Pushover Analysis of Reinforced Concrete T - Beam Bridge

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Abstract— Bridge is a very important structure because it should be withstand even after a disaster for the emergency transport. So the bridge design should be safe, strong and economical. Pushover analysis is a displacement based technique to analyse the capacity of the existing structure. Indian code explains this nonlinear static procedure in brief manner. So a number of assumptions are made by designers to do pushover analysis using Indian code. This may lead high risk in analysis results obtained. FEMA 356 hinge actually meant for steel columns are often used in concrete columns by designers because of its default values. But from analysis results obtained it is found that this will show over strength to hinges than actual. Caltrans hinge is preferred in case of concrete columns. In that case hinge length need to calculate so Indian code should add additional clauses on hinges. Need clarification in hinge number at piers, integral pier-superstructure bridge need additional hinge assignment at top of pier, this is not specified in Indian code also lead to assume over strength of structure. Detail study needed on above clauses and code must need revision on that basis. A reinforced concrete T beam bridge, modelled in SAP2000 used for the study.

Index Terms — Displacement based analysis, Pushover analysis, Target displacement, Plastic hinge, Caltrans hinge, hinge length, RC bridges, SAP Bridge.

1 Introduction

EARTHQUAKES are one of the worst among the natural disasters. These lead to the loss of life, property damage and socio-economic disruption. About 1 lakh earthquakes of magnitude more than three hit the earth every year. India has witnessed more than 650 earthquakes of Magnitude greater than 5 during the last hundred years. The main reason for this is the driving of Indian plate into Asian plate at 47 mm/year.

In an engineering point of view, like any other natural disaster, it is possible to minimize the effect of earthquake in structures by proper planning and design. Bridge is a very important structure because it should withstand even after a disaster for the emergency transport. So the bridge design should be safe, strong and economical. Bridge movements are difficult to predict. There are six potential degrees of freedom (DOF) of movement for a bridge structure. Seismic forces and wind forces impart movements in bridge components and at bridge joints. The movements at joints are generally due to the daily and annual expansion and contraction due to temperature changes, elastic shortening due to post tensioning, creep, and shrinkage, movement due to live load fluctuations etc. However, live load movements are usually negligible.

A properly engineered structure does not necessarily have to be extremely strong or expensive. It has to be properly designed to withstand the seismic effects while sustaining an acceptable level of damage. International studies concentrate on Displacement based analysis instead of Force based analysis to achieve this goal efficiently. Here the performance of a structure is expressed in terms of displacement instead of force. So displacement based design gives structural capacity more preciously than force based design. Indian codes available are weak in handling the seismic effect on structures especially on bridges. The previous available code for earthquake Resistant Design of Bridges is given in a single clause (clause 6) of IS: 1893-1984. Recently BIS is released a separate code for the same IS 1893 (part 3). But the latest code is also focussed on force based. IS 1893 (part 3) explains the nonlinear analysis methods using single paragraphs and it is vague, so structural designers cannot do a displacement based analysis with the given IS provision. Therefore, some assumptions are made by the designers and it will affect the analysis result, design and finally the structural safety.

1.1 Objective

Study the advantages of displacement based analysis over the force based analysis; the one followed by Indian code. To get a better idea of using pushover analysis, a displacement based technique suggested in Indian code, and to find out the limitation of Indian code provisions in using this analysis tools.

The scope of this project is limited to

- Compare the Indian code provisions with AASHTO code provisions and practices to find the limitation of IS code in Pushover analysis using a theoretical study.
- Carry out a case study of pushover analysis of a reinforced concrete T-Beam Bridge modelled in SAP2000.
- Study the effect of plastic hinge properties in Pushover analysis to find the code limitations in Pushover anlaysis.

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2 LITERATURE REVIEW

From the literature study it is observed that only limited studies are available on displacement based analysis of bridges using IS codes. Displacement based analysis is a performance based analysis gives capacity of structure more accurately than force based analysis. It ensures the design of a properly engineered structure and it does not have to be extremely strong or expensive. That structure has to be properly designed to withstand the seismic effects while sustaining an acceptable level of damage.

Pushover analysis is a displacement based analysis tool (nonlinear static analysis), but the IS code does not explain about the procedure in a satisfactory manner. So the structure, especially bridges need to study on basis of any other country's code. Wide assumptions are needed in such cases and it will affect the result. Plastic hinge type and its position in a bridge pier control the analysis very high manner. But proper guidelines are not available in IS code. Several study use FEMA 356 hinges instead of Caltrans in concrete bridges. In AASHTO code they mentioned FEMA 356 for steel column and Caltrans hinge for concrete column.

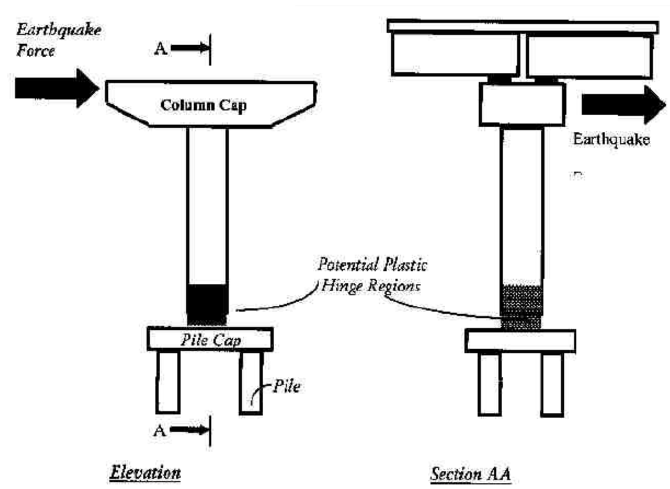
2.1 Pushover Analysis

Pushover analysis is a static nonlinear procedure in which the magnitude of the lateral load is increased monotonically maintaining a predefined distribution pattern along the height of the structure. Structure is displaced till the 'control node' reaches 'target displacement' or structural collapses. The sequence of cracking, plastic hinging and failure of the structural components throughout the procedure is observed. The relation between base shear and control node displacement is plotted for all the pushover analysis. Generation of base shear – control node displacement curve is single most important part of pushover analysis. This curve is conventionally called as pushover curve or capacity curve. 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.

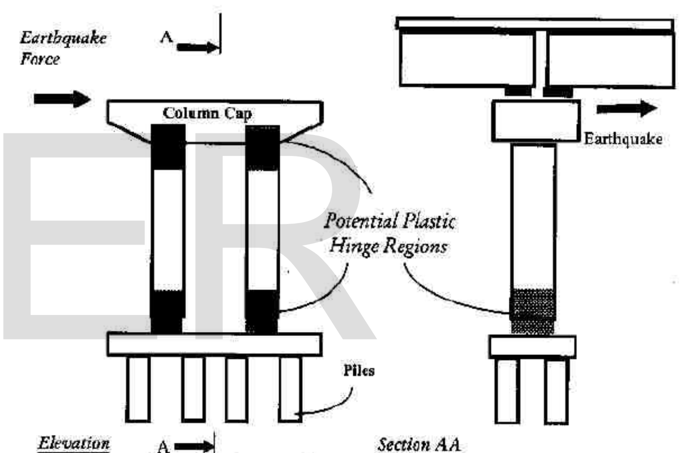
Plastic hinge formation

In the implementation of pushover analysis, the model must account for the nonlinear behaviour of the structural elements. A point-plasticity approach is considered for modelling nonlinearity, wherein the plastic hinge is assumed to be concentrated at a specific point in the frame member under consideration. In bridge structure hinges are assumed to be form at column/pier instead of weak beam strong column concept, i.e. Hinge at pier should fail before superstructure or foundation failure. Piers are modelled with flexure (P-M2-M3) hinges at possible plastic regions under lateral load (i.e., both ends of the beams and columns). Properties of flexure hinges must simulate the actual response of reinforced concrete components subjected to lateral load.

IS 1893(part 3) include some suggestion in plastic hinge assignment at appendix A. The clause says that in case of bridges the plastic hinge formation should be at piers instead of superstructure or substructure. In case of single pier, the hinge is at bottom near pier foundation only. In multiple piers, the hinge should consider at both top and bottom of pier (refer fig.2.1 and 2.2, provided in IS code).



(a) Single column or pier type substructures



(b) multi-column or frame type substructures
Fig. A-1: Potential location of plastic hinges in substructures (Clause A-0).

Figure 2.1: Potential hinge at Single column substructure [Source: IS 1893 (part 3):2014 figure A-1(a)]
Figure 2.2: Potential hinge at Multi- column substructure [Source: IS 1893 (part 3):2014 figure A-1(b)]

In **AASHTO code** gives large details about the plastic hinge and an equation to calculate the plastic hinge length, which is necessary in analysis procedure.

$L_p = 0.08L+0.022F_{ye}d_{bl} \geq 0.044\, F_{ye}d_{bl}$ (mm, MPa)

[Source: Clause 7.6.2.1 of Caltrans Seismic design criteria version 1.7:2013]

Where L = length of column from point of maximum moment to the point of moment contra-flexure,

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

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

The hinge length is compared to the value for the minimum hinge length, described as $L_p=0.3\, f_{ye}\, d_{bl}$, and the larger value

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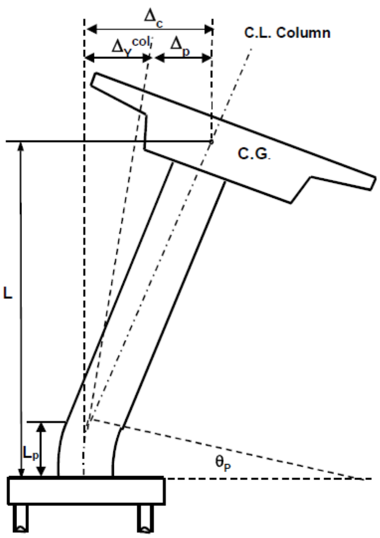
is used.

The literal studies show that the concept of hinge at bottom of pier only is not correct if the bridge deck and column connection is integral. In that case the bridge is considered as a frame structure so the hinges should be considered at both top and bottom of column (fig 2.3 and fig.2.4, reference images from Caltrans Seismic design criteria April 2013).



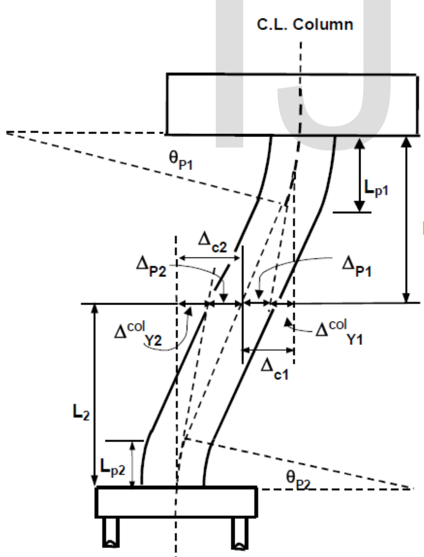


Figure 2.3: Local Displacement Capacity – Cantilever Column with Fixed Base
[Source: Figure 3.1.3-1 of Caltrans Seismic design criteria version 1.7:2013]

Figure 2.4: Local Displacement Capacity – Framed Column, assumed as Fixed-Fixed
[Source: Figure 3.1.3-2 of Caltrans Seismic design criteria version 1.7:2013]

For the steel columns, based on the Section 4.11.8 in AASHTO Seismic Guide Specification, the plastic hinge region is determined as the maximum of 1/8 of the clear height of a steel column or 1.5 times the gross cross-sectional dimension in the direction of bending. Calculated hinge lengths may be different for bending in the longitudinal or transverse direction of the bents. However, the hinges are placed on the bent columns at each end of the column at distances from each end equal to 1/2 the hinge length, as shown below Figure 2.5 for numerical analysis.

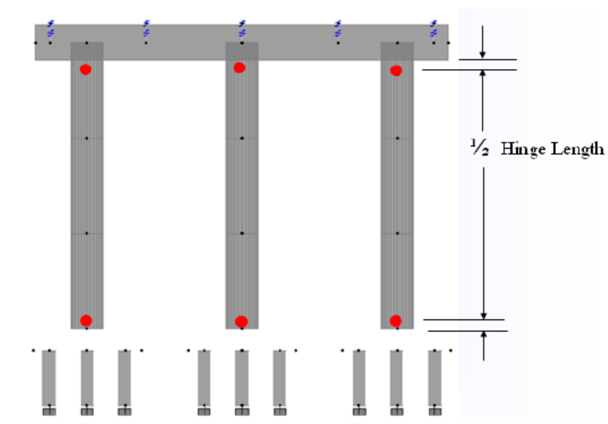
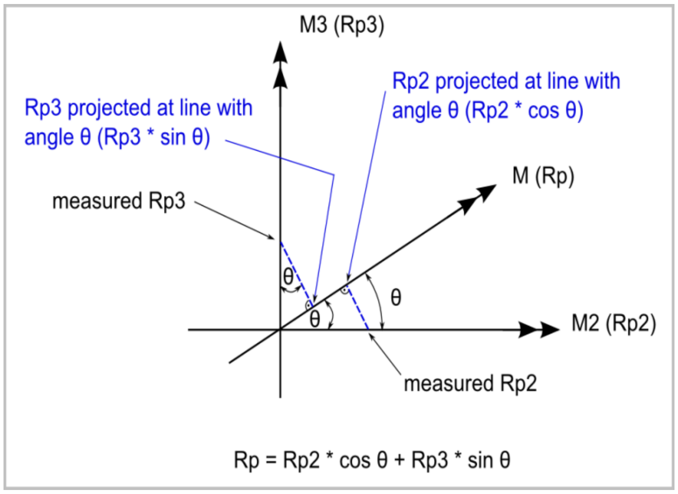


Figure 2.5: Hinge at both end fixed pier in numerical analysis model
[Source: CSI Bridge 2017- Bridge seismic design]

Moment-rotation curve

The moment-rotation curve of a P-M2-M3 hinge is a monotonic backbone relationship used to describe the post-yield behavior of a beam-column element subjected to combined axial and biaxial-bending conditions. The 3D interaction surface of a P-M2-M3 hinge indicates the envelope of yield points. Performance beyond this limit state must be interpolated from one or more moment-rotation curves. Because P-M2-M3 response extends linearly through 3D coordinates to the yield surface, then beyond in a manner that will not exactly resemble the input moment-rotation curve, an energy-equivalent curve is created by holding the area under the user-defined curve constant. Deformation capacity is reduced or increased to maintain equivalency, based on yield-



point distance from the M2-M3 plane. This energy-equivalent curve then extends from the interaction surface in a nonlinear manner.

P-M2-M3 parameters

A moment-rotation curve is defined by the relationship between a series of resultant moments M and projected plastic rotations R_p. As described in the CSI Analysis Reference Manual (Moment-Rotation Curves, page 137), these coordinates are obtained through an iterative and qualitative experiment in which the element is modeled in SAP2000 and subjected to a constant axial load P and moments M_2 and M_3 which increase according to a fixed ratio ($cos\theta$, $sin\theta$) which corresponds to their moment angle θ, shown in Figure 2.6.

- Resultant moment M is then given as $M = M_2 \cos\theta + M_3 \sin\theta$, and projected plastic rotation R_p is given as $R_p = R_{p2} \cos\theta + R_{p3} \sin\theta$.
- These relationships indicate that the moment and rotation values of a P-M2-M3 moment-rotation curve are obtained through basic geometric relationships between components projected along the M2 and M3 axes, as shown in Figure 2.5

Figure 2.5: Relation between resultant moment and local axis moment

The yield rotation of a hinge is calculated as yielding curvature (My/EI) multiplied by hinge length.

Pushover curve

It is the plot of the lateral force V on a structure, against the lateral deflection d, of the top of the structure. This is often referred to as the 'capacity' curve. Performance point and location of hinges in various stages can be obtained from pushover curves as shown in the fig. The range AB is elastic range, B to IO is the range of immediate occupancy IO to LS is the range of life safety and LS to CP is the range of collapse prevention.

Figure 2.6: Pushover curve

Target displacement

Target displacement is the displacement demand for the structure at the control node subjected to the ground motion under consideration. This is a very important parameter in pushover analysis because the global and component responses (forces and displacement) of the structure at the target displacement are compared with the desired performance limit state to know the structural performance. So the success of a pushover analysis largely depends on the accuracy of target displacement. There are two approaches to calculate target displacement:

(a) Displacement Coefficient Method (DCM) of FEMA 356
(b) Capacity Spectrum Method (CSM) of ATC 40.

Both of these approaches use pushover curve to calculate global displacement demand on the structure from the response of an equivalent single-degree-of-freedom (SDOF) system. The only difference in these two methods is the technique used.

3 Analysis, Results & Discussions

A four-span existing concrete bridge having a total length of 120m is taken for the study. The deck of the bridge is designed as continuous T-beam structure and it is integrated with the piers. Three Piers, round concrete columns of 2 m diameter and 6 m high, equally spaced between the abutments (30m c/c).

Bridge details are provided in the table

1	Number of Span	4
2	Centre to Centre Length of each Span of Bridge	30 m
3	Width of Bridge	9.85m
4	Number of Main Girders	4
5	Slab thickness (average)	0.20m
6	Longitudinal Girder Size	0.6mx1.9m
7	Pier (round column)	2m diameter x6m height
8	Pier cap	7.6m x 2.3m x 1.5m

Assumptions

- Bridge is assumed to be a SDOF system to do pushover analysis.
- All girder and pier sections are assumed to be prismatic. Hinges can't be assigned with non-prismatic sections.
- Hinge location is not specified in Indian code. Hinges are assumed to be located at 5% and 95% of span length at beams (For frame structure weak beam strong column concept is used).
- As per IS 1893 (part 3) in bridge structure plastic hinge should be located at columns not in

superstructure and foundation. In single pier, the hinge should be at bottom pier end. Assumed it is concentrated at 5% of pier height from bottom. International practises hinge is assigned at both bottom and top of pier if the hinge is an integral structure. In that case hinge location is assumed at 5% and 95% of span length at column as initial case.

- Response modification factor is equal to 1. [R=1 for displacement based studies in IS 1893(part 3) and AASHTO LRFD 2012]

Figure 3.1: bridge model under study

3.1 Modal Analysis

For getting the actual site condition in India we do two set of analysis one with Indian code factors and the other with AASHTO site factors, base reaction values are noted. Then the model is analysed using AASHTO site factors. Five iterations are done by changing Ss and S1 value of AASHTO code. This Ss and S1 value gives the zonal factor in AASHTO instead of a direct Z value as in Indian code. As per IS code site is in zone III and corresponding Z=0.16. But bridge is an important structure so Z value is assumed to be 0.36 corresponding to very intense earthquake. The soil property is also matched. Actual site is under soil type II in Indian code having N value in between 10 and 30. Use class D soil of AASHTO code, have N value in between 15 and 50. By did random iterations and comparing the base shear obtained (table 4.1 and table 4.2) it is found out that Ss=1 and Ss=0.01 give best result for the zone factor, Z=0.36.

From the modal analysis with Ss=1 and S1=0.1 the base shear value involved in the analysis is obtained as 14658 KN. Twelve modes are participated in analysis and time period of first mode of structure is found to be 0.458 seconds and fundamental frequency= 2.179 Hz.

The Base shear value used for design as per IS code is given by F= (Z/2)*(I/R)*(Sa/g)*W

Where Z=zonal factor =0.36 in this case

I=importance factor=1.2

R=response reduction factor=1 (for displacement based studies)

Sa/g= average acceleration coefficient=2.5 (corresponding t=0.4s)

W= Weight of structure= 29562.256KN (obtain from analysis)

F= 15963.6 KN (value obtained by manual calculation). The base shear obtain from IS code is slightly greater than that obtained from Modal analysis with Ss=1 and S1=0.1.

3.2 Pushover Analysis

Pushover analysis is run after modal analysis to study the performance of a structure. Here the analysis is done on basis of AASHTO code, but with the soil class and zonal factor obtained from initial analysis (explained in previous session) to maintain same site condition. AASHTO code is preferred because IS code has a number of limitation in doing displacement based analysis.

Assigning of plastic hinge is the most essential step in pushover analysis. Number of hinges in member, selection of hinge type and its location are important in this procedure. In this session importance in number of hinges in a member is discussed with two cases case 1 and case 2.

Case 1: Pushover analysis – hinge at pier bottom only

Case 2: Pushover analysis – hinge at bottom and top of pier

Base reaction

Table 4.1: Base reaction from Pushover analysis-hinge at bottom only

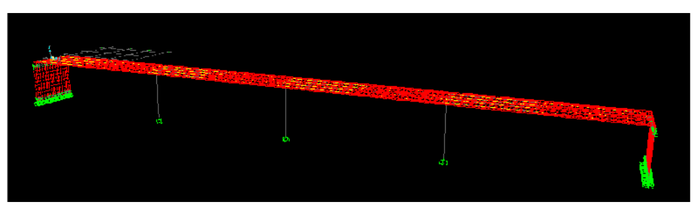
Pushover analysis- hinge at bottom only			
LOAD NAME	Case Type	Max Base shear KN	Displacement m
PUSH X	Non-linear	38763.723	0.026
PUSH Y	Non-linear	16731.16	0.001

Table 4.2: Base reaction from Pushover analysis-hinge at top & bottom of pier

PUSHOVER ANALYSIS- HINGE AT BOTTOM & TOP			
LOAD NAME	Case Type	MaxBase	Displacement
PUSH X	Non-linear	38989.58	0.029
PUSH Y	Non-linear	20624.99	0.002

From the base shear values given in table 4.1 and 4.2, it is clear that the actual structure can withstand very high earthquake than the base shear 15693.6KN calculated as per IS code.

The base shear corresponding to push X in both case



shows almost same result, >38x10^{3}KN. Only 226 KN increase for case 2 than case 1. But this is 60% higher than the base shear calculated as per IS code. Structure showed a top displacement above 26 mm for 38x10^{3}KN base shear. Displacement shows an increase of 3 mm in case 2 than case 1

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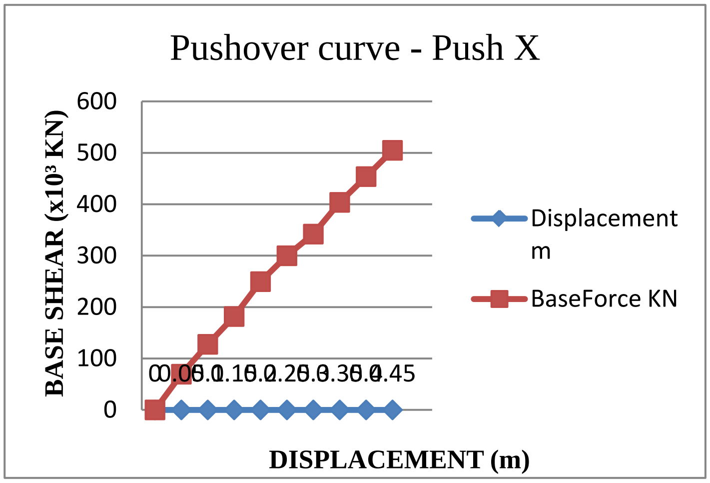
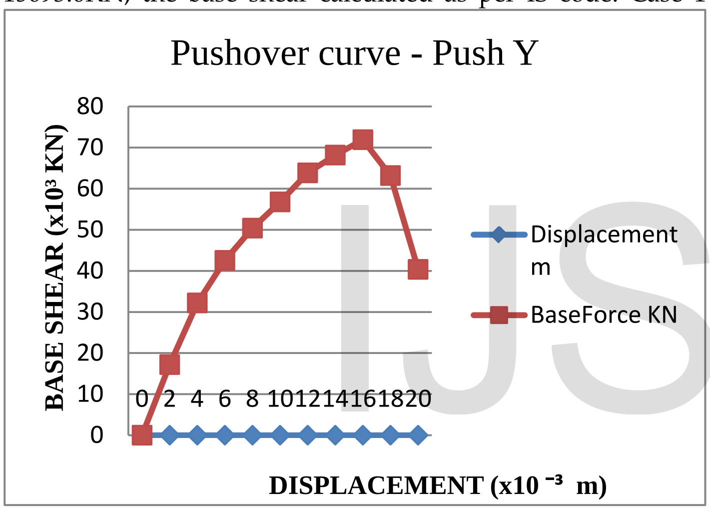
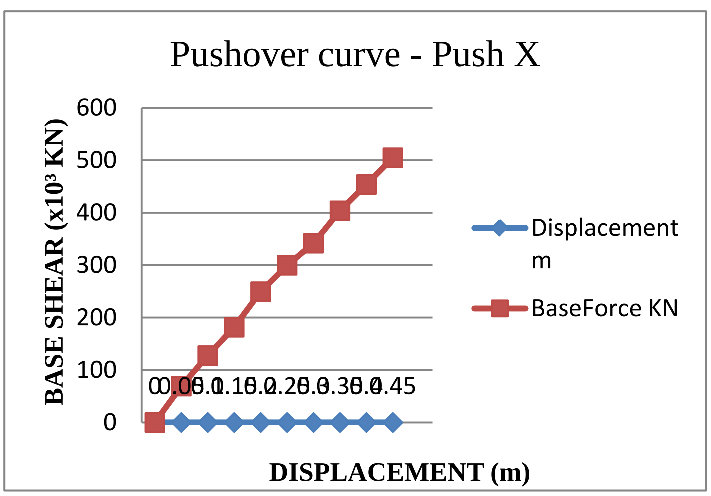


in x direction push due to the small increase in shear. In push Y direction case 2 exhibits an increase of 3893 KN than the value obtained for case 1. Both the base shear values are above 15693.6KN, the base shear calculated as per IS code. Case 1

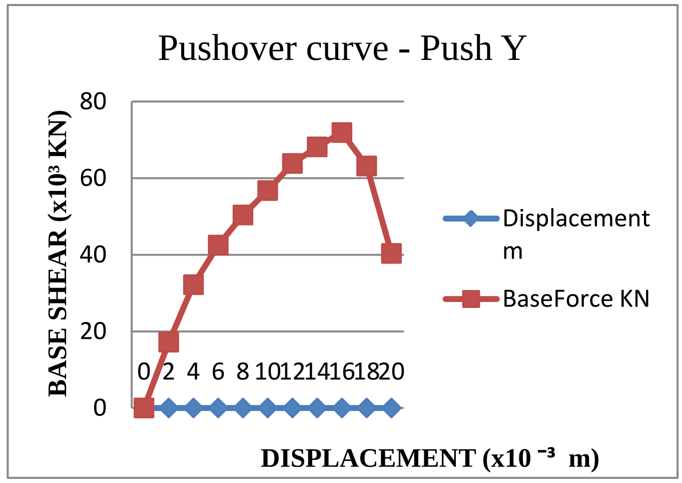


shows nearly 1000KN difference but case 2 shows nearly 5000KN difference with the calculated base shear. That means



an increase of 24%. The displacement showed by the structure is very low 1 mm and 2 mm correspondingly for case 1 and case 2. This is because of high lateral stiffness of structure.

Pushover curve



Pushover analysis is a displacement based analysis and the structure will push up to a target displacement and the force involved is calculated. In this study, a value of 0.41m is assumed as target displacement. Load is applied laterally as incremental values till the displacement is reached. The same displacement is applied in both x and y direction and for the two cases.

Figure 4.1: Pushover curve- push x
(Case: Hinge at bottom only)

Figure 4.2: Pushover curve- push y
(Case: Hinge at bottom only)

Figure 4.3: Pushover curve –push X
(Case: Hinge at top & bottom)

Figure 4.4: Pushover curve –push Y
(Case: Hinge at top & bottom)

From pushover curves, it is clear that both cases (case with one hinge at bottom only and case with one hinge at top and one at bottom) show almost equal performance in pushover analysis. In both the cases force applied in y direction show negligible displacement.

Pushover curve analysis also gives capacity spectrum plot (ATC 40) and FEMA 356 coefficient method plot along with resultant displacement versus displacement diagram. Performance point of a structure can be find out using ATC 40 capacity spectrum or FEMA 356 coefficient method.

Table 4.3: Performance point results
(Case: Hinge at bottom only)

Method	Base shear (KN)	Target Displacment(m)
ATC 40	17248.574	0.012

FEMA356	23050.417	0.017

Table 4.4: Performance point results
(Case: Hinge at top & bottom)

Method	Base shear (KN)	Target Displacment(m)
ATC 40	17581.955	0.012
FEMA 356	23546.117	0.017

ATC 40 method gives much conservative value for base shear and displacement than FEMA 356 method. The base shear value obtained is the capacity of the structure and it is more than the designed base shear. So the structure is said to satisfy the performance criteria and so no need of retrofitting for the structure. The displacement capacity of structure is obtained as 0.012 m or 12 mm in ATC40 method and 0.017m or 17 mm in FEMA 356 method.

From hinge results it is evident that all the hinges are under 'immediate occupancy' level till step 4 of analysis for case 1. From pushover curve for case 1 the min base shear corresponding to step 4 = 29.93x10^{3}KN. It is twice than that of IS code. The structure is in 'Life safety' level upto step 8 of analysis. Base shear can be increased above 45x10^{3}KN within that step. And till step 10 hinges at pier 1 and pier 2 was at 'Life safety' level. But the hinge at pier 3 (extreme end from loading point) reach collapse prevention in step 10.

The hinges are under 'immediate occupancy' level till step 2 of analysis for case 2. From pushover curve for case 2 the min base shear corresponding to step 2 = 16.76x10^{3}KN. The hinge result does not show a life safety level or collapse prevention level. This may due to moment developed. From table, it is evident that moment increased very rapidly after step 2.

Hinge results shows that the risk factor is higher in case 2. So, two hinge cases give more conservative result than hinge at pier bottom only.

3.3 FEMA 356 and Caltrans Hinge

From literal studies, it is observed that many studies use default hinge properties for non-linear analysis of hinges. FEMA 356 is actually meant for steel bridge columns are often used in concrete bridge design also because of its easy assignments. Caltrans hinge is suggested for concrete bridge columns by Computers and Structures Inc. But for assigning Caltrans we want to calculate the hinge length or do seismic analysis on basis of AASHTO code. In that case they calculated hinge length automatically.

The effect of use of FEMA356 hinge instead of Caltrans hinge is studied using a number of pushover analyses. Importance of hinge location assignment or hinge length calculation is also checked. FEMA 356 hinges are used at relative distance of 0.05 and 0.95 distances in most of the studies. So, to find the effect of location 5 sets of analysis is conducted at relative distances 0.97, 0.96, 0.95, 0.94, 0.93. In this study, hinge provided at bottom only for ease of analysis.

From analysis result FEMA 356 hinge at different location shows that in all the assigned location hinges showed the same hinge status but the load involved varies. 0.95 relative distance involved comparably higher load so it may be considered as the critical hinge location, but the change in load involved in other locations vary in small amounts only. So, more detail studies need to reach the exact conclusion.

From the FEMA356 and Caltrans hinge assigning it is clear that FEMA356 show higher safety to the structure because of its steel property. But Caltrans gives sudden jump to collapse prevention from immediate occupancy. So, Caltrans hinge is safer to use in concrete column to get more conservative analysis result.

FEMA 356 shows a capacity of 37x10^{3}KN in immediate occupancy level, where Caltrans show nearly 17x10^{3}KN for the same performance. That means there is an exaggeration of 100% and more in result. Use of FEMA 356 hinge in concrete bridges will affect the structural safety.

3 Conclusion

- Displacement based analysis give more economic structure while achieving the same performance expected as per Indian code. The structure using in this study designed for a base shear of 15693KN as per Indian code. But the actual structure shows 60% extra capacity in case of hinge at bottom of pier (as per IS 1893(part 3):2014) and 24 % extra capacity in case of hinge at top and bottom (as per AASHTO LRFD: 2012).
- By calculating target displacement required for a structure we can design structure with best economic. Target displacement of the studies bridge is calculated as 0.012 m or 12mm.
- Base shear value obtained for performance point is higher than the designed base shear so the structure is safe and no need of retrofitting for the structure.
- Indian code should include more detail clauses towards performance based analysis. Present code gives only a brief note towards pushover analysis, which is inadequate for the proceedings.
- Using Sap2000, we can't assign every condition for seismic design on IS code basis. So, either study should conduct on basis of AASHTO code basis with some random assumptions about site and zone conditions or with IS code with some assumptions about hinge properties and other assignments.
- AS per Indian code, single pier bridges need only hinge at bottom for design. But from this study it is obtained that hinge at both top and bottom of pier give more safe analysis result. Sudden failure of hinge occurs after target displacement due to rapid increase in moment. The base shear value and target displacement obtained have only slight variations.
- FEMA 356 hinge shows 100% or higher safety than Caltrans hinge so use of FEMA 356 hinge in concrete bridges will affect the structural safety.
- This study is based on a pushover analysis with concentrated plasticity (plastic hinges) which is an approximate method. A more reliable nonlinear dynamic analysis with distributed plasticity model

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can be implemented if necessary.

4.1 Future scope

The effect of hinge property in nonlinear analysis is not just about its location. Detail study can be conducted to analyse the other depending factors such as span of column and Ast or grade of concrete in detail and generate a formulae suggestion for IS code for calculating hinge length.

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REFERENCES

[1] Abey E. T, T. P. Somasundaran, A. S. Sajith "*A Critical Evaluation of the Indian Codes on Seismic Analysis of Concrete Bridges*" IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE)

[2] Ian BUCKLE, Wen-huei (Phillip) YEN, Lee MARSH3 and Eric MONZON "*Implications of bridge performance during great east Japan earthquake for u.s. seismic design practice*" Proceedings of the International Symposium on Engineering Lessons Learned from the 2011 Great East Japan Earthquake, March 1-4, 2012, Tokyo, Japan

[3] Payam Tehrani and Denis Mitchell "*Seismic Risk Assessment of Four-Span Bridges in Montreal Designed Using the Canadian Bridge Design Code*" ASCE journal of bridge engineering, 2014.

[4] B Li, J Butterworth and N Chouw "*Seismic design of New Zealand highway bridges under spatially varying ground excitations*" NZ Transport Agency research report 504, December 2012

[5] Qiang Xue, Chyuan Jhang "*Performance-based Seismic Design Concept for Roadway/Highway Bridges in the USA and Japan*" Sinotech Engineering. Vol.115. April 2012.

[6] Ashok Venkittaraman "*Seismic resilience of highway bridges*" Thesis :The Pennsylvania State University ,May 2013

[7] C.V.R Murthy and Sudhir K Jain "*A proposed draft for Indain code provision on seismic design of bridges –part II : Commentary*" Journal of structural engineering , vol.27 july 2000.

[8] Redwood City, CA And Buffalo, NY "*Comprehensive Specification for the Seismic Design of Bridges*", NCHRP report 472.

[9] M. Lee Marsh And Stuart J. Stringer " *Performance based-seismic bridge design*" NCHRP report 440

[10] Wancheng YUAN Mi ZHOU and Dong YAN "Study and trend on conceptual seismic design for bridges in China" The 14th World Conference on Earthquake Engineering , October 12-17, 2008, Beijing, China

[11] IS: 1893 -2002, *Indian Standard Criteria for Earthquake Resistant Design of Structures: Part 1 General Provisions and Buildings*. Bureau of Indian Standards, New Delhi, 2002.

[12] IS: 1893(part 3) draft, *Indian Standard Criteria for Earthquake Resistant Design of Structures: Part 3 Bridges and Retaining walls*. Bureau of Indian Standards, New Delhi.

[13] IRC:6-2010, *Standard Specifications and Code of Practice for Road Bridges*. The Indian Road Congress, New Delhi, 2010.

[14] IRC: 6 -2014, *Standard Specifications and Code of Practice for Road Bridges*. The Indian Road Congress, New Delhi.

[15] AASHTO LRFD Bridge- Design specifications 2012