

Pushover Analysis of Reinforced Concrete Bridge Pier Designed as Per IRC-6 Codal Provision

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Abstract

The seismic evaluation for the damage caused by ground motion to existing bridges has attracted focus of structural engineers in recent years. It is the first step towards curbing loss of life and property. Most of the reinforced concrete bridges in India were designed as per previous building codes. Those codes seldom accounted for large seismic motions and were insufficient to sustain the seismic loads acting laterally. It is necessary to evaluate damages caused to already constructed bridges. In this paper nonlinear static (pushover) method is focused for performing seismic analysis of RCC Bridge. It is conceptually easier to understand and model and requires low time for computation. Major advancement in pushover analysis procedures is seen in last 10 years and it has led to its introduction to international codes/guidelines for seismic analysis. The pier are subjected to dead load, live load and seismic loading and designed as per IRC-6 2012. The study aimed to determine the seismic performance of the typical reinforced concrete bridge pier designed as per Indian codes with displacement based pushover analysis approach.

Keywords: Bridges • Seismic • Concrete • Plastic hinges

Introduction

Majority of the Indian bridges were inadequately designed to resist seismic forces as per outdated building codes. The design shear capacities for short piers (having aspect ratio between 2 to 3) is found to be smaller than the corresponding shear demand under condition of flexural over strength. The lower transverse reinforcement as per previous codes resulted in lower displacement ductility and weaker post yield response. When seismic loading is applied to redundant RCC structure, as the members' moment capacities are reached, discontinuities develop in structure. Plastic hinges develops and response of structure becomes inelastic. Due to smaller transverse reinforcement in the plastic hinge region at the ends of the piers, the longitudinal reinforcement lacks in developing required strength which results in spalling of the concrete, de-bonding and initiate slippage. Ultimately the pier base experiences either a brittle pull-out failure, or a brittle shear failure.

The bridge structure, in general, lacks in structural redundancy and hence suffers severe damage which leads to failure during ground motion. This paper conducts investigation at determining the adequacy of Strength of the reinforced concrete bridge designed as per the current seismic provisions of the Indian codes for bridge design, namely the IRC: 6-2011, IRC: 21-2010 and IRC: 78-2012. In this paper multi span RCC highway bridges with simply supported at ends are modelled and analyzed using IRC Class AA loading and structural response parameters such as Bending Moment, shear and deflection are obtained to obtain the serviceability. Further, pushover analysis if the bridge structure is performed on structural analysis software SAP 2000.

Literature Review

Pushover analysis methods

In this method of analysis direct lateral loads based on specific load

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patterns are applied on the structure, lateral load is monotonically increased until the structure reaches specific level of displacement. Failure patterns and the possible weak points and of a structure are identified. The status of plastic hinges, formed is used as gauge to evaluated performance of the structure at performance point or target displacement corresponding to specified ground motion (the particular response spectrum). The seismic performance of structure is satisfactory if the seismic demand is less than capacity at all plastic hinges. As the evaluation procedures and lateral loading are empirical with respect to the actual seismic events, it is different from the rigorous dynamic analysis (time history analysis) in many ways.

All the pushover procedures available in literature for structural evaluation are different but the basic principles are the same for all and the bilinear approximation of the pushover curve is used by all of them. The non-linear static procedure converts the properties of Multi degree of freedom (MDOF) structures to corresponding Single degree of freedom (SDOF) equivalents, and using various approximations.

Capacity Spectrum Method (CSM) of ATC 40 (1996)

In this method the nonlinear system is equivalently linearised into a linear system. Most important and basic assumption here is that the maximum inelastic deformation for a nonlinear SDOF system can be approximated from the maximum deformation of a linear elastic SDOF system with an equivalent period and damping. This procedure uses the estimates of ductility to calculate effective period and damping. Three procedures (A, B and C) are described in ATC 40 for the CSM and B is used in the study [1].

Displacement Coefficient Method (DCM) of FEMA 356 (2000)

In this method the elastic displacement of an equivalent SDOF system is estimated assuming initial linear properties and damping for the ground motion excitation under consideration. Then the total maximum inelastic displacement response of the structure is estimated by multiplying with a set of displacement coefficients. These coefficients are based on empirical equations derived using a large number of dynamic analyses for calibration [2].

Equivalent Linearization Method ELM of FEMA 440 (2005)

This method is modified version of capacity spectrum method in which the basic assumption is same as CSM but for equivalent stiffness and damping properties are obtained from large number of response of seismic analysis for different earthquake. Modified equations for calculating effective time period and effective damping are provided in FEMA 440. These are empirical

equations derived from data of statistical analysis of large no of seismic studies with varying earthquake intensities and structural properties [3].

Displacement Modification Method of FEMA 440 (2005)

This method is an improvement over displacement coefficient method. In this method the general equation for calculation of max deflection at performance point is same but the set of coefficients are obtained from completely different equations. The definitions of coefficients are suitably modified and new equations are derived so as to minimize the errors in the estimation of peak responses. The details of modifications with concerned equations are provided in FEMA 440.

Structural modeling

The structural modeling of multi-spanned simply supported bridges is done on structural analysis software SAP 2000. 3D frame elements are utilized for modeling piers, pier cap and simply supported girder. The pier and girder joints are modelled using end-offsets in the frame elements, to evaluate the forces and bending moments at the beam and column faces. The moment is released at the girder ends to make girder-cap joint as a pin joint. The bridge deck is not modeled physically. The foundation and pier bottom joint is considered as fixed. Plastic hinges are applied at both end of pier to introduce non-linear behaviour in structure.

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.

Series I-fixed span bridges

The bridge considered consists of two spans each of 30m. The bridge deck is placed over simply supported concrete girders. Pier caps provided the bearing to RCC girders locked in the transverse direction. The height of supporting piers is equal for same bridge and is varied to obtain the desired series. Bridge model NWBR H5M, NWBR H10M. NWBR H15M, NWBR H20M

& NWBR H25M with pier heights of 5 m, 10 m, 15 m, 20 m and 25 m are adopted for the study. The width of the bridge is 10.5 m.

Series II- fixed pier height bridges

The bridge considered consists of two spans of same length. The bridge deck is placed over simply supported concrete girders. Pier caps provided the bearing to RCC girders locked in the transverse direction. The height of supporting piers is 15m 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 20 m, 30 m, 40 m, 50 m and 60 m are adopted for the study. The width of the bridge is 10.5 m [4].

The modelled bridges have two 2 lanes and the reinforced concrete bridge has total width of 10.5 m. Class AA loading as per IRC is used as vehicle live load per lane. M40 grade of concrete and Fe 500 grade steel is adopted. To accommodate for ductility and strength enhancement due to enhanced confinement, the stress-strain curve adopted for analysis is modified Mander's model as shown in Figure 1. As for steel, the stress-strain curve provided in IS: 456 is used in the study as shown in Figure 1.

For the application of pushover analysis, the nonlinear behaviour must be accommodated in the structural model. In this work, nonlinearity is modeled by incorporating a point-plasticity approach in which the plastic hinge is considered to be present at a particular point in the frame elements. Plastic hinges are assumed at an offset of 0.05 L from both ends Figures 2 and 3.

Behaviour of plastic hinges and its properties must replicate the actual response of reinforced concrete components subjected to lateral load. For practical purpose, the default hinges properties documented in the FEMA-356 and ATC-40 documents are preferred due to convenience and simplicity. For modeling the hinge properties, Moment-rotation parameters are the actual input and these can be obtained from the curvature-moment relation. The idealized moment-rotation curve is shown in Figure 4.

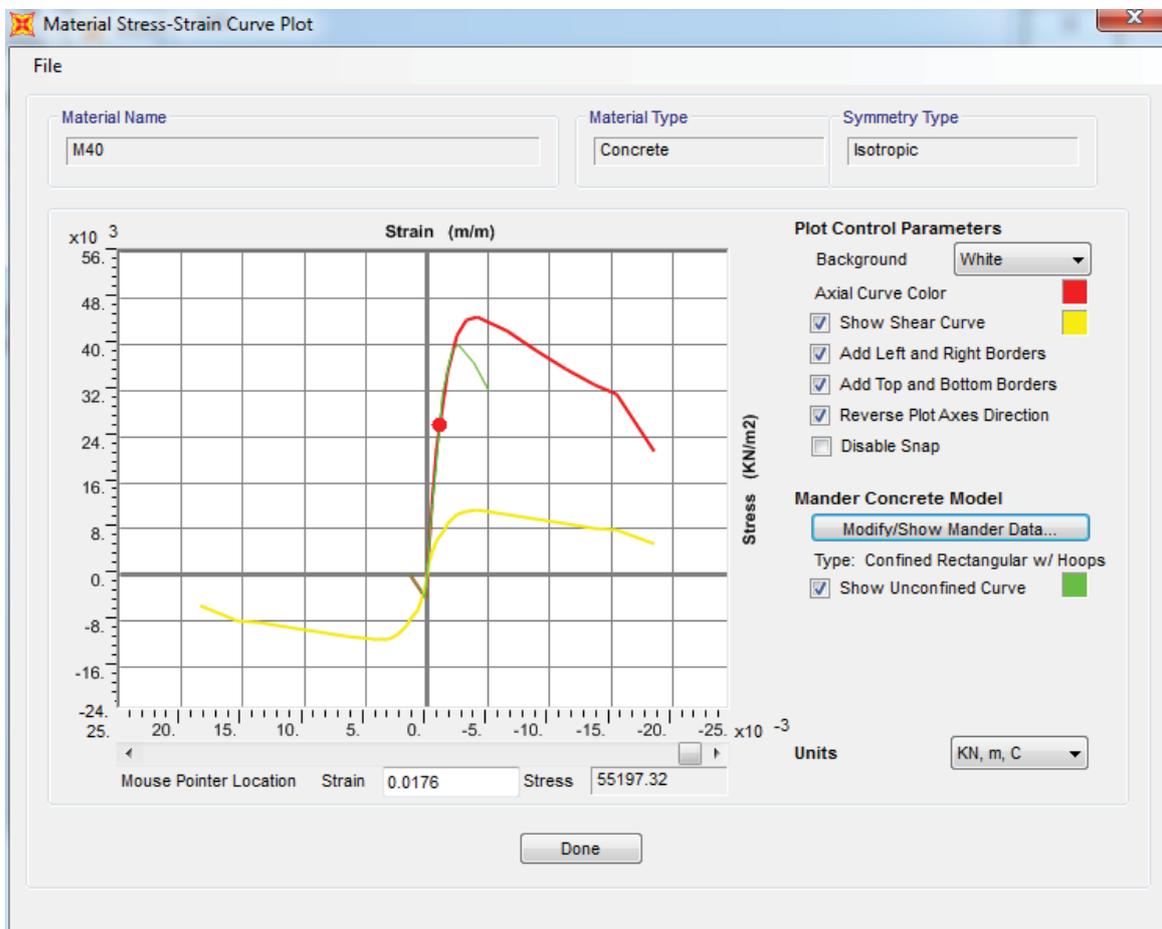


Figure 1. Stress-strain characteristics plot for M-40 grade of concrete as per Modified Mander's model.

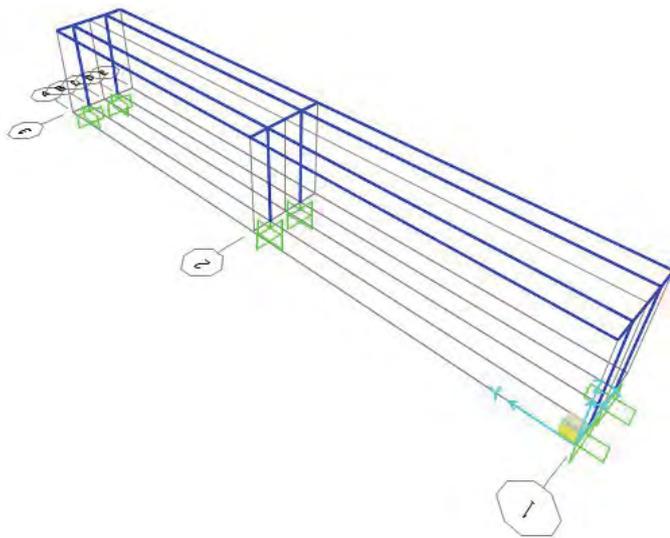


Figure 2. 3D model of bridge.

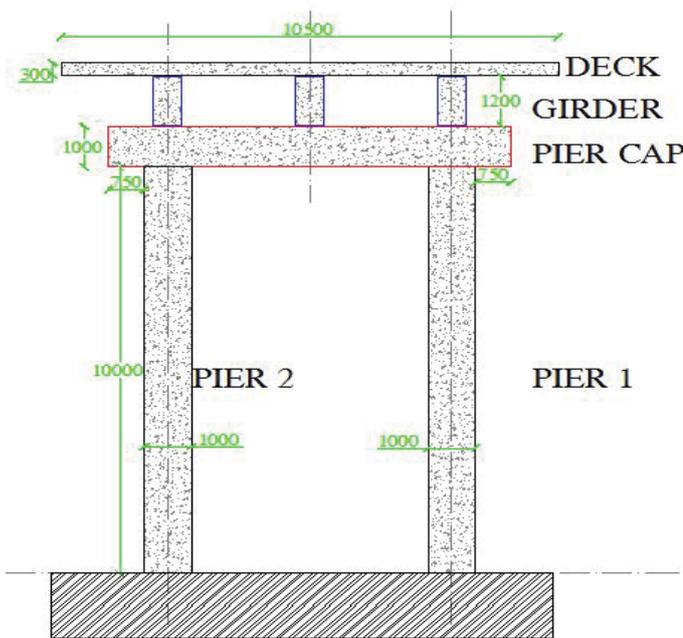


Figure 3. Typical cross-section of bridge.

Seismic analysis of bridge

Pushover analysis is performed first in a load control manner than in a displacement control manner. Initially all gravity loads are applied on to the structure (gravity push). Then a lateral pushover analysis in transverse direction was performed which starts at the end of gravity push. It is established in the various literature reviews that load pattern based on inertial mass at different node i.e., load pattern 1 give conservative results and closest to the full-fledged time history analysis, hence capacity curves for various bridges with load pattern 1 are further discussed [5]. The pushover demands obtained from these analyses are monitored against the design seismic demand corresponds to the Zone V (PGA = 0.36 g) of India as per the current bridge design codes (IRC: 112-2011 & IRC: 6-2016).

Capacity curve for displacement coefficient method

Basics of the method are already discussed above. 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. In defining FEMA general response spectrum site class is taken as D which corresponds to medium stiff soil site as per Indian code. The values

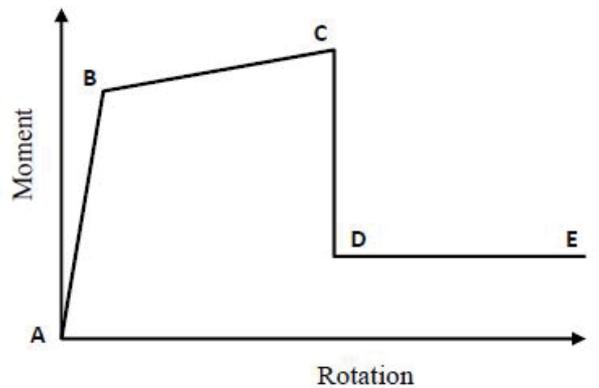
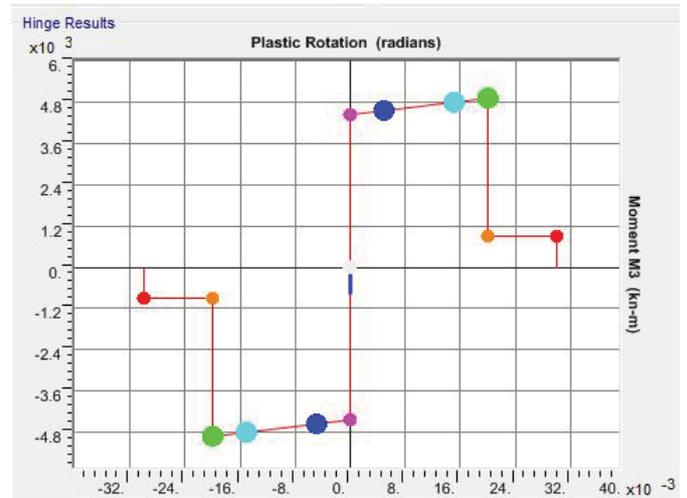


Figure 4. Moment-rotation curve idealized for RCC elements.

of S_s and S_l (spectral acceleration at short and long periods) is calculated as 2.5 g and 1.36 g from response spectra for medium stiff soil in Indian code. The values of coefficients C_0 , C_1 , C_2 and C_3 are calculated by the software. Typical pushover curve plotted for bridge model NWBR S30M by DCM method is shown in Figure 5.

Capacity curve for capacity spectrum method

The pushover curve for this method is plotted in ADRS format, details for which are discussed in former chapters. Similar to previous method, the seismic parameter of ATC-40 are modified to incorporate Indian code for seismic analysis IS: 1893-2016. Coefficient of ATC-40 demand spectrum C_v and C_a are determined by comparing the response spectra curves for ATC-40 and IS code. The values of C_a and C_v are taken as 0.245 and 0.18 respectively for medium stiff soil. As per ATC-40 recommendation for RCC structures, the hysteresis behaviour of bridge is provided as type B. Typical pushover curve plotted for bridge model NWBR S30M by CSM method is shown in Figure 6.

Capacity curve for equivalent linearization method

This method is an improvement over Capacity Spectrum Method (ATC-340). Demand spectrum parameters are same as CSM method. Soil structure iteration effects are included in the analysis. This method aims at better prediction of effective time period and effective damping at each iteration step, thus minimizing error in predicting performance point for the pushover analysis. T_{eff} and B_{eff} are obtained by SAP using simplified expressions provided in FEMA440. Typical pushover curve plotted for bridge model NWBR S30M by ELM method is shown in Figure 7. Also showing the values of S_a , S_d , T_{eff} , B_{eff} , ductility ratio along with base shear and pier top displacement at performance point.

Capacity curve for displacement modification method

This method is an improvement over displacement coefficient method (FEMA356). Demand spectrum parameters, site class S_s and S_l are same

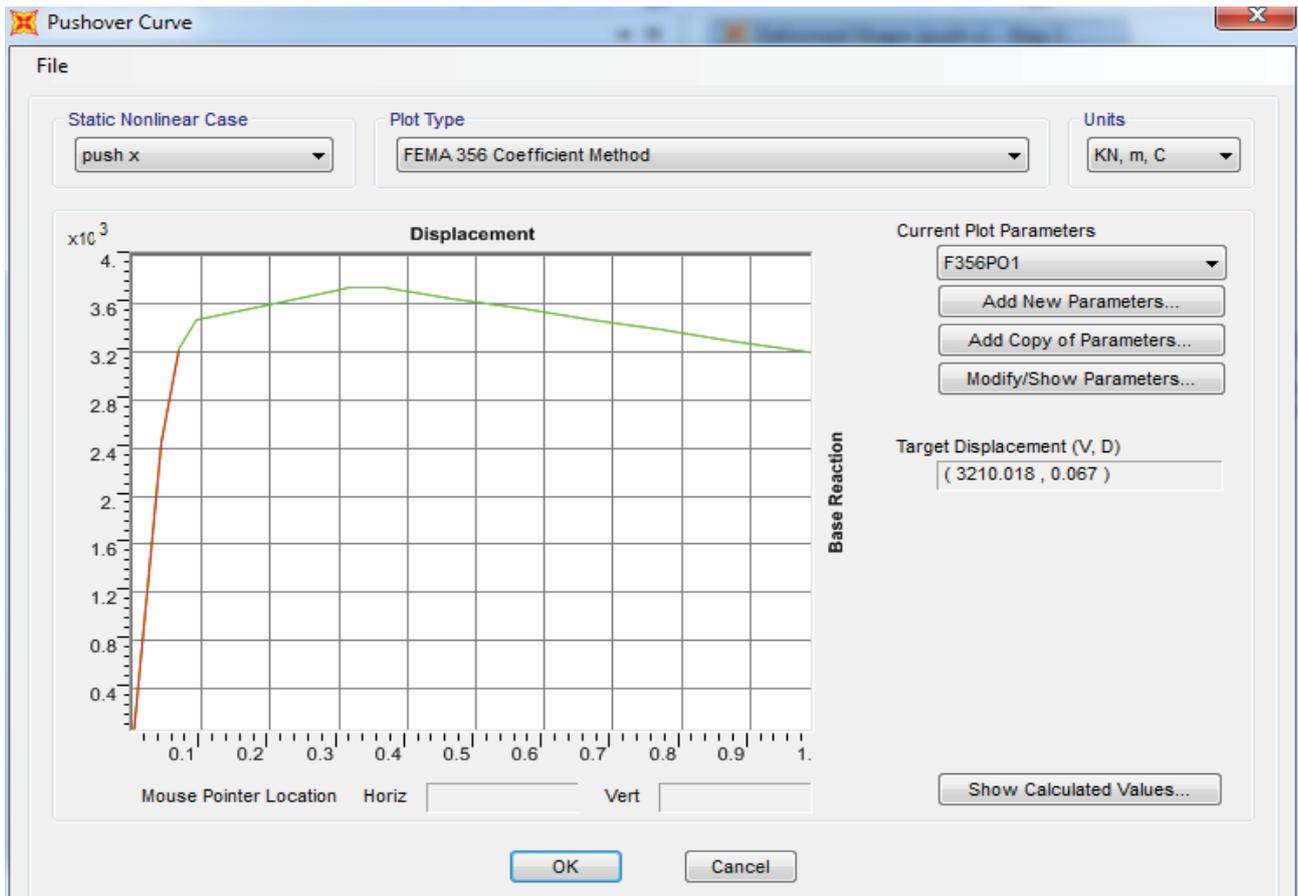


Figure 5. Capacity curve of the bridge NWBR S30M by DCM.

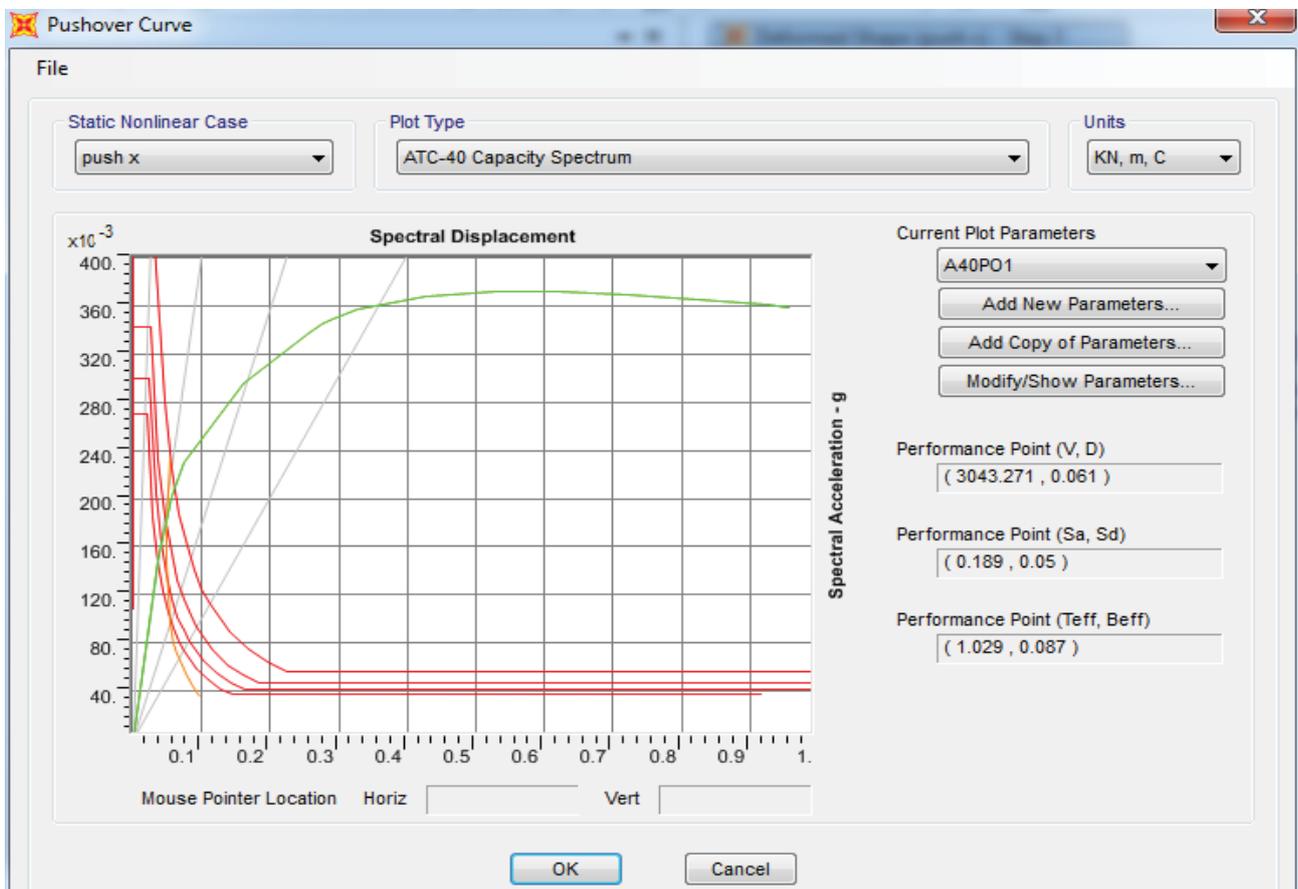


Figure 6. Capacity curve of the bridge NWBR S30M by CSM.

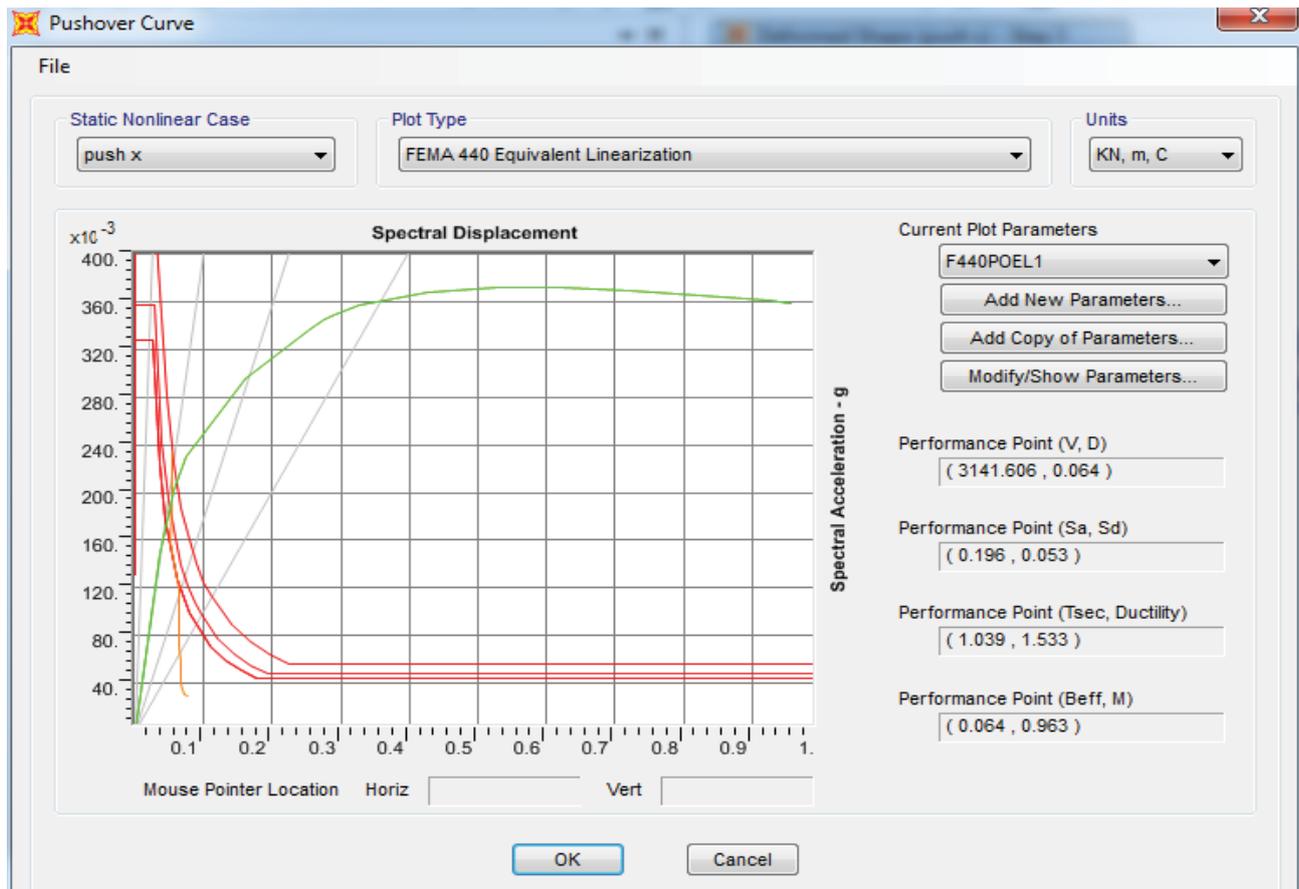


Figure 7. Capacity curve of the bridge NWBR S30M by ELM.

as DCM method. Soil structure iteration effects are included in the analysis. The coefficients C1 and C2 are calculated by new simplified expressions as discussed in the literature review. Typical pushover curve plotted for bridge model NWBR S30M by DMM method is shown in Figure 8 [6].

Results and Discussion

Target displacements and performance point

Target displacements and base shear are calculated for four different pushover analysis methods at performance point as per the procedures. Table 1 presents the base shear and target displacement values for bridge model. NWBR S30M calculated as per FEMA 356 displacement coefficient methods, capacity spectrum method (ATC 40), displacement modification method (FEMA 440) and equivalent linearization method (FEMA 440). These results are compared with Equivalent Static Method (ESM) as per IS Code [7].

It is seen that base shear from all the methods is in similar range. DCM overestimates the shear demand slightly, but the deviation is small enough to be neglected. It is also noticeable that the differences in values of base shear and target displacement between the two basic methods (i.e., CSM and DCM) are reduced when obtained with their improved modification method (i.e., ELM and DMM).

Comparison of NSP with ESM shows that NSP demand is greater than two times the ESM demand for all the cases. Similar trends were seen in the results of the other bridge models also, that are discussed below.

Base shear and pier top displacement at performance point and the three performance levels, namely immediate occupancy (IO), life safety (LS) and collapse prevention (CO), for the two series of bridge models (series 1 varying pier height and series 2 varying span) are provided in Table 2 and Table 3 respectively.

In case of series 1 base shear at performance point is greatest for 5 m pier

height and decreases suddenly as the height of pier is increased. Further the values remain similar for last three bridges of the series. Similar trend were also seen for base shear at various performance levels, the values of base shear for NWBR H5M are very high as compared to other bridges. At lower pier height the stiffness of bridge pier is very high and thus develop very high base shear at very low displacement. As for displacement at performance point and other performance levels, it is very small for the first bridge of series and goes on increasing. Last two bridges in series showing large displacements particularly at levels of LS and CP. Except for the first case, the performance point of all other bridges lies between IO and LS [8].

Base shear as well as displacement trends for series 2 is completely different from series 1. Base shear for the smallest span is lowest, increases with increase in span but shows decrement for last bridge. This trend is same for considered parameters (PP, IO, LS and CO). Displacement variations are similar at performance point with lowest values for smallest span and increases with increase in span of bridge. This trend is not true for displacement at other performance levels, showing random trends with increase in span. As expected the displacement values for LS and CP are on the higher side.

Pushover demand comparison with indian standard code

The inquiry of the Indian codal provisions for design of RC pier considering the international seismic design practices, and significance of implementing the performance based design approach in bridge design demands the comparison of performance based demand (NSP analysis) for piers with design demand as per the existing Indian standards. To facilitate the same the seismic analysis of the two series of model bridges is also performed with the approach stipulated by Indian Codes. The codes used for the analysis of bridges are IRC: 6-2016 (latest edition), IRC: 112-2011 (last edition) and IS1893-2016 Part I.

The results obtained from seismic analysis of bridges with two different approaches, i.e., Nonlinear Static Analysis and Indian Code base Linear Static analysis, are compared. The comparison is based on total base shear demand of bridge and max shear demand of critical pier as shown in Table 4 The shear

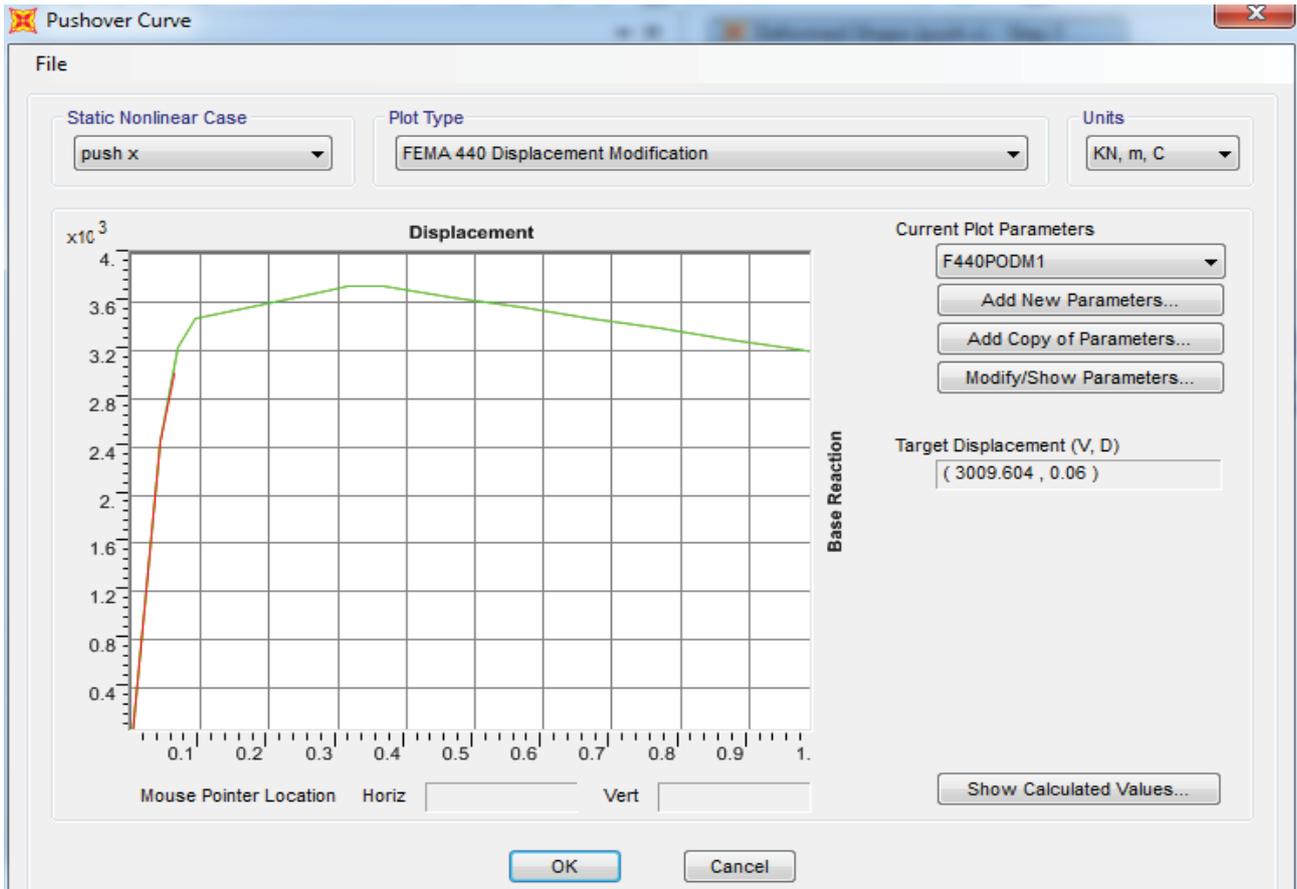


Figure 8. Capacity curve of the bridge NWBR S30M by DMM.

Table 1. Target displacements for PA Methods for model NWBR S30M.

Siesmic analysis method	Performance Point	
	Base Shear	Target Displacement
CSM	3043 kN	61 mm
DCM	3210 kN	67 mm
ELM	3142 kN	64 mm
DMM	3009 kN	60 mm
ESM (IS code)	1446 kN	28 mm

Table 2. Base shear and displacement for series 1 (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 3. Base shear and displacement for series 2 (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

demand values obtained for linear static method are factored 1.5 times to reach codal demand.

The comparison of base shear bridges shows that pushover demand is very high against codal seismic demand for all the model bridges. The

Table 4. Comparison of result of pushover analysis and linear static analysis.

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

difference in the two demands is described by ratio Bp/Bi. Model with smallest pier height NWBR H5M has largest difference with ratio of 3.03 while model NWBR S60M with largest span shows smallest variation having ratio of 1.28. Similar trends are seen in case of max shear demand at critical pier also. The average values of the two ratios Bp/Bi and Vp/Vi for the ten model bridges are 2.21 and 2.27 respectively [9].

Conclusion

Only limited analysis is performed using only few analytical models and the following points can be drawn from this study.

1. For most cases performance point for pushover analysis lies between Immediate Occupancy and Life Safety level of performance. Thus Pushover methodology demands the structure to go beyond linear yielding.
2. The difference between the Pushover demand and Codal demand is very high and thus it is recommended to introduce non-linear static analysis approach in the Indian Codes.
3. The design procedure outlined in IRC codes does not account for the possibility of plastic hinge formation in an extreme seismic event. Non-linearity is completely neglected in seismic analysis.
4. Difference between base shear and target displacement for the two basic methods (i.e., CSM and DCM) are reduced when obtained with their improved modification method (i.e., ELM and DMM).

5. Bridge with small pier height shows very high values of base shear at very small deflection, thus failure of pier occurs before formation of plastic hinges. Further work is required to come up with plausible performance based analysis for smaller pier height bridges.

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