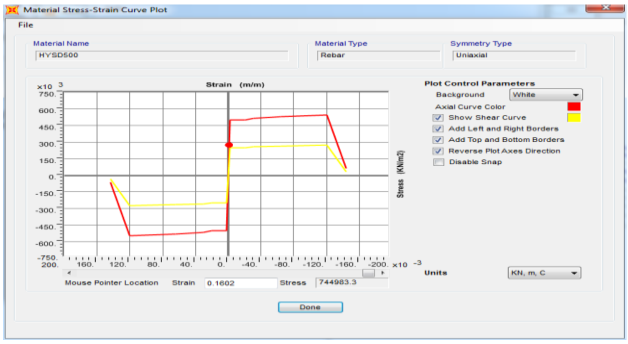

Fig3.4: Plot of stress-strain characteristics for M- 40 grade of concrete as per Modified Mander's model

The expressions for critical compressive strains (ref. Fig. 5.6) are expressed in this model as follows:

$$f'_{cc}=f'_{co}\left[1+3.7\left(\frac{0.5k_e\rho_s f_{yh}}{f'_{co}}\right)^{0.85}\right]$$

f'_{co} is unconfined compressive strength=0.75 f_{ck}, ρ_s=volumetric ratio of confining steel, f_{yh}=grade of the stirrup reinforcement, ε_{sm}= steel strain at maximum tensile stress and k_e is the "confinement effectiveness coefficient", having a typical value of 0.95 for circular sections and 0.75 for rectangular sections.

The advantage of using this model can be summarized as follows:

1. A single equation defines the stress-strain curve (both the ascending and descending branches) in this model.

2. The same equation can be used for confined as well as unconfined concrete sections.

3. The model can be applied to any shape of concrete member section confined by any kind of transverse reinforcement (spirals, cross ties, circular or rectangular hoops).

4. The validation of this model is established in many literatures.

3.7 Stress-Strain Characteristics for Reinforcing Steel -

The constitutive relation for reinforcing steel given in IS 456 (2000) is well accepted in literature and hence considered for the present study. The 'characteristic' and'design' stress strain curves specified by the Code for Fe-500 grade of reinforcing steel (in tension or compression) are shown in Fig. 3.6.

3.8 Moment-Rotation Parameters – The input of hinges that is found by moment curvature

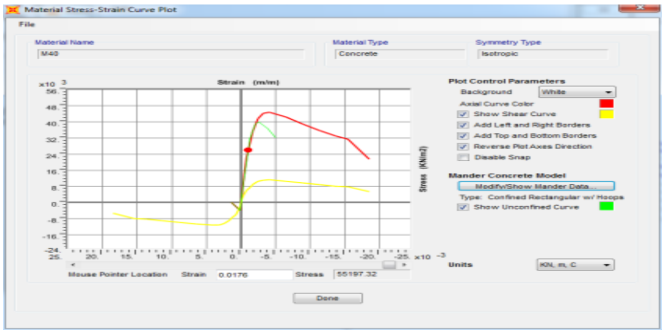

Fig3.5: Stress-strain relationship for reinforcement –IS 456 (2000)

A represent loaded condition
B show nominal yield strength and yield rotation θy
C show ultimate strength, ultimate rotation θu
D represent residual strength limited by=20%of yield strength
E show maximum deformation capacity =15θy or θu , whichever is greater.

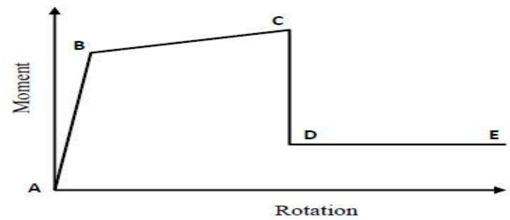

Fig. 3.6: Idealized moment-rotation curve of RC elements [;

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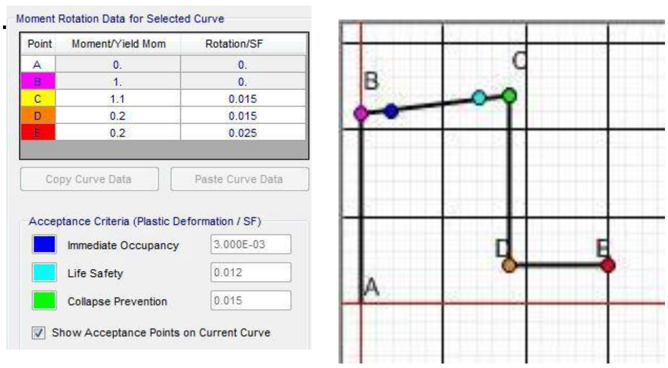


Fig. 3.7: Generated moment-rotation curve of RC elements with acceptance criteria

In this study hinges are defined as auto hinge types which are based on table in FEMA 356.Table6-8 (Concrete Column-Flexure). Item is selected as defining hinge behavior type.

Component type is primary with degree of freedom as P-M2-M3 type. Transverse reinforcement is conforming. The moment-rotation curve used by SAP2000 along with various performance level

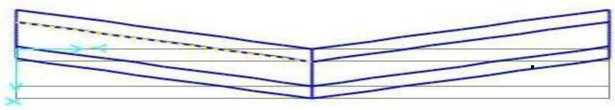
4. RESULTS AND DISCUSSIONS

The two series of model bridges are analyzed using displacement coefficient method (FEMA 356), capacity spectrum method (ATC 40), displacement modification method (FEMA 440) and equivalent linearization method (FEMA 440). Pushover in load control manner is applied for gravity loads and lateral analysis in transverse axis is performed in displacement controlled form. For Zone V IRC 112:2011 and 6:2016 PGA value 0.36 g results are obtained.

Modal Properties- Linear dynamic modal analysis was performed to obtain the modal properties of the bridge models. Table 4.1 shows the details of the important modes of the bridge intransverse direction (X direction). The table shows that participating mass ratio in thefirst mode and cumulative mass participating ratio for first four modes for modeledbridges. The average contribution of first mode in modal mass participation is 54.4%while the average cumulative mass participating ratio for first four modes is 96.4%. for building that are normal higher mode is obtained and is valuable. Fig-4.1 and fig. 4.2 show 1st four mode shapes. We assume 100% fundamental modes in structural response which is not for bridges.

Table 4.1: Elastic Dynamic Properties of the Bridge for Lateral vibration (X- direction)

S.NO	MODEL NAME	PRINCIPLE MODE				
		PERIOD (SEC)	FREQUENCY (Hz)	Eigen value (rad²/sec²)	Uy	TT**
1	NWBR H5M	0.18	5.556	1216.55	0.47	0.92
2	NWBR H10M	0.73	1.370	73.35	0.46	0.91
3	NWBR H15M	1.067	0.937	34.69	0.53	0.99
4	NWBR H20M	1.276	0.784	24.25	0.54	0.99
5	NWBR H25M	1.106	0.904	32.27	0.91	0.99
6	NWBR S20M	0.964	1.037	42.472	0.54	0.99
7	NWBR S30M	0.89	1.124	49.39	0.53	0.99
8	NWBR S40M	0.644	1.553	95.09	0.47	0.98
9	NWBR S50M	1.55	0.645	16.31	0.48	0.99
10	NWBR S60M	1.329	0.752	22.354	0.51	0.89



Uy =modal mass participation for first mode

TT**= cumulative mass participating ratio for first four modes

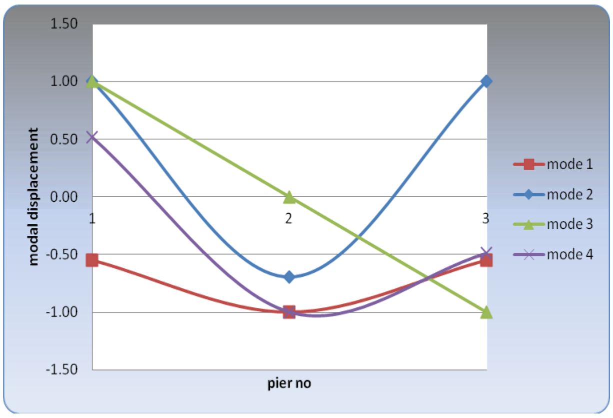


Fig. 4.1: First four modes of the bridge (normalized to Pier# 2)

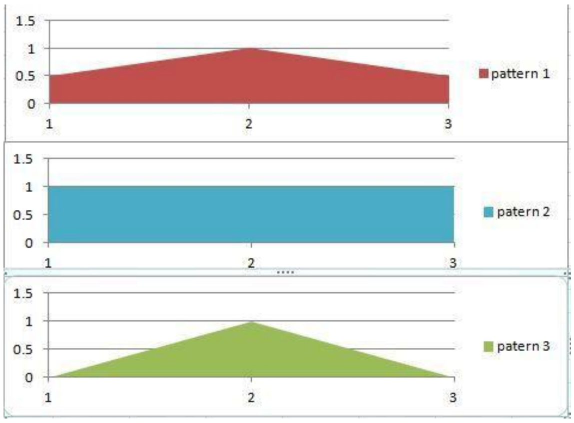
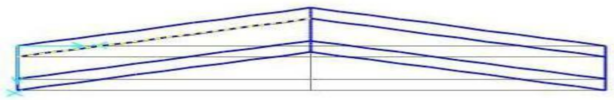
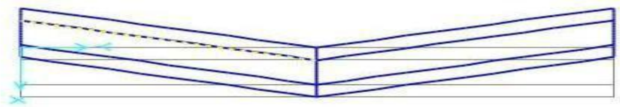


Fig. 4.2: Different lateral load pattern used

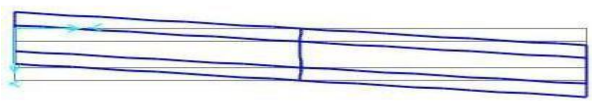
Pushover Analysis-Pushover analysis is carried out using FEMA 356 displacement coefficient method, ATC 40 capacity spectrum method, FEMA 440 equivalent linearization method (modified CSM) as well as FEMA 440 displacement modification method (Improvement for DCM). A triangular load pattern was used for standard pushover analysis (FEMA 356). Fig. 4.3 shows the load pattern used for standard pushover analysis.



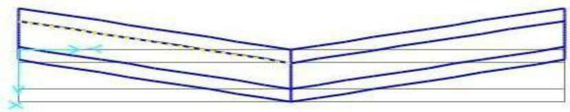
a) First mode



b) Second mode



c) Third mode



d) Fourth mode

Fig. 4.3: First four modes of the bridge (plan view)

Lateral Load Pattern-Three different load patterns are used to represent the load intensity produced by earthquake as shown in fig 4.3. The first pattern, which is the Trapezoidal Pattern, is based on lateral forces that are proportional to the total mass assigned to each node. The second pattern, which is uniform pattern, is based on standard load pattern as per FEMA 356. The third pattern, which is triangular, is based on shape of principle mode deformation as shown in fig 4.2

Capacity Curve -Capacity curve of the bridge as obtained from the four pushover analyses (displacement coefficient method capacity spectrum method, displacement modification method and equivalent linearization method) and three different load patterns are plotted and presented in Fig. 4.4. Fig. 4.4 shows that load pattern1 estimates a very high base-shear capacity of the bridge in transverse direction as compared to the triangular load pattern analysis. However the estimated ductility is almost same for all three load patterns. Fig 4.4 demonstrates the influence of lateral load pattern on the capacity curve of the structure. Lower shear capacity of bridge for triangular pattern load is caused by large deviation in base shear of individual piers. At performance point the base shear for pier 2 is almost same for all load patterns but at pier 1 and pier 3 there is large variation in base shear for different pattern resulting in variations in the total shear capacity of bridge.

There are different load pattern at different nodes i.e. load pattern1 give conservative results and closer to the full fledged time history analysis, hence capacity curves for various bridges with load pattern1 are further discussed.

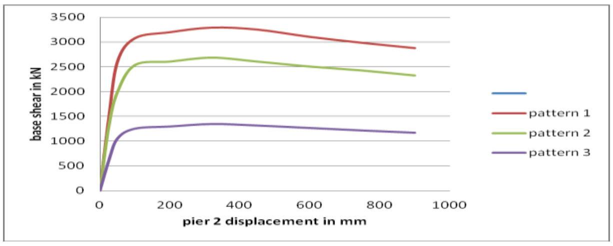


Fig. 4.4: Capacity curve of the bridge NWBR S30M

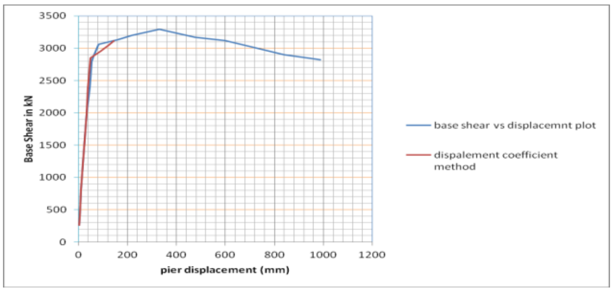


Fig. 4.5: Capacity curve of the bridge NWBR S30M by DCM

Capacity Curve for Displacement Coefficient Method- The Pushover analysis has not been introduced in the Indian Standard code yet. Thus the procedure described in FEMA 356 is adapted to accommodate seismic parameters of IS:1893-2016.

<u>Capacity Curve for Capacity Spectrum Method</u>-

The curve is plotted acc to ADRS format And ATC -40 value are altered acc to IS 1893:2016, and comparison of demand spectrum of Ca, Cv are found. In this test we have taken Ca value as 0.18 and Cv values as 0.245 for medium soil and hysteresis bridge behaviors as type B

Typical pushover curve plotted for bridge model NWBR S30M by CSM method is shown in fig 4.6.

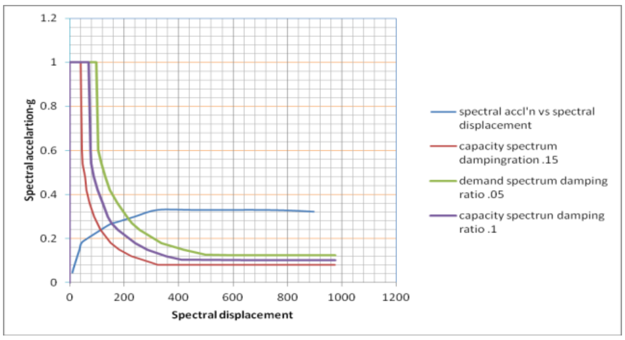


Fig. 4.6: Capacity curve of the bridge NWBR S30M by CSM

<u>Capacity Curve for Equivalent Linearization Method</u>- Improvement of capacity spectrum method. In this method effective damping and time period is found out by using SAP Acc to FEMA 440 And curve is plotted for bridge NWBR s30 by ELM. as in fig 4.7.

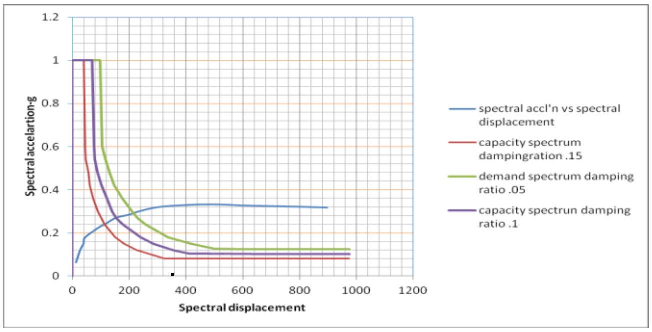


Fig.4.7: Capacity curve of the bridge NWBR S30M by ELM

Capacity Curve for Displacement Modification Method- improvement of displacement coefficient method (FEMA356). Demand spectrum parameters, site class Ss and Sl are same as DCM method. Effect of SSI are included in the analysis. The coefficients C1 and C2 are calculated by new simplified expressions Typical pushover curve plotted for bridge model NWBR S30M by DMM method is shown in fig 4

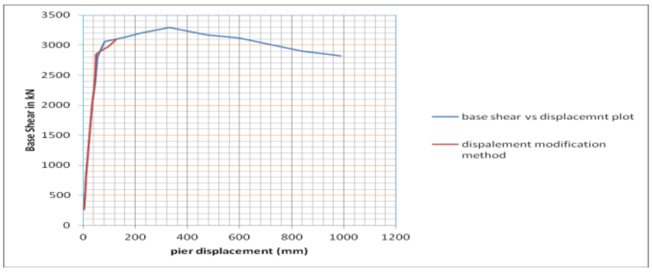


Fig. 4.8: Capacity curve of the bridge NWBR S30M by DMM

Target Displacements and Performance Point-Target displacements and base shear are calculated for four different pushover analysis methods at performance point

Table 4.3 shows bridge model NWBR S 30 M base shear value and target displacement values. Acc to FEMA 440 And 400.

Base shear is same while DCM overestimates the shear demand slightly; the deviation is small enough to be neglected. the values between shear and target displacement of CSM and DCM is reduced in ELM and DMM method.

Table-4.2 Target displacements for PA Methods for model NWBR S30M

PA method	Performance Point	
	Base Shear	Target Displacement
CSM	3043kN	61mm
DCM	3210kN	67mm
ELM	3142kN	64mm
DMM	3009kN	60mm

Series 1 – performance of base shear for 5 m pier is highest which decreases suddenly as height increases.

NWBR H5M values of base shear is high as compared to other bridge.

If height is low stiffness is very high and thus base shear is very high with low displacement.

Table 4.3: Base Shear and Displacement for Series1 (varying height models)

Bridge Model	Base Shear(in kN)				Pier top displacement(in mm)			
	PP	IO	LS	CP	PP	IO	LS	CP
NWBR H5M	4715	6152	10654	10706	3.26	14.4	56	95
NWBR H10M	2400	2198	2127	2300	52	35	97	156
NWBR H15M	2009	1795	1836	1952	60	58	118	228
NWBRH20M	2422	2271	2291	2745	50	82	187	251
NWBR H25M	2040	1608	1839	2136	83	73	266	297

Table 4.4: Base Shear and Displacement for Series2 (varying span models)

Bridge Model	Base Shear(in kN)				Pier top displacement(in mm)			
	PP	IO	LS	CP	PP	IO	LS	CP
NWBR S20M	1894	1734	2105	2289	56	49	177	237
NWBR S30M	3210	3048	3151	3256	67	92	191	290
NWBR S40M	3743	3703	4097	4237	79	75	211	312
NWBR S50M	3721	3386	3737	3956	90	82	200	290
NWBR S60M	2914	2735	2862	3027	104	90	210	297

Performance point of bridges lies in IO and LS except last two which has large displacement at LS and CP and displacement is small for first bridge then increases in other.

Series 2 is different from series 1 where for smallest span base shear is lowest and gets on increasing s span also increases but is less in last bridge.

Demand Comparison with Indian Standard Code-The review of the Indian code provisions for RC pier design in light of the international seismic design practices, and importance of employing the performance based design concept in bridge design necessitates. To analyse any bridge model iS code 1893:2016 part 1 is used along with IRC-6:2016, and 112-2011. And the result are compared in NSP and IS code method.

The comparison is based on total base shear demand of bridge and max shear demand of critical pier as shown in table 4.5, fig 4.9 and fig 4.10. In linear static method shear demand is multiplied by 1.5 FOS for coda demand. Base shear comparison of bridge tells us that pushover demand curve is high in IS code provision for all models. Bp/Bi ratio for model demand difference is calculated. Where smallest pier has highest difference 3.03 while largest has smallest 1.28 .

Bridge Model	Base Shear(in kN)for bridge			Max shear demand for critical pier		
	IS Code(Bi)	NSP(Bp)	Ratio Bp/Bi	IS Code (Vi)	NSP(Vp)	Ratio Vp/Vi
NWBR S20M	982	1894	1.93	225	461	2.05
NWBR S30M	1446	3210	2.22	333	712	2.14
NWBR S40M	1718	3743	2.18	407	866	2.13
NWBR S50M	1440	3721	2.58	339	897	2.65
NWBR S60M	2276	2914	1.28	548	724	1.32
NWBR H5M	1557	4715	3.03	362	1138	3.15
NWBR H10M	1122	2400	2.14	264	595	2.25
NWBR H15M	1119	2009	1.80	263	505	1.92
NWBRH20M	842	2422	2.88	195	558	2.87
NWBR H25M	963	2040	2.12	217	484	2.23

5. CONCLUSIONS

The ends of bridges are restrained so that its dynamic characteristics differ from buildings.By analyzing the structure using Displacement Coefficient Method (FEMA 356), Capacity Spectrum Method (ATC 40), Displacement Modification Method (FEMA 440) and Equivalent Linearization Method (FEMA 440) it was concluded that:

i. High base shear in transverse direction is obtained

ii. Modal mass of first mode is 54.4% and of other is 96.4%

iii. CSM and DCM values are reduced over ELM And DMM

iv. Pushover curve as compared to codal are high so instead of nonlinear static analysis , static analysis is preferred.

v. The Target deflection in transverse direction for longest span bridge is more than 100 mm and highest bridge is more than 80 mm.

vi. Performance point lies between Immediate Occupancy and Life Safety level of performance.

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