A Thesis on

PUSHOVER ANALYSIS OF A REINFORCED CONCRETE BRIDGE

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MASTER OF TECHNOLOGY

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STRUCTURAL ENGINEERING

By

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DECLARATION

I declare that the research thesis entitled "**Pushover Analysis of a Reinforced Concrete Bridge**" is the bonafide research work carried out by me, under the guidance of **Mr. Md Tasleem Assistant. Professor, Department of Civil Engineering, Integral University, Lucknow**. Further I declare that this has not previously formed the basis of award of any degree, diploma, associate-ship or other similar degrees or diplomas, and has not been submitted anywhere else.

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CERTIFICATE

Certified that the thesis entitled **"Pushover Analysis of a Reinforced Concrete Bridge"** is being submitted by **Mr. Mohammad Farhan (Roll no-1700102824)** in partial fulfillment of the requirement for the award of degree of Master of Technology (Structures) of Integral University, Lucknow, is a record of candidate's own work carried out by him/her under my supervision and guidance.

The results presented in this thesis have not been submitted to any other university or institute for the award of any other degree or diploma.

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List of Symbols and Abbreviations

Symbol	***Description***
Bi,	Base shear for bridge by ESLM
Bp	Base shear for bridge by NSP
c	classical damping
C0	factor for MDOF displacement
C1	factor for inelastic displacement
C2	factor for strength and stiffness degradation
C3	factor for geometric nonlinearity
Keq	Equivalent Stiffness
Ki	Initial Stiffness
R	normalized lateral strength ratio
Sa	spectral acceleration
Sd	spectral displacement
S_l	spectral acceleration at long period
S_s	spectral acceleration at short period
T	fundamental natural period of vibration
Teq	Equivalent Period
Ti	initial elastic period of the structure
Tn	nth mode natural period
TT**	cumulative mass participating ratio for first four modes
Uy	modal mass participation
Vi	max shear demand for critical pier by ESLM
Vp	max shear demand for critical pier by NSP
α	post-yield stiffness ratio
βeq	equivalent damping
βi	initial elastic damping

βs	damping due to structural yielding
θu	ultimate rotation
θy	yield rotation
μ	displacement ductility ratio

Abbreviations

ACI	American Concrete Institute
ADRS	Acceleration Displacement Response Spectrum
AMC	Adaptive Modal Combination
ATC	Applied Technology Council
CQC	Complete Quadratic Combination
CSM	Capacity Spectrum method
DCM	Displacement Coefficient Method
DMM	Displacement Modification Method
ELM	Equivalent Linearization Method
ESLM	Equivalent Static Load Method
FEMA	Federal Emergency Management Agency
IO	Immediate Occupancy
IRC	Indian Road Congress
IRSA	Incremental Response Spectrum Analysis
IS	Indian Standard
LS	Life Safety
MDOF	Multi Degree of Freedom
MMPA	Modified Modal Pushover Analysis
MPA	Modified Pushover Analysis
MPA	Modal Pushover Analysis
NSP	Non-Linear Static Pushover

PGA	Peak Ground Acceleration
RC	Reinforcement Concrete
RHA	Response History Analysis
SAP	Structural Analysis Program
SPA	Standard Pushover Analysis
SRSS	Square Root of Sum of Squares
UMRHA	Uncoupled Modal Response History Analysis
UPBA	Upper Bound Pushover Analysis

ABSTRACT

Seismic vulnerability assessment of existing structure has gained nation-wide attention, particularly after 2001 Gujarat Earthquake and 2005 Kashmir Earthquake. There are many literatures available on the seismic evaluation procedures of multi-storeyed buildings using nonlinear static (pushover) analysis. There is not much effort 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 (ATC 40), displacement modification method (FEMA 440) and equivalent linearization method (FEMA 440). Each series consist of five bridges one with varying span and other with varying pier height. Some of the analysis parameters were suitably modified to use in a bridge structure. The evaluation results presented here shows that the modelled bridges designed as per IS codes falls short to meet the desired performance level as per nonlinear pushover scheme.

CHAPTER 1

INTRODUCTION

1.1 Background

India has had a number of the world's greatest earthquakes in the last century. In fact, more than fifty percent area in the country is considered prone to damaging earthquakes. The northeastern region of the country as well as the entire Himalayan belt is susceptible to great earthquakes of magnitude more than 8.0. After 2001 Gujarat Earthquake and 2005 Kashmir Earthquake, there is a nation-wide attention to the seismic vulnerability assessment of existing buildings. Also, a lot of efforts were focused on the need for enforcing legislation and making structural engineers and builders accountable for the safety of the structures under seismic loading. The seismic building design code in India (IS 1893, Part-I) is also revised in 2016. 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. There are many literature (*e.g.*, IITM-SERC Manual, 2005) available that presents step-by-step procedures to evaluate multi-storey buildings. This procedure follows nonlinear static (pushover) analysis as per FEMA 356.

The attention for existing bridges is comparatively less. However, bridges are very important components of transportation network in any country. The bridge design codes, in India, have included seismic design provision at present. But, a large number of bridges were designed and constructed without considering seismic forces. Therefore, it is very important to evaluate the capacity of existing bridges against seismic force demand. There are presently no comprehensive guidelines to assist the practicing structural engineer to evaluate existing bridges and suggest design and retrofit schemes. 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 structures. Bridges are structurally very different from a multi-storey building. So, in the present study an improved pushover analysis is also used to verify the results.

1.2 PUSHOVER ANALYSIS

The use of the nonlinear static analysis (pushover analysis) came in to practice in 1970's but the potential of the pushover analysis has been recognized for last 10-15 years. This procedure is mainly used to estimate the strength and drift capacity of existing structure and the seismic demand for this structure subjected to selected earthquake. This procedure can be used for checking the adequacy of new structural design as well. The effectiveness of pushover analysis and its computational simplicity brought this procedure in to several seismic guidelines (ATC 40 and FEMA 356) and design codes (Eurocode 8 and PCM 3274) in last few years.

Pushover analysis is defined as an analysis wherein a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and element of the building shall be subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a 'target displacement' is exceeded. Target displacement is the maximum displacement (elastic plus inelastic) of the building at roof expected under selected earthquake ground motion. Pushover analysis assesses the structural performance by estimating the force and deformation capacity and seismic demand using a nonlinear static analysis algorithm. The seismic demand parameters are global displacements (at roof or any other reference point), storey drifts, storey forces, component deformation and forces. The analysis accounts for geometrical nonlinearity, material inelasticity and the redistribution of internal forces.

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

1. Estimates of force and displacement capacities of the structure. Sequence of the member yielding and the progress of the overall capacity curve.
2. Estimates of force (axial, shear and moment) demands on potentially brittle elements and deformation demands on ductile elements.
3. Estimates of global displacement demand, corresponding inter-storey drifts and damages on structural and non-structural elements expected under the earthquake ground motion considered.
4. Sequences of the failure of elements and the consequent effect on the overall structural stability.
5. 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.

Pushover analysis delivers all these benefits for an additional computational effort (modeling nonlinearity and change in analysis algorithm) over the linear static analysis. Step by step procedure of pushover analysis is discussed next chapter.

1.3 Pushover Analysis Procedure

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 building (Fig 1.1 a). Building is displaced till the 'control node' reaches 'target displacement' or building 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 (Fig1.1 b).

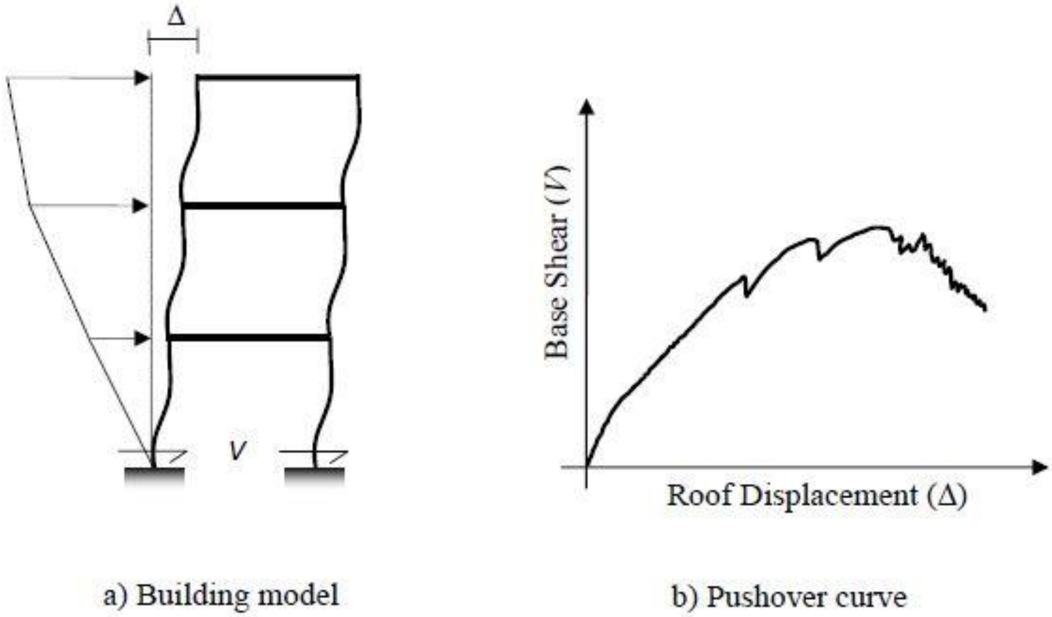


Fig 1.1: Schematic representation of pushover analysis procedure

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 capacity curve is the basis of 'target displacement' estimation. 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 seismic demands for the selected earthquake (storey drifts, storey forces, and component deformation and forces) are calculated at the target displacement level.

The seismic demand is then compared with the corresponding structural capacity or predefined performance limit state to know what performance the structure will exhibit. Independent analysis along each of the two orthogonal principal axes of the building is permitted unless concurrent evaluation of bidirectional effects is required.

The analysis results are sensitive to the selection of the control node and selection of lateral load pattern. In general, the centre of mass location at the roof of the building is considered as control node. For selecting lateral load pattern in pushover analysis, a set of guidelines as per FEMA 356 is explained. The lateral load generally applied in both positive and negative directions in combination with gravity load (dead load and a portion of live load) to study the actual behavior.

1.3.1 Lateral Load Patterns

In pushover analysis the building is pushed with a specific load distribution pattern along the height of the building. The magnitude of the total force is increased but the pattern of the loading remains same till the end of the process. Pushover analysis results (*i.e.*, pushover curve, sequence of member yielding, building capacity and seismic demand) are very sensitive to the load pattern. The lateral load patterns should approximate the inertial forces expected in the building during an earthquake. The distribution of lateral inertial forces determines relative magnitudes of shears, moments, and deformations within the structure.

The distribution of these forces will vary continuously during earthquake response as the members yield and stiffness characteristics change. It also depends on the type and magnitude of earthquake ground motion. Although the inertia force distributions vary with the severity of the earthquake and with time, FEMA 356 recommends primarily invariant load pattern for pushover analysis of framed buildings. Several investigations (Mwafy and Elnashai, 2000; Gupta and Kunnath, 2000) have found that a triangular or trapezoidal shape of lateral load provide a better fit to dynamic analysis results at the elastic range but at large deformations the dynamic envelopes are closer to the uniformly distributed force pattern. Since the constant distribution methods are incapable of capturing such variations in characteristics of the structural behavior under earthquake loading, FEMA 356 suggests the use of at least two different patterns for all pushover analysis. Use of two lateral load patterns is intended to bind the range that may occur during actual dynamic response. FEMA 356 recommends selecting one load pattern from each of the following two groups:

Group – I:

i) Code-based vertical distribution of lateral forces used in equivalent static analysis (permitted only when more than 75% of the total mass participates in the fundamental mode in the direction under consideration).

ii) A vertical distribution proportional to the shape of the fundamental mode in the direction under consideration (permitted only when more than 75% of the total mass participates in this mode).

iii) A vertical distribution proportional to the story shear distribution calculated by combining modal responses from a response spectrum analysis of the building (sufficient number of modes to capture at least 90% of the total building mass required to be considered). This distribution shall be used when the period of the fundamental mode exceeds 1.0 second.

Group – II:

i) A uniform distribution consisting of lateral forces at each level proportional to the total mass at each level.

ii) An adaptive load distribution that changes as the structure is displaced. The adaptive load distribution shall be modified from the original load distribution using a procedure that considers the properties of the yielded structure.

Instead of using the uniform distribution to bind the solution, FEMA 356 also allows adaptive lateral load patterns to be used but it does not elaborate the procedure. Although adaptive procedure may yield results that are more consistent with the characteristics of the building under consideration it requires considerably more analysis effort. Fig. 2.2 shows the common lateral load pattern used in pushover analysis.

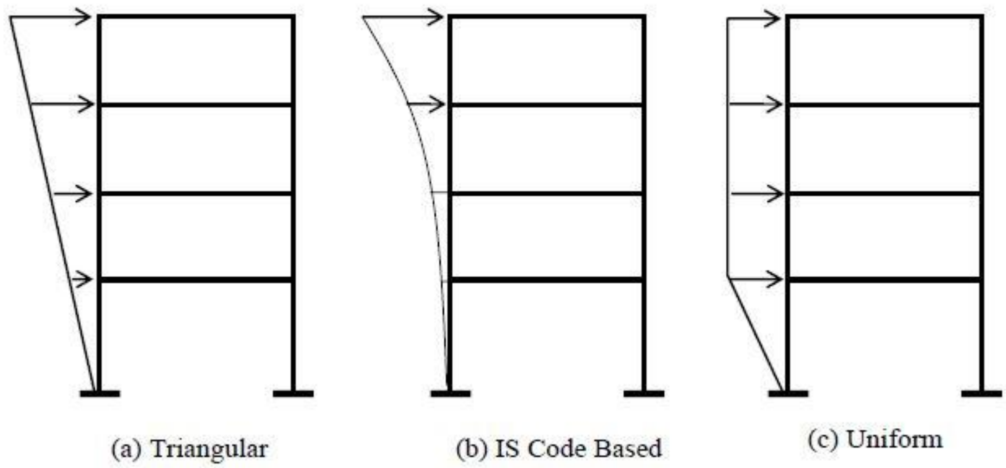


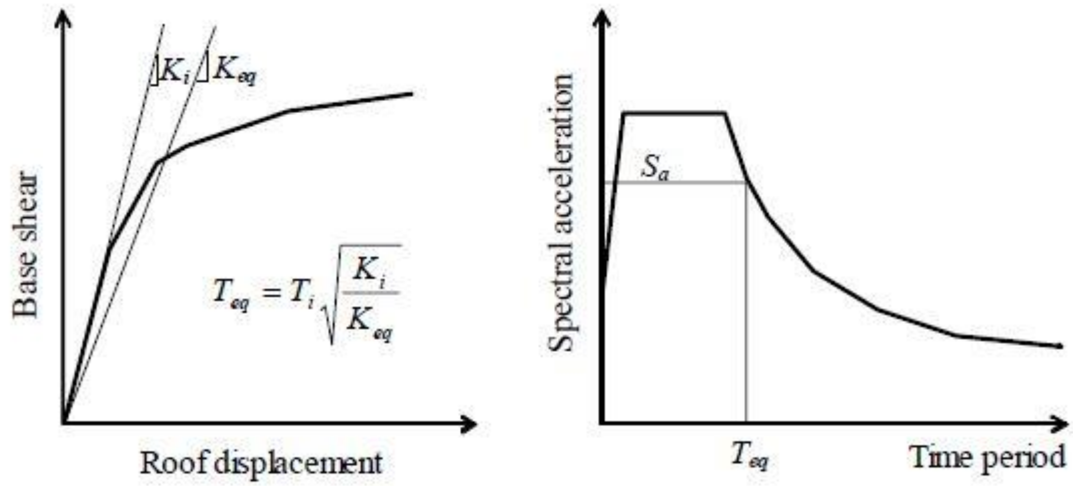
Fig 1.2: Lateral load pattern for pushover analysis as per FEMA 356 (considering uniform mass distribution)

1.3.2 Target Displacement

Target displacement is the displacement demand for the building 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 building at the target displacement are compared with the desired performance limit state to know the building 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, (c) Displacement Modification Method (FEMA 440) and (D)Equivalent Linearization Method (FEMA 440). First two methods use pushover curve to calculate global displacement demand on the building from the response of an equivalent single-degree-of-freedom (SDOF) system. The only difference in these two methods is the technique used. Next two method are improved forms of first two methods.

1.3.2.1-Displacement Coefficient Method (FEMA 356)

This method primarily estimates the elastic displacement of an equivalent SDOF system assuming initial linear properties and damping for the ground motion excitation under consideration. Then it estimates the total maximum inelastic displacement response for the building at roof by multiplying with a set of displacement coefficients.



(a) Pushover Curve (b) Elastic Response Spectrum

Fig 1.3: Schematic representation of Displacement Coefficient Method (FEMA 356)

The process begins with the base shear versus roof displacement curve (pushover curve) as shown in Fig. 3a. An equivalent period (*Teq*) is generated from initial period (*Ti*) by graphical procedure. This equivalent period represents the linear stiffness of the equivalent SDOF system. The peak elastic spectral displacement corresponding to this period is calculated directly from the response spectrum representing the seismic ground motion under consideration (Fig. 3b).

$S_d = T_{eq}^2 * S_a / 4\prod^2$ [1.1]

Now, the expected maximum roof displacement of the building (target displacement) under the selected seismic ground motion can be expressed as:

$$\delta_t = C_0 C_1 C_2 C_3 S_d = C_0 C_1 C_2 C_3 \frac{T_{eq}^2}{4\pi^2} S_a$$

[1.2]

C0 = a shape factor (often taken as the first mode participation factor) to convert the spectral displacement of equivalent SDOF system to the displacement at the roof of the building.

C1 = the ratio of expected displacement (elastic plus inelastic) for an inelastic system to the displacement of a linear system.

C2 = a factor that accounts for the effect of pinching in load deformation relationship due to strength and stiffness degradation

C3 = a factor to adjust geometric nonlinearity (P-Δ) effects

These coefficients are derived empirically from statistical studies of the nonlinear response history analyses of SDOF systems of varying periods and strengths and given in FEMA 356.

1.3.2.2 Capacity Spectrum Method (ATC 40)

The basic assumption in Capacity Spectrum Method is also the same as the previous one. That is, the maximum inelastic deformation of 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. This procedure uses the pushover curve in an acceleration displacement response spectrum (ADRS) format. This can be obtained through simple conversion using the dynamic properties of the system. The pushover curve in an ADRS format is termed a ‘capacity spectrum’ for the structure. The seismic ground motion is represented by a response spectrum in the same ADRS format and it is termed as demand spectrum (Fig. 2.4). The equivalent period (*Teq*) is computed from the initial period of vibration (*Ti*) of the nonlinear system and displacement ductility ratio (μ). Similarly, the equivalent damping ratio (β*eq*) is computed from initial damping ratio (ATC 40 suggests an initial elastic viscous damping ratio of 0.05 for reinforced concrete building) and the displacement ductility ratio (μ). ATC 40 provides the following equations to calculate equivalent time period (*Teq*) and equivalent damping (β*eq*).

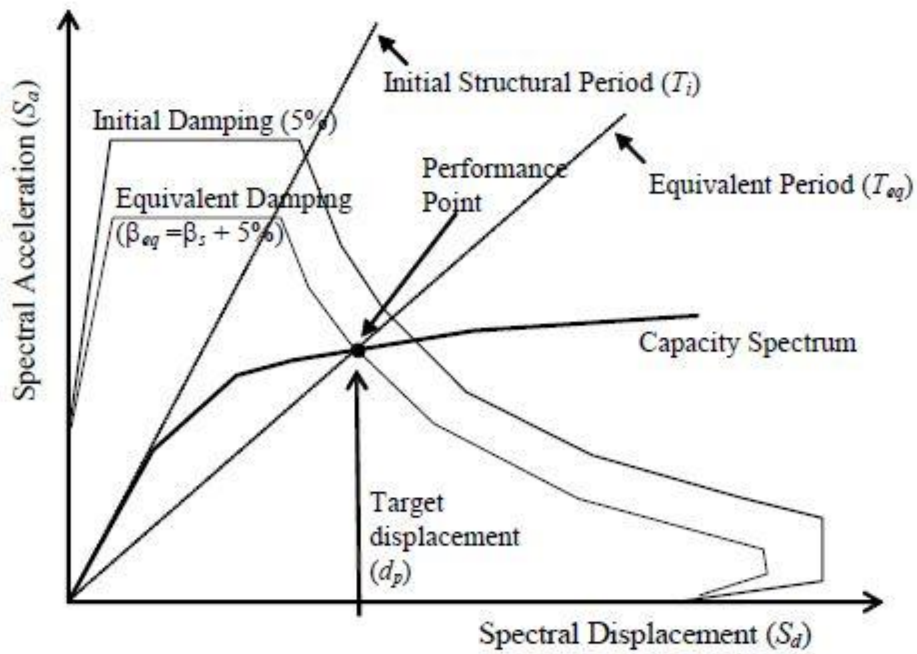


Fig. 1.4: Schematic representation of Capacity Spectrum Method (ATC 40)

$$T_{eq} = T_i \sqrt{\frac{\mu}{1 + \alpha\mu - \alpha}}$$

$$\beta_{eq} = \beta_i + \kappa \frac{2}{\pi} \frac{(\mu - 1)(1 - \alpha)}{\mu(1 + \alpha\mu - \alpha)} = 0.05 + \kappa \frac{2}{\pi} \frac{(\mu - 1)(1 - \alpha)}{\mu(1 + \alpha\mu - \alpha)}$$

[1.3]

where α is the post-yield stiffness ratio and κ is an adjustment factor to approximately account for changes in hysteretic behavior in reinforced concrete structures.

1.3.2.3 Displacement Modification Method (FEMA 440)

This improvement for the earlier Displacement coefficient method uses advanced equations for different coefficients. The procedure and basic equation used for obtaining expected target displacement is same as given in FEMA 356. But the method calculation for coefficients C1 and C2 are modified. These are:

$$C_1 = 1 + \frac{R - 1}{90\, T^2}$$

[1.4]

$$C_2 = 1 + \frac{1}{800}\left(\frac{R-1}{T}\right)^2 \quad [1.5]$$

1.3.2.4 Equivalent Linearization Method (FEMA 440)

This improved version of equivalent linearization is derived from the statistical analysis of large number of responses against different earthquake ground motions. The assumption in CSM that the equivalent stiffness of inelastic system will be the same as its secant stiffness is not used here. Instead, the equivalent stiffness is obtained from effective time period and damping properties derived using equations from statistical analyses.

Effective viscous damping values, expressed as a percentage of critical damping, for all hysteretic model types and alpha values have the following form:

For $1.0 < \mu < 4.0$:

$$\beta eff = A(\mu - 1)^2 + B(\mu - 1)^3 + \beta_0 \quad [1.6]$$

For $4.0 \le \mu \le 6.5$:

$$\beta eff = C + D(\mu\text{-}1) + \beta_0 \quad [1.7]$$

For $\mu > 6.5$:

$$\beta_{\text{eff}} = E\left[\frac{F(\mu-1)-1}{[F(\mu-1)]^2}\right]\left(\frac{T_{\text{eff}}}{T_0}\right)^2 + \beta_0 \quad [1.8]$$

Values of the coefficients in the equations for effective damping of the model oscillators is provided in FEMA 440.

1.4 Research Objective

Following are the main objectives of the present study:

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.

1.5 Methodology

1. A thorough literature review on application of Adaptive pushover analysis for RCC Bridge and seismic performance of bridge piers.
2. Carry out bridge modeling in suitable software and design the bridge as per design code IRC 21-2000 & IRC 6-2002 and perform pushover analysis.
3. Model the bridge with varying span sizes and perform pushover analysis.
4. Repeat the Bridge modeling 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. Compare the modeled bridges for various performance levels.

CHAPTER 2
LITERATURE REVIEW

A lot of work has been done in analysing multi-storey building with nonlinear static pushover analysis method. At first the technique was used for building with symmetric geometry and regular plan. A Modified Pushover Analysis model is developed to analyse buildings with irregularities in elevation and plan. This model is further developed with necessary altercation in procedure to analyze RC bridges. Several authors reported their work on Pushover analysis of Reinforced Concrete Bridges.

2.1 Application of Pushover analysis to Bridges

N.K. Manjula et al. (2013) compared and identified the differences among the pushover analysis methods given in international standards, considering one reinforced concrete (RC) building frame, designed as per IS 1893-2002 provisions. The performance of the building which is designed based on strength based method with sufficient ductile detailing is also evaluated. They performed the pushover analysis using two different levels of ground motion with ATC-40 method, FEMA340 method and their modified method in FEMA440. It was concluded that the difference between the results for four method is negligible for low seismic activity while appreciable variation in case of high seismic activity.

Nasim K. Shatarat (2012), emphasized on determining the nonlinear properties of the bridge element. In this study, pushover analysis of two highway bridges built with little attention to seismic forces was performed in an effort to evaluate the difference in global response predicted by using the user-defined nonlinear hinge properties and automated hinge properties in the software SAP2000. The results demonstrated that user-defined hinge model is capable of capturing the effect of local failure mechanisms, in the plastic hinge region, on the global response of the bridge; while the automated-hinge model cannot capture this effect. Therefore, automated-hinge properties should be used with a lot of care, especially for old bridges that might include local failure mechanisms in the plastic hinge region. User defined hinge properties can be obtained using the recommendations of the Seismic Retrofit Manual by the Federal Highway Administration. Automated-hinge properties in SAP2000 are computed automatically from the element material and section properties according to Caltrans criteria.

Bernardo. F (2012), assessed the feasibility and accuracy of non-linear static analysis in comparison with Time History Analysis. He developed two MATLAB programs and performed pushover analysis (NSPA), one as per methodology presented in Eurocode-2008 and second by ATC-40, Capacity Spectrum method. Analysis was performed on existing multi-span RC bridge of total length 360m with 12 equal spans. Pushover demand was compared with Time History Analysis results of bridge. He concluded that Demand Displacement obtained from CSM was smaller than Eurocode and time history analysis and Euro code results were more accurate and safer. For complex bridges the relative differences between the displacements obtained with both pushover methodologies were higher for more irregular structures and the difference increased with the value of the force ductility factor.

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Bindhu K.R, Rahul Leslie, (2012), utilized the SAP 2000 software for performing adaptive pushover analysis for two dimensional reinforced concrete buildings. A procedure to utilize SAP2000 package for performing adaptive pushover analysis is developed. However, the pushover curve of the conventional analysis and the adaptive analysis shows a close resemblance; which implies that there is no apparent and appreciable advantage of the adaptive approach over the conventional pushover analysis, using multimode lateral load, for buildings with regular configuration. He concluded that the study to be extended to Irregular geometry, plan and elevation, to reach any consensus.

M.Pand,K Seth (2011),performed NSPA(nonlinear static pushover analysis) on an existing multi-span RC Bridge consisting of 12 span of 30m each. The modeling of bridge was done using SAP 2000 and non-linear hinge properties were generated using improved curves for concrete and reinforcing steel. Two load patterns were selected as per FEMA-356, while target displacement was estimated using (a) Upper Bound Pushover Analysis (UPBA) and (b) FEMA-356 Displacement Coefficient Method. The study concluded that the performance of the bridge was below seismic demand, hence needed retrofitting. The study pointed the critical elements and locations for retrofitting. The author concluded that further investigation on bridges with different configuration was required to arrive at a generalized NSPA procedure for bridges.

Kapposa et al. (2010), used Modal Pushover analysis as means of seismic assessment of bridge structure. They investigated the extension of the modal pushover method to bridges, and also its applicability in the case of complex bridges. He proposed a clearly defined procedure for applying the MPA in the case of bridges and then attempted to quantify the relative accuracy of the three main inelastic analysis methods (i.e. SPA, MPA and nonlinear time history) by focusing on the realistic case of a complex, long and curved actual bridge. To this effect, a real, long and curved bridge is chosen, designed according to current seismic codes; this bridge is assessed using the aforementioned three nonlinear analysis methods. Comparative evaluation of the calculated response of the bridge illustrates the applicability and potential of the modal pushover method for bridges, and quantifies its relative accuracy compared to that obtained through the ‘standard’ pushover approach. The author concluded that MPA is a promising approach and yielded more accurate results compared to the ‘standard’ pushover, without requiring the high computational cost of the Non Linear Time History Analysis.

Jingyao Zhang, et al. (2008), proposed a new approach for determination of equivalent static seismic loads, for evaluating peak seismic responses. The responses were estimated by series of multi-modal pushover analysis considering possible phase differences in the dominant modes: the loads are directly applied in the elastic systems, and the damping due to plastic dissipation is modeled by equivalent linearization in inelastic systems. The accuracy of the proposed method was demonstrated in the numerical example of an arch-type long-span truss. Numerical studies on a long-span arch model, of which the seismic response is dominated by two vibration modes, show that the proposed method has good performance in estimating the peak responses for elastic systems as well as inelastic systems. Although the proposed method requires eigen value analysis and pushover analysis for several load patterns, it is effective and accurate enough for estimating peak seismic responses of spatial structures, as an alternative tool of the time-consuming time-history analysis.

R. Pinho, et al (2007) investigated the effectiveness of pushover analysis in assessing bridges subjected to seismic action. The author proposed adaptive pushover scheme based on variable force distribution accounting for structural yielding and the associated changes in the vibration properties. The author conducted parametric study on a suite of continuous multi-span bridges applying convention NSP with invariant force distribution and the proposed adaptive NSP approach. The study showed attainment of improved predictions over two pushover methods. Two bridges consisting of four span(50m) with continuous deck and different pier height were selected for the study. Pushover analysis of bridges was performed as per Eurocode 8. Target displacement at top of central pier was from 45mm to 160mm for different mode shapes. He concluded that the use of single-run pushover analysis might still be feasible even for such irregular bridge configurations, for as long as a displacement-based adaptive version of the method is employed.

Muljati and Warnitchai (2007) investigated the performance of Modal Pushover Analysis (MPA) to predict the inelastic response of the continuous bridge decks with no intermediate movement joints. He performed pushover analysis using an invariant lateral force distribution for each mode independently. The peak responses determined from every mode are combined using square-root of sum-of-square (SRSS) combinations. The authors reported that the performance of MPA in nonlinear range shows a similar tendency with MPA in linear range. Being an approximate method, MPA gives an acceptable accuracy beside of simplicity and efficiency in calculation.

Kalkan and Kunnath (2007) investigated the accuracy of pushover procedures for the seismic evaluation of buildings. These were the conventional pushover analysis using the Mode Shape load distribution and the Uniform load distribution, the Modified Modal Pushover Analysis, MMPA, the Upper-bound Pushover Analysis, and the Adaptive Modal Combination Procedure, AMC. These were applied to a 6- and 13-storey steel building, and to a 7- and a 20-storey RC moment frame building. The results from these analyses were compared to the results from nonlinear dynamic analyses based on the behaviour of these buildings to far-field and near-fault ground motions. The quantities of interest in this study were the displacement demands, inter-storey drifts and rotation demands. The study found that the conventional pushover analysis overestimated the displacement demands in the low and intermediate storeys for all buildings and ground motions. The upper-bound pushover analysis on the other hand underestimated the displacement demands. The MMPA and the AMC procedures overestimated the displacement demands but with the smallest error. These last two procedures predicted very similar results. Regarding the inter-storey drift demands the conventional pushover procedures significantly underestimated the drifts in the upper storeys and overestimated them in the lower storeys for most of the buildings. The upper-bound pushover analysis on the other hand, overestimated the drifts in the upper storeys and underestimated them in the lower storeys. The MMPA and the AMC methods performed slightly better with reasonable accuracy in the lower storeys but with overestimation in the upper storeys for most of the buildings. Finally the plastic rotation demands were compared between the MMPA, AMC and nonlinear dynamic analyses only. It was found that that the MMPA was able to capture the rotation demands mostly in the lower storeys. The AMC procedure was the most effective for estimating this quantity across the buildings' floors.

Tjhin, et al. (2006), evaluated the energy-based pushover method proposed by Hernandez-Montes *et al.* (2004) by studying the behavior of five building models; a three-storey and an eight-storey steel moment-resisting frames, a reinforced concrete wall building and two weak storey variants of the steel frames. The results showed that the proposed method was in general satisfactory to approximate the target displacements and inter-storey drifts. Furthermore, it was pointed out that conventional pushover procedures tended to underestimate the roof displacements. Questionable estimates were obtained for the storey shears and the overturning moments. The authors prompted for more clarification of the cases where these methods could be reliable.

Moghaddam and Hajirasouliha (2006) investigated the accuracy of pushover analysis when seismic demands need to be estimated for braced steel frames. Three steel braced frames of 5-, 10-, and 15- storey's were considered. Three different load patterns were used; the first mode distribution, the uniform distribution and the inverted triangular distribution. The results showed significant sensitivity to the choice of the load patterns for all the structures and were generally inaccurate. In this study the authors proposed a modified-shear building model that incorporated shear-type and flexural-type characteristics using springs to account for the shear and the additional flexural displacements of the building floors. The results were similar the conventional results and did not show much improvement. However the simplified model they proposed was computationally efficient and could predict the behavior very similarly to the detailed model.

Rupen Goswami and C. V. R. Murty (2005) reviewed the seismic strength design provisions for reinforced concrete (RC) bridge piers given in Indian codes. The author designed Bridge piers of varying width and pier height as per Indian codes and performed pushover analysis. The shear capacities of circular and rectangular sections, both solid and hollow, with nominal transverse reinforcement as recommended by IRC:21-2000 are found insufficient for target shear demand, premature brittle shear failure of piers is expected before the full flexural strength is achieved. The horizontal deflection of pier at performance point was in the range of 0.9-3.1% of pier height.

Craig D. COMARTIN et al. (2004) performed scrutiny for the efficacy of the then available two NSP procedure (i.e. Capacity Spectrum Method ATC-40 and Displacement Coefficient Method) and improve the application of inelastic analysis procedures for use with performance based engineering methods for seismic design, evaluation, and rehabilitation of buildings. They evaluated the current procedures on parameters predominant hysteretic behavior, basic global strength (R) and strength degradation. A total of 50 periods of vibration and 100 earthquake ground motions recorded on different site conditions were used in this study. The result of the variation in these basic parameters was a database of 180,000 nonlinear response history analyses representing the maximum displacement response of a SDOF oscillator subject to earthquake motions. The accuracy of the approximate nonlinear static procedures was determined by the comparing the predictions to actual response histories as a benchmark. The author proposed several improvements to the basic displacement modification procedure in *FEMA 356*, redefining coefficient of the basic equation and also propped modified method for obtaining it. For the procedure in ATC-40, he expressed equivalent period and equivalent damping as functions of ductility. These relationships are based on an optimization process whereby the error between the displacement prediction using the equivalent linear oscillator and using nonlinear response history analysis is minimized.

Hernandez *et al.* (2004) attempted to use an energy-based formulation for first- and multiple-mode nonlinear static pushover analyses. The method was presented in later section. The proposed method was compared with the conventional pushover analysis of a three-storey steel frame. It was concluded that the energy-based formulation provided a stronger theoretical basis for establishing the capacity curves of the first and higher mode equivalent SDOF systems by avoiding load reversals of the nonlinear static curves. The results provided a good estimate of target displacement with up to 10% error. The authors argued that the roof displacement was a useful index for the first mode response of many structures including structures for which displacements over the height of the structure did not increase proportionately. However, for structures outside this assumption field – for example braced structures-, the roof displacement index was deemed questionable, even for elastic response.

Matsumori and Shiohara (2004), compared results from nonlinear earthquake response analyses and static pushover analyses of two 12-story and three 18-story structures. Their main objective was to estimate member deformation demands, the distribution of storey displacements and member ductility demands across the structures' heights. According to their results the earthquake response analyses of the mentioned structures above, yielded ductility demands that varied significantly with different ground motions and structures. Secondly they concluded that the earthquake responses could be reasonably estimated by the results of pushover analyses by using a story shear distribution corresponding to the sum and the difference of the first two modes.

Chiorean (2003) evaluated a nonlinear static (pushover) analysis method for reinforced concrete bridges that predicts behavior at all stages of loading, from the initial application of loads up to and beyond the collapse condition. He developed NSP method using "line elements" approach, and are based on the degree of refinement in representing the plastic yielding effects. Distributed plasticity model and plastic hinge model were used to model elasto-plastic behavior. He investigated the collapse behavior of a three span pre-stressed reinforced concrete bridge of 115m in total length and used Capacity Spectrum Method as per Eurocode 8 (2003).The bridge modeling was done using NAFCAD(structural analysis software).Target displacement, base shear and deformation of plastic zone (hinges) were obtained and to be compared with time history analysis as future work.

Chung and Hamed (2003), performed seismic analysis of bridges using control displacement approach. A three-span bridge of 97.5 meters (320 ft) in total length was analyzed using both the Nonlinear Static Procedure/Displacement Coefficient Method and nonlinear time-history. Nine time-histories were implemented to perform the nonlinear time-history analysis. Three load patterns were used to represent distribution of the inertia forces resulting from earthquakes. Demand (target) displacement, base shear, and deformation of plastic hinges obtained from the Nonlinear Static (Pushover) Procedure are compared with the corresponding values resulting from the nonlinear time history analysis. Analysis was performed using two levels of seismic load intensities (Design level and Maximum Considered Earthquake level). Performance of the bridge was evaluated against these two seismic loads. Comparison shows that the Nonlinear Static Procedure gives conservative results, compared to the nonlinear time history analysis, in the Design Level while it gives more conservative results in the Maximum Considered Earthquake level.

A.K. Chopra and R. K. Goel, (2002), determined the exact response of the 9 storey SAC building by the two approximate methods, Uncoupled Modal Response History Analysis (UMRHA) and Modified Pushover Analysis (MPA) and compared with the exact results of non-linear RHA. The author performed the Pushover analysis and obtained peak responses for all mode shapes. The responses are combined by modal combination rule (SSRS rule) leading to the MPA procedure. The author reported that the approximate MPA procedure provided good estimates of floor displacements and storey drifts, and identified locations of most plastic hinges; however, plastic hinge rotations were less accurate. The author concluded that the MPA procedure is accurate enough for practical application in building evaluation and design. That said, however, all pushover analysis procedures considered do not seem to compute accurately local response quantities, such as hinge plastic rotations.

Yang and Wang (2000), applied the pushover method to three frame structures of 8, 12, and 15 storeys and compared the results with nonlinear time-history analyses. The results provided were estimates of roof displacement and floor rotations. In one case a difference of up to 30% could be observed but generally results could be deemed satisfactory. The differences in the results were mainly attributed by the authors to the frequency contents of the ground motions used. Also it was noted that the bilinear representation of the pushover curves introduced errors in the estimation of the base shear and the yield displacement. These in turn resulted in differences in the calculated responses between analyses.

Hosseini and Vayeghan (2000), recognized that some irregular buildings that had been designed using recent seismic codes had shown some vulnerability against earthquakes. Their observations led them to the conclusion that there was some need for further modifications to the design standards. They investigated the three-dimensional response of an existing irregular 8-story steel building designed according to the Iranian National Seismic Code by performing three-dimensional linear dynamic and nonlinear pushover analyses. Time-history analyses were performed by applying accelerograms of some local 56 earthquakes in different configurations to account for possible cases of seismic response. The response quantities that were investigated and compared were the displacements in different levels of the building, shear and axial forces and also bending moments in some corner, side and middle columns and some bracing elements, and finally the stresses in the critical members. Their numerical results showed that in the case of multi-component excitations the response values could be much higher than those predicted by code recommended loadings. Furthermore, it was observed that by considering geometric and material nonlinearity the ultimate sustained displacement of the building was decreased. Finally it was concluded that the nonlinear behaviour of the building was very different from that assumed in the Code Seismic Analysis. This difference was very obvious for the corner columns. Therefore further modifications would be needed in the code for considering irregular buildings.

Peter and Badoux (2000), applied the capacity spectrum method to a 9-storey reinforced concrete building with reinforced concrete and masonry structural walls. The structure was subjected to two strong ground motions. Three types of lateral load patterns were used to simulate seismic behaviour in a static manner. These were the uniform distribution, the modal distribution and the modal adaptive force distribution. The conclusion the authors drew from their study were that the CSM method was adequate to estimate seismic demands such as inter-storey drifts. Furthermore, the uniform load pattern proved to be quite effective. A need for more reliable structural models was acknowledged.

Kunnath and Gupta (1999a), introduced a spectrum-compatible pushover analysis method. The main differences between the conventional pushover analysis and the proposed method were that the latter included site-specific ground motion characteristics and secondly the applied load pattern changed depending on the instantaneous dynamic properties of the system. The proposed method was evaluated using a 14-storey moment-resisting frame. The results showed superiority of the method with respect to the conventional ones to capture plastic hinging especially in the upper stories when compared to the nonlinear dynamic analysis results. It was also concluded that the smooth spectra mostly used in the conventional methods were not sufficient to identify hinging at the upper stories. It was proposed that the method should be carried out on a greater number of structures to identify its potential.

Kunnath and Gupta (1999b), compared the responses of an 8-storey building derived from the spectra-compatible pushover method and the conventional pushover method. The superiority of the spectra-compatible method to the conventional pushover method for capturing upper-storey demands was pointed out, when results were compared with nonlinear dynamic analysis results. It was observed that the square root of the sum of the squares (SRSS) combination used, tended to magnify some modal contributions and this resulted in an underestimation of the lower- storey demands.

Summary of Literature Review

The ultimate aim of any pushover analysis should be the practical estimation of the seismic demand, in other words the estimation of the peak response quantities associated with the nonlinear deformation of the structure and its elements. The notion of practicality requires that the application of the pushover analysis should be simple enough and it should not involve any time-history analysis. Nonlinear pushover analysis is a powerful tool and is widely used for analytical evaluation of the Behaviour of structure in the inelastic range which identifies failure mechanisms and weak structural elements. But conventional nonlinear pushover analysis is limited by its overtly restrictive assumptions such as fundamental mode controlling structural response, fixed spatial distribution of lateral force predetermination of monitoring node and target displacement. Effort to include higher modes effects to 'standard' pushover analysis (SPA) so as to match the results of nonlinear time history analysis attracted the attention of researchers. In First of many efforts to include the effects of higher modes in pushover analysis, the multi-mode pushover methodology was utilized by Freeman [42].He extended the Capacity Spectrum Method (CSM) to compare earthquake demand with building capacity. Multiple adaptive pushover analogies were proposed by Kunnath et al[18],Gupta et al[26,28] and Antoniou et al[39] including multiple modes in defining lateral load patterns which are determined by combining modal load using modal combination rules(SRSS).Modal combination is performed at the stage of loading.

In a different approach for modal superimposition Goel and Chopra [7,8], proposed performing pushover analysis for each significant mode individually and combining the response quantities using an appropriate combination rule (SRSS or CQC). With new development in recent years an alternative types of nonlinear static analysis were proposed [5, 9, 18, 22, 24, 26, 30] involving multiple run pushover analyses. Each run corresponded to a given modal distribution and the overall structural response was estimated by combining individual seismic responses (displacement drifts etc) with combination rule. Since pushover procedure followed was conventional such approach had advantage of compatibility as it could be performed commercially available standard software packages.

In further development another theory for nonlinear static analysis called “incremental response spectrum analysis (IRSA)” was proposed by Aydinoglou [3], which proposed that with every formation of hinge, the elastic modal spectrum analysis for structure to be performed considering the changed dynamic properties of structure due to yielding. It was observed that for regular bridge geometry single run conventional pushover analysis yielded conservative results. In case of irregular or complex bridge structure multiple run adaptive pushover approach performed better and lead to better estimation of seismic responses. The level of accuracy in all mentioned approaches for pushover analysis is satisfactory but needs further work to reach international consensus of researchers.

2.3 Indian Code Provisions

IS: 1893(Part 1)-2016 provides the seismic loading criteria for structures in India. However, loads and stresses (including those due to seismic effects) for the design and construction of road bridges in India are governed by the Indian Road Congress specification IRC:6-2016. Additional design provisions specifically for concrete structures are specified in Indian Road Congress specification IRC:21-2010 and IRC:112-2011 (earlier in IRC:21-1987 & IRC:21-2000) and for bridge foundations and substructures in IRC:78-2014. (earlier in IRC:78-1983 & IRC:78-2000). In IRC:6-2016, the horizontal design earthquake load on bridges is calculated based on a *seismic coefficient*.

The horizontal seismic forces acting at the centers of mass, which are to be resisted by the structure as a whole, shall be computed as follows:

$$F_{eq}=A_h \text{ (Dead Load + Appropriate Live Load)}$$

Where, Feq = Seismic force to be resisted

Ah = Horizontal seismic coefficient = (Z/2)*I*(Sa/g)

Appropriate live load is taken as 0.2 times live load applied

Z = zone factor

I = importance factor

T = fundamental period of bridge in sec for horizontal vibration

Sa/g = Average responses acceleration coefficient for 5 percent damping of load resisting elements depending upon the fundamental period of vibration T as given in **Fig.20** of IRC 6:2016

Fundamental time period of the bridge member is to be calculated by any rational method of analysis adopting the Modulus of Elasticity of Concrete (Ecm) as per IRC:112, and considering moment of inertia of cracked section which can be taken as 0.75 times the moment of inertia of gross uncracked section, in the absence of rigorous calculation. The fundamental period of vibration can also be calculated by method given in **Annex D** of IRC 6:2016.

CHAPTER 3
STRUCTURAL MODELLING

3.1 Introduction

The study is based on nonlinear analysis of RC bridge models. This chapter presents a summary of various parameters defining the computational models, the basic assumptions and the bridge geometry considered for this study. Accurate modeling of the nonlinear 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.

Modeling a building involves the modeling and assemblage of its various load-carrying elements. The model must ideally represent the mass distribution, strength, stiffness and deformability. Modeling of the material properties and structural elements used in the present study is discussed below.

3.2 Structural Elements

Piers, cap and girders supporting deck are modeled by 3D frame elements. 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. However, the weight of the deck is applied on the beam as Dead Load. Also, mass of the deck is considered for modal analysis.

3.3 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.

3.3.1 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 model NWBR H5M, NWBR H10M. NWBR H15M, NWBR H20M & NWBR H25M with pier heights of 5m, 10m, 15m, 20m and 25m are adopted for the study. The width of the bridge is 10.5m

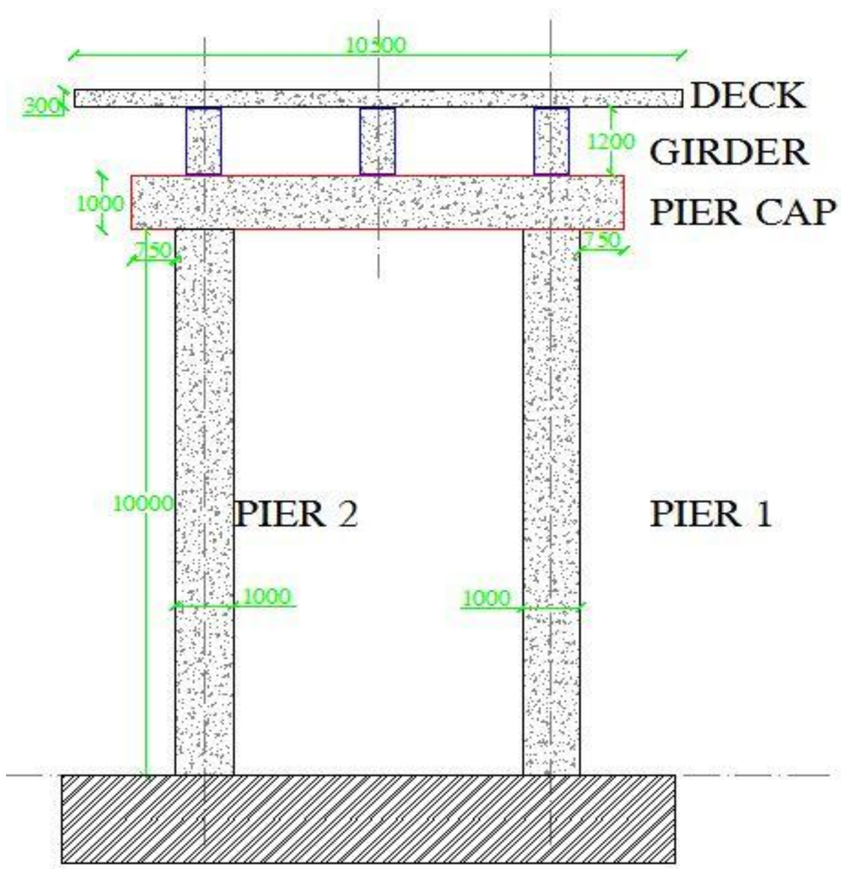


Fig. 3.1: Typical Cross-sectional details of the bridge

3.3.2 Fixed 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 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 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 SAP2000V 18.0.1.Ultimate. A 3D computer model is shown in below.

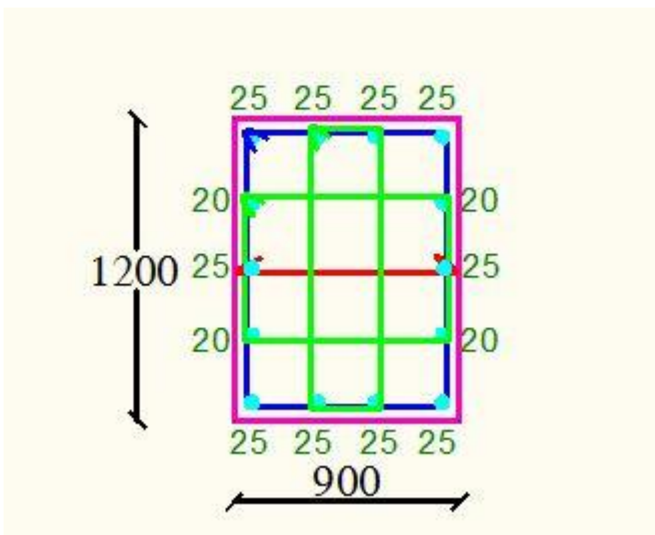


Fig. 3.2: Typical Details of the pier section

3.4 Modeling of flexural plastic hinges.

The development of sound model to explicitly define the nonlinear 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 , wherein the plastic hinge is assumed to be concentrated at a specific point in the frame member under consideration. 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).

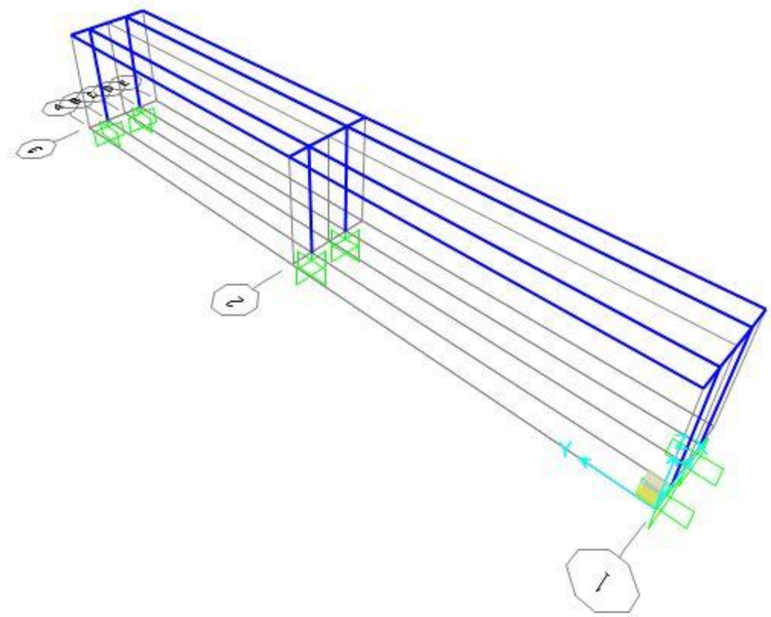


Fig 3.3: 3D model of the bridge

Plastic hinges are assumed at an offset of .05L from both ends. Properties of flexure hinges must simulate the actual response of reinforced concrete components subjected to lateral load. In practical use, most often the default properties provided in the FEMA-356 and ATC-40 documents are preferred due to convenience and simplicity. SAP2000 performs nonlinear static pushover analysis incorporated with the implementation of default flexural hinge properties based on FEMA-356 and ATC-40. It also allows modifying the default properties. In this study the concept of generated properties is used in SAP2000. When generated properties are used, the program combines its built-in criteria (FEMA-356 and ATC-40) with the defined section properties for each object to generate the final hinge properties.

Flexural hinges in this study are defined by moment-rotation curves calculated based on the cross-section and reinforcement details at the possible hinge locations. For calculating hinge properties it is required to carry out moment–curvature analysis of each element. Constitutive relations for concrete and reinforcing steel, plastic hinge length in structural element are required for this purpose. Although the axial force interaction is considered for pier flexural hinges the rotation values were considered only for axial force associated with gravity load.

3.4.1 Stress-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 behaviour 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). In reality, as evidenced by experimental testing, the post-peak behaviour is characterized by a descending branch, which is attributed to 'softening' and micro-cracking in the concrete. Also, models as per these codes do not account for strength enhancement and ductility due to confinement. However, the stress-strain relation specified in ACI 318M-02 consider some of the important features from actual behaviour. A previous study (Chugh, 2004) on stress-strain relation of reinforced concrete section concludes that the model proposed by Panagiotakos and Fardis (2001) represents the actual behaviour 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:

$$fc = f`_{cc} x\, r / r-1+x^{n} \tag{3.1}$$

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

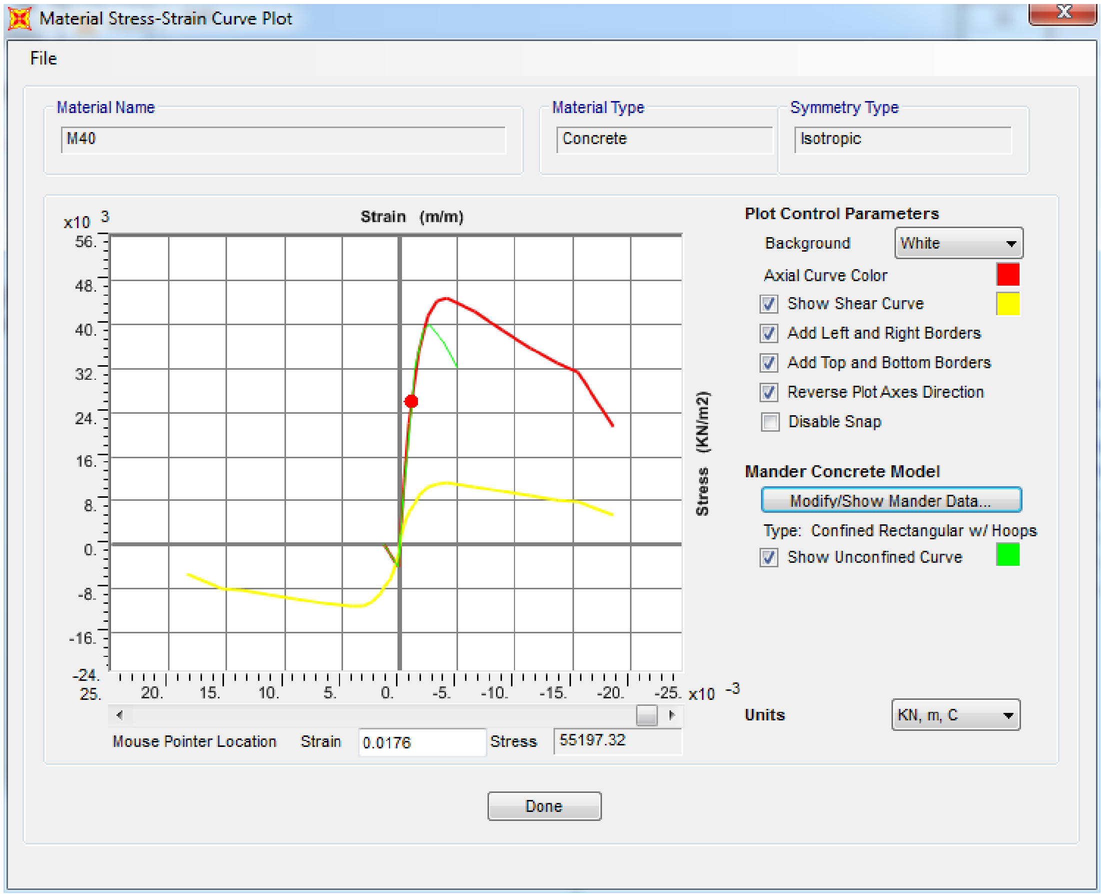


Fig3.4: Plot of stress-strain characteristics for M-40 grade of concrete as per Modified Mander's model

$$f'_{cc} = f'_{co}\left[1 + 3.7\left(\frac{0.5 k_e \rho_s f_{yh}}{f'_{co}}\right)^{0.85}\right] \tag{3.2}$$

The expressions for critical compressive strains (ref. Fig. 3.6) are expressed in this model as follows:

$$\varepsilon_{cu} = 0.004 + \frac{0.6 \rho_s f_{yh} \varepsilon_{sm}}{f'_{cc}} \tag{3.3}$$

$$\varepsilon_{cc} = \varepsilon_{co}\left[1 + 5\left(\frac{f'_{cc}}{f'_{co}} - 1\right)\right] \tag{3.4}$$

Where, 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:

- A single equation defines the stress-strain curve (both the ascending and descending branches) in this model.
- The same equation can be used for confined as well as unconfined concrete sections.
- 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).
- The validation of this model is established in many literatures.

3.4.2 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.

3.4.3 Moment-Rotation Parameters

Moment-rotation parameters are the actual input for modeling the hinge properties and this can be calculated from the moment-curvature relation. The moment-rotation curve can be idealized as shown in Fig. 3.7, and can be derived from the moment-curvature relation. The main points in the moment-rotation curve shown in the figure can be defined as follows:

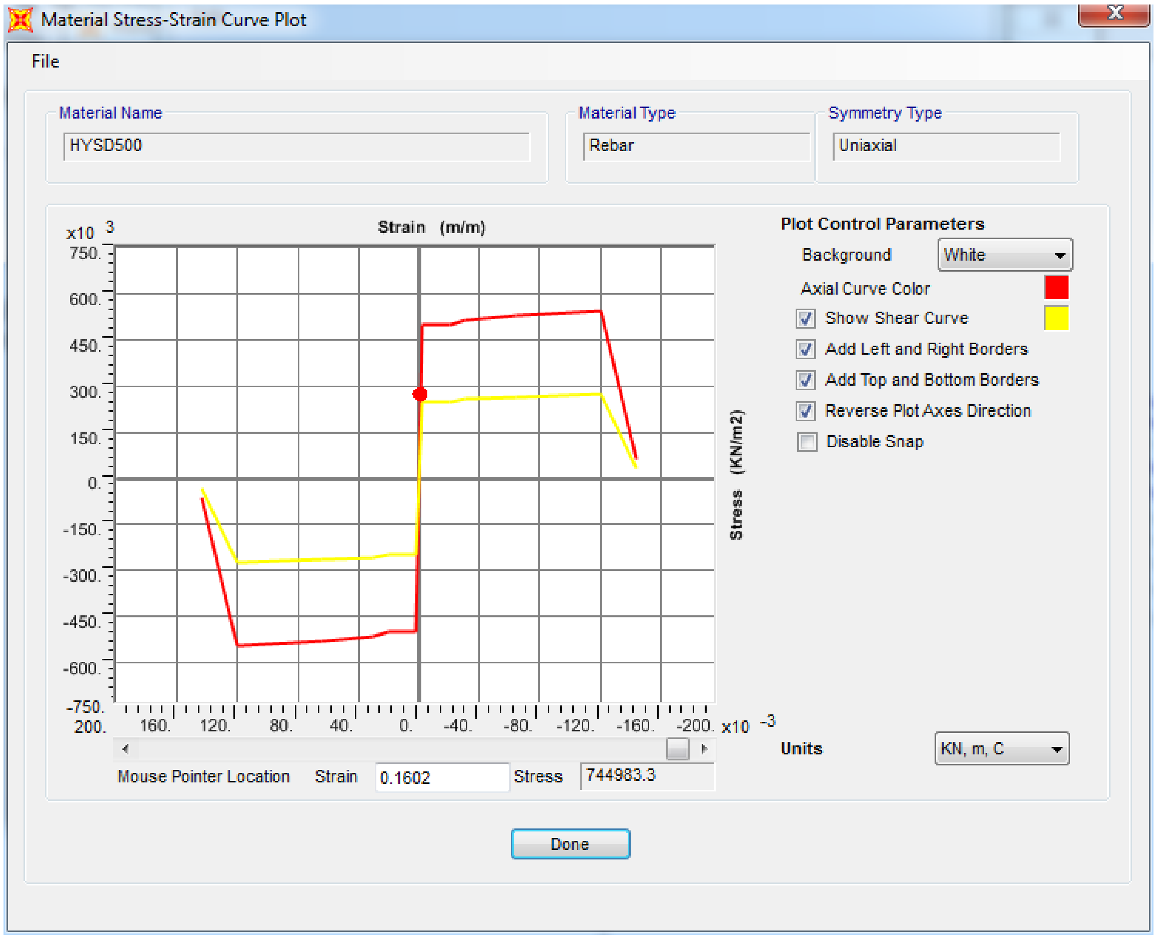


Fig3.5: Stress-strain relationship for reinforcement – IS 456 (2000)

- The point 'A' corresponds to the unloaded condition.
- The point 'B' corresponds to the nominal yield strength and yield rotation θy .
- The point 'C' corresponds to the ultimate strength and ultimate rotation θu , following which failure takes place.
- The point 'D' corresponds to the residual strength, if any, in the member. It is usually limited to 20% of the yield strength, and ultimate rotation, θu can be taken with that.
- The point 'E' defines the maximum deformation capacity and is taken as $15\theta y$ or θu , whichever is greater.

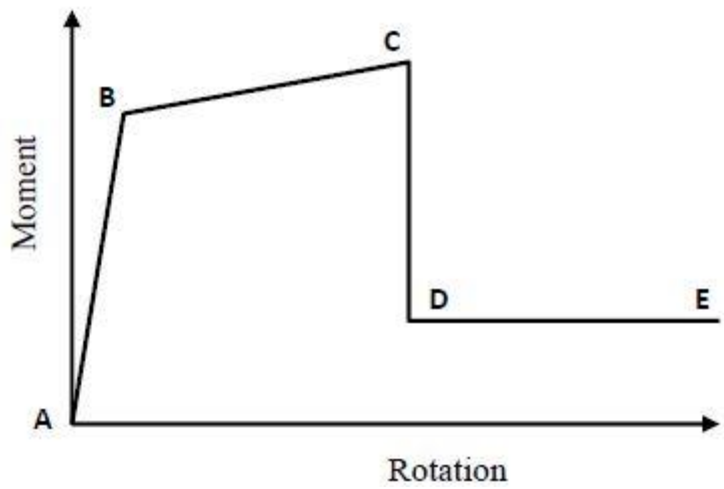


Fig. 3.6: Idealized moment-rotation curve of RC elements

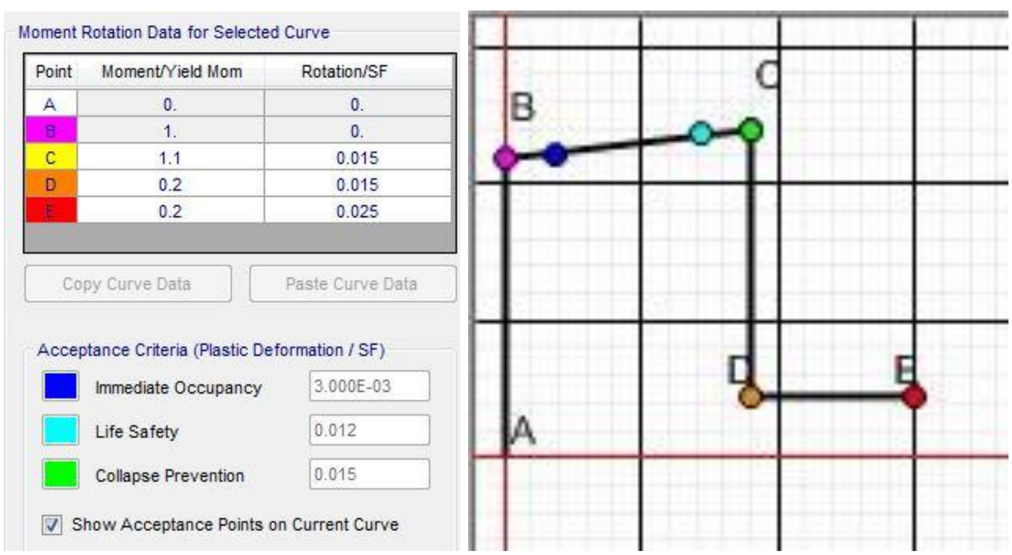


Fig. 3.7: Generated moment-rotation curve of RC elements with acceptance criteria

In this study hinges are defined as auto hinge types which are based on table in FEMA 356.Table 6-8(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 values is as shown in Fig. 3.8

3.5 SUMMARY

This chapter presents details of the basic modeling technique for the linear and nonlinear analyses of RC framed structures. It also describes the selected bridge geometries used in the present study. This chapter briefly discusses about modeling plastic flexural hinge.

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 Introduction

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). This chapter presents elastic modal properties of the bridge, pushover analysis results and discussions. Pushover analysis was performed first in a load control manner to apply all gravity loads on to the structure (gravity push). Then a lateral pushover analysis in transverse direction was performed in a displacement control manner starting at the end of gravity push. The results obtained from these analyses are checked against the seismic demand corresponds to the Zone V (PGA = 0.36g) of India as per the current bridge design codes (IRC:112-2011 & IRC:6-2016).

4.2 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 in transverse direction (X direction). The table shows that participating mass ratio in the first mode and cumulative mass participating ratio for first four modes for modeled bridges. 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%. Therefore, unlike regular buildings the higher mode participation in the response of bridge is significant. Figs. 4.1 and 4.2 present the first four mode shapes in the transverse direction.

One of the main assumptions for the standard pushover analysis (FEMA 356) is hundred percent fundamental mode contributions in the structural response which is not true for the bridges. Therefore, standard pushover analysis as per FEMA 356 is not suitable for the bridges.

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

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

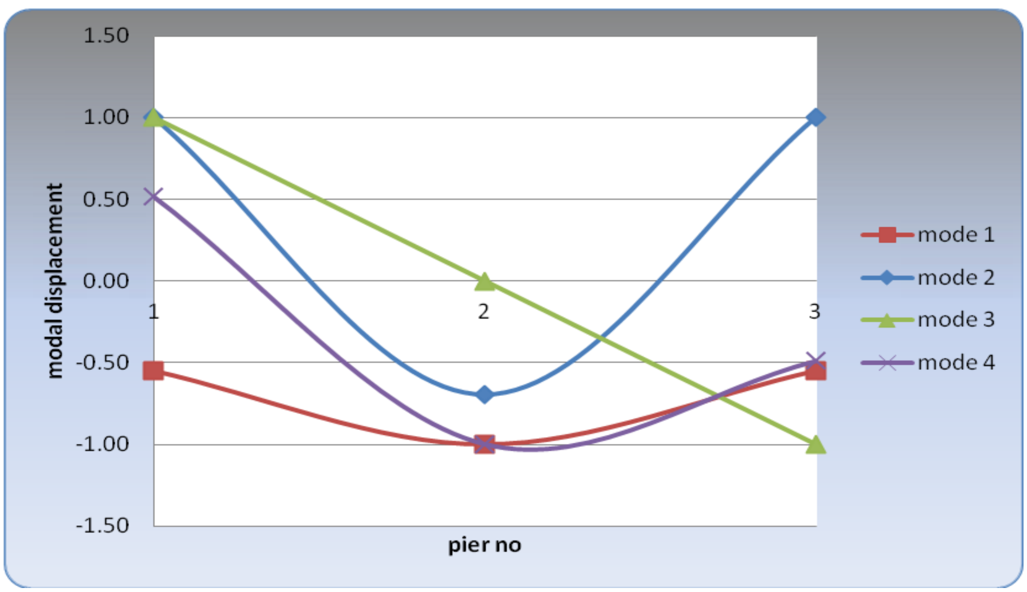
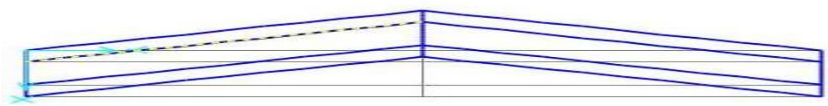


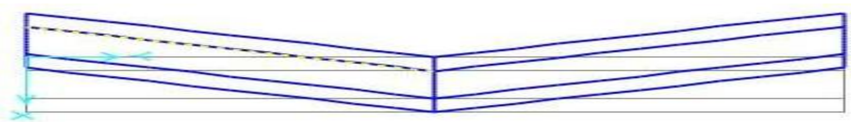
Fig. 4.1: First four modes of the bridge (normalized to Pier# 2)

4.3 Pushover Analysis

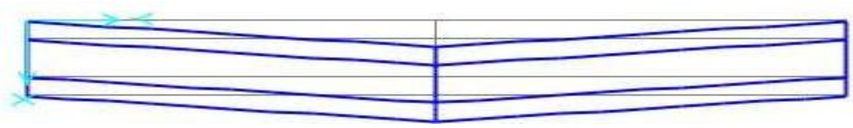
Pushover analysis is carried out using FEMA 356 displacement coefficient method, ATC 40 capacity spectrum method, FEMA 440 equivalent linearization method (modified CSM) as well as FEMA 440 displacement modification method (Improvement for DCM). A triangular load pattern was used for standard pushover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



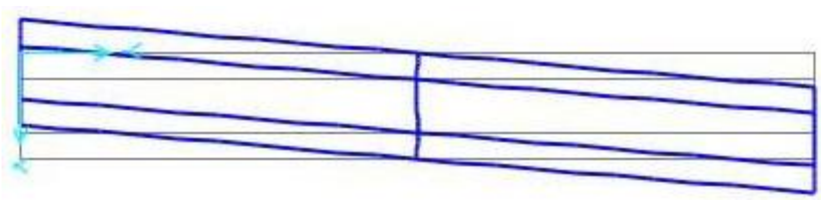
(a) first mode



(b) second mode



(c) third mode



(d) fourth mode

Fig. 4.2: First four modes of the bridge (plan view)

4.3.1 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

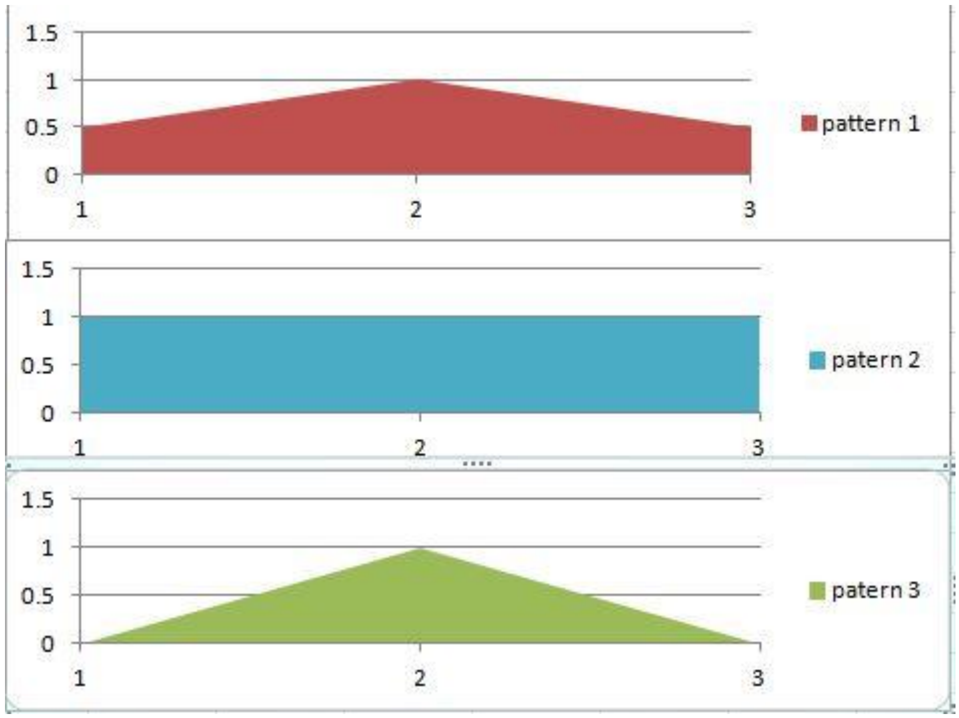


Fig. 4.3: Different lateral load pattern used

4.3.2 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. The basic of capacity curve is already discussed in Chapter 2.

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.

It is established in the various literature reviews that load pattern based on inertial mass at different node 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.

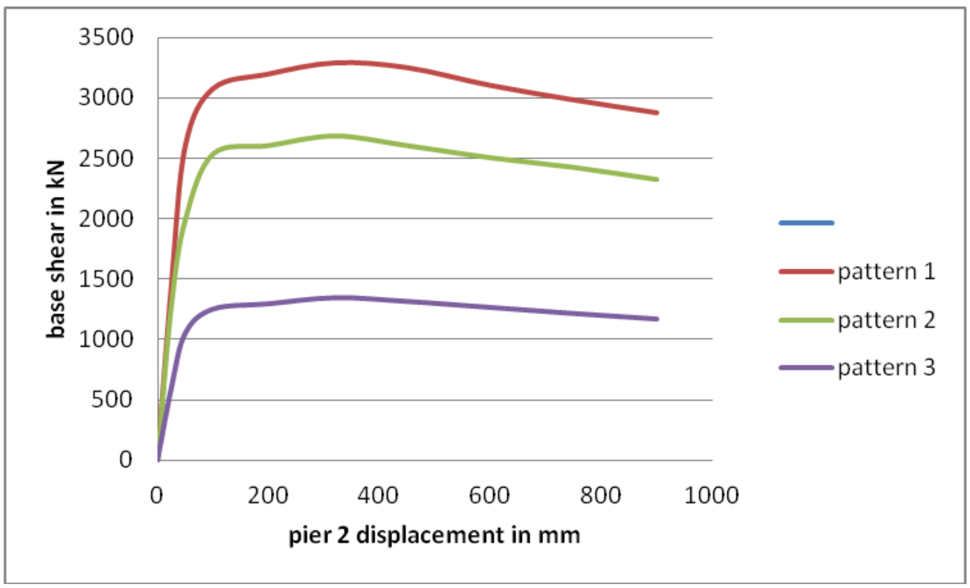


Fig. 4.4: Capacity curve of the bridge NWBR S30M

4.3.2.1 Capacity Curve for Displacement Coefficient Method

Basics of the method are already discussed in chapter1 and chapter2. 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 of S_s and S_l (spectral acceleration at short and long periods) is calculated as 2.5g and 1.36g 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 fig 4.5.

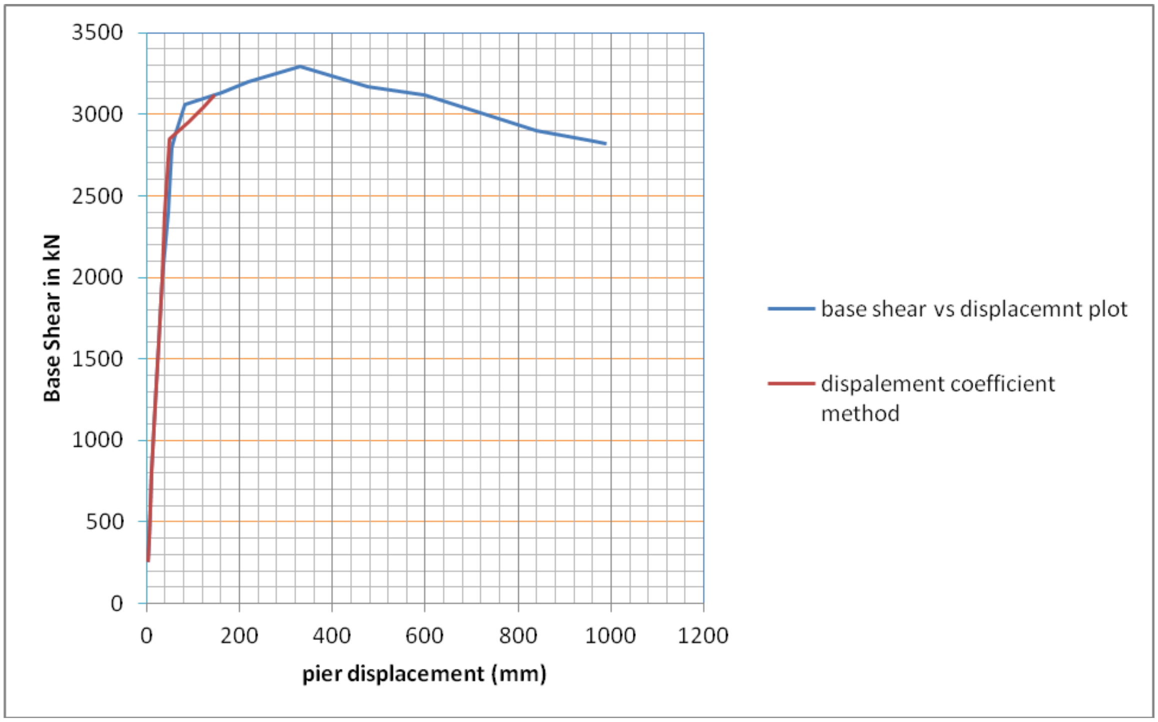


Fig. 4.5: Capacity curve of the bridge NWBR S30M by DCM

4.3.2.2 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 Ca and Cv are determined by comparing the response spectra curves for ATC-40 and IS code. The values of Ca and Cv are taken as 0.18 and 0.245 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 fig 4.6.

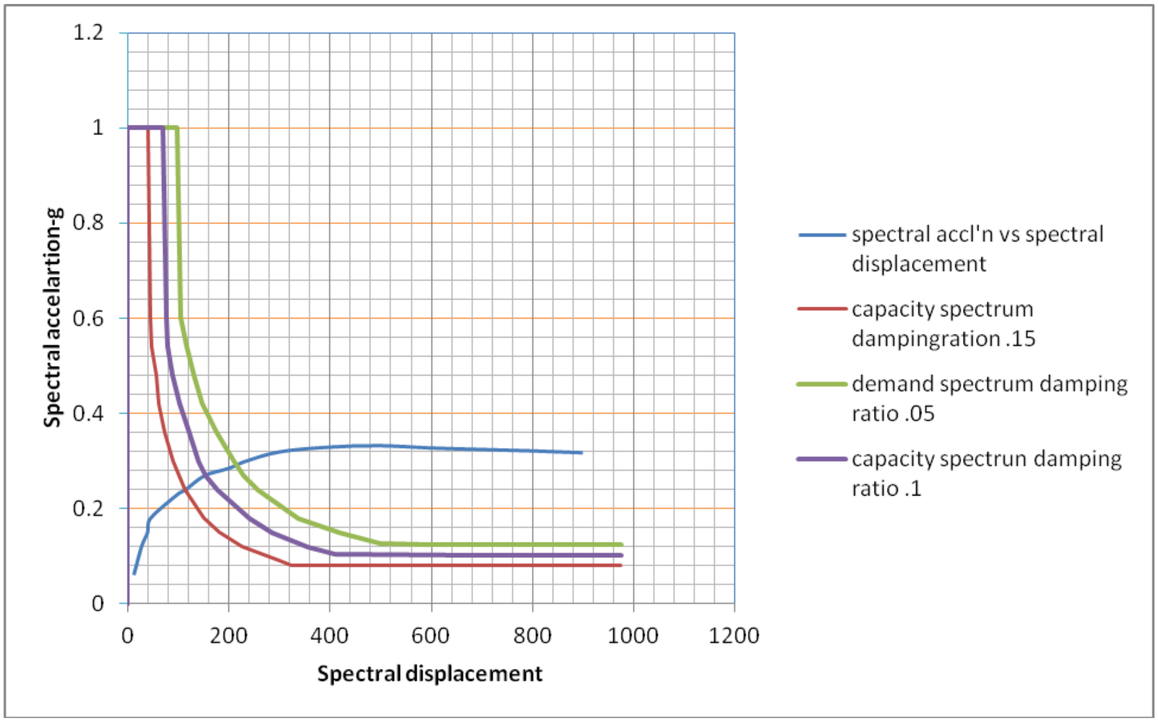


Fig. 4.6: Capacity curve of the bridge NWBR S30M by CSM

4.3.2.3 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. Teff and Beff 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 fig 4.7. Also showing the values of Sa, Sd, Teff, Beff, ductility ratio along with base shear and pier top displacement at performance point.

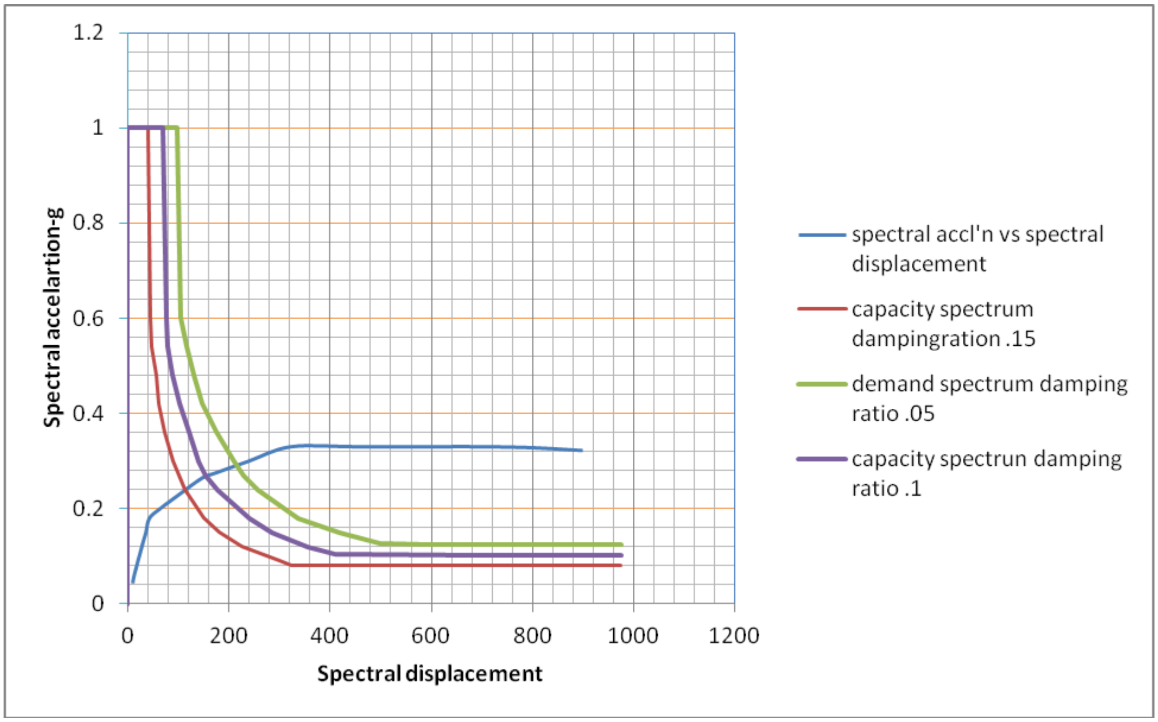


Fig. 4.7: Capacity curve of the bridge NWBR S30M by ELM

4.3.2.4 Capacity Curve for Displacement Modification Method

This method is an improvement over displacement coefficient method (FEMA356). Demand spectrum parameters, site class Ss and Sl are same 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 fig 4.8.

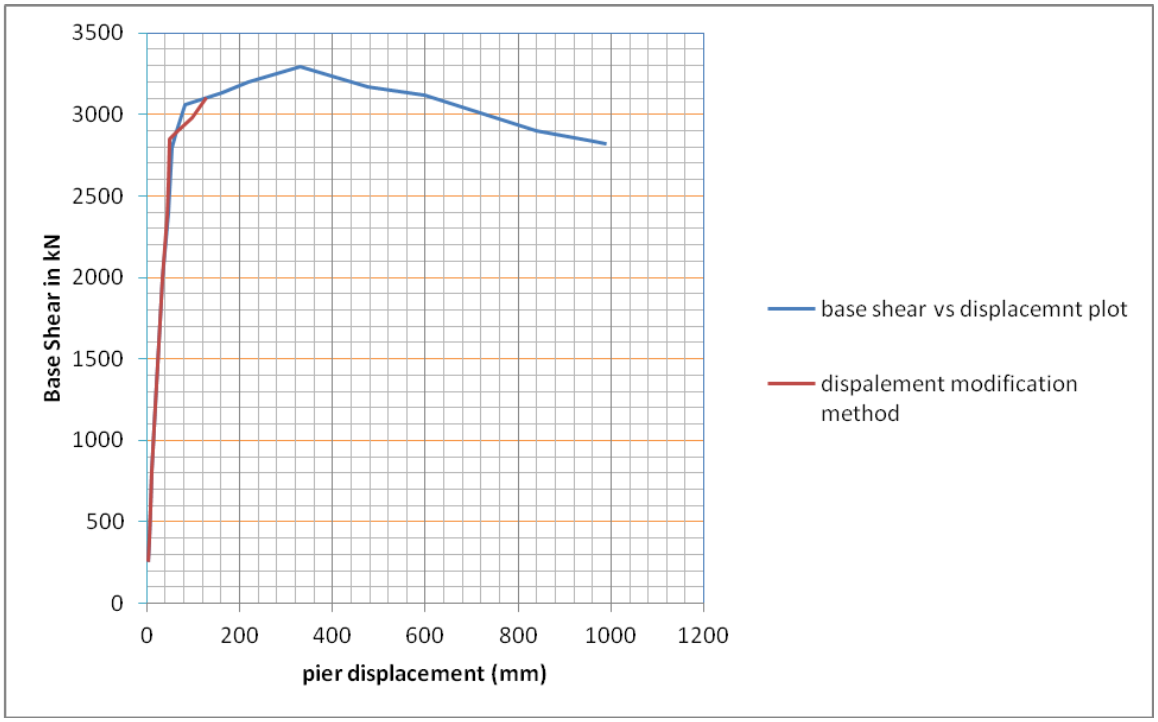


Fig. 4.8: Capacity curve of the bridge NWBR S30M by DMM

4.3.3 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 discussed in Chapter1&2. Table 4.3 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).

It is seen that base shear from all the methods is in similar range. While 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). Similar trends were seen in the results of the other models also, that are discussed below.

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

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

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 (series1 varying pier height and series2 varying span) are provided in table 4.3 and table 4.4 respectively.

In case of series1 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.

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 4.3: 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 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

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

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.

Base shear as well as displacement trends for series2 is completely different from series1. 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.

4.4 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 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 detail of provisions used in the analysis given in the aforementioned codes is discussed in chapter2.

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 4.5, fig 4.9 and fig 4.10. The shear demand values obtained for linear static method are factored 1.5 times to reach codal demand.

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

Table 4.5: Comparison of result of pushover analysis and linear static analysis

Bi, = Base shear for bridge by ESLM
Vi= max shear demand for critical pier by ESLM
Bp= Base shear for bridge by NSP
Vp= max shear demand for critical pier by NSP

The comparison of base shear bridges shows that pushover demand is very high against codal seismic demand for all the model bridges. The 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. These large variations are highlighted in fig 4.9 and 4.10

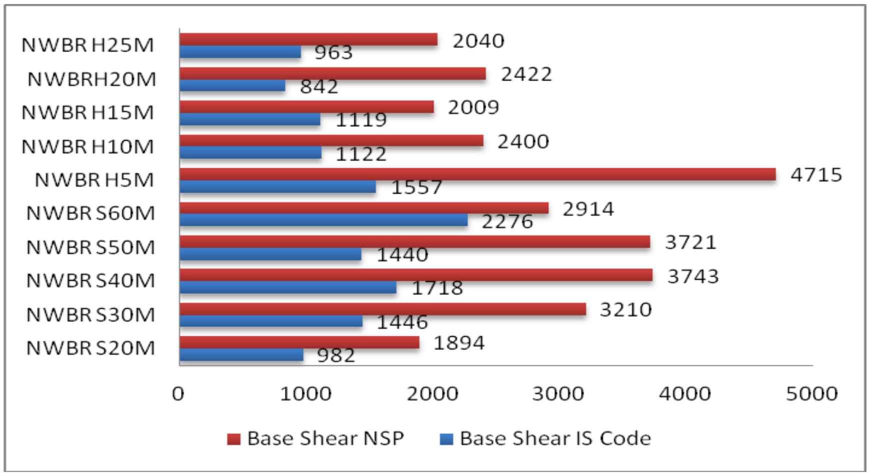


Fig. 4.9: Base shear comparison between NSP and LSM

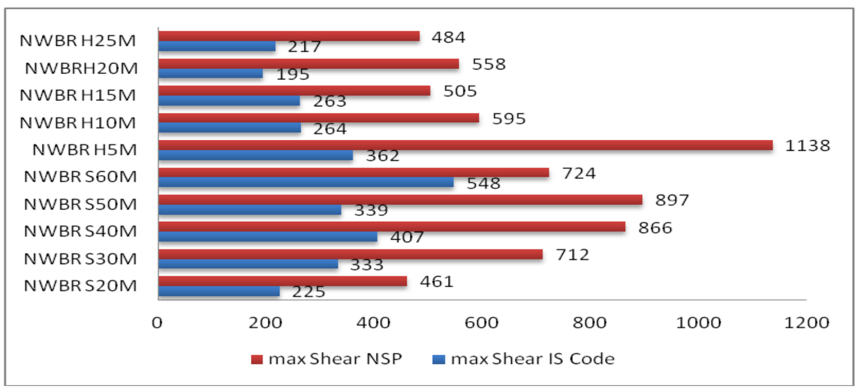


Fig. 4.10: Max shear comparison between NSP and LSM

CHAPTER 5

SUMMARY AND CONCLUSIONS

5.1 Summary

The bridge design codes, in India, have included seismic design provision at present. But, a large number of bridges were designed and constructed without considering seismic forces. Therefore, it is very important to evaluate the capacity of existing bridges against seismic force demand. There are many literatures available on the seismic evaluation procedures of multi-storey buildings using nonlinear static (pushover) analysis. There is no much effort available in literature for seismic evaluation of existing bridges although bridge is a very important structure in any country. There are presently no comprehensive guidelines to assist the practicing structural engineer to evaluate existing bridges and suggest design and retrofit schemes. Further there is a need to include non linear static linear pushover analysis in the seismic analysis due to its advantages and applicability to existing structures as well as new construction. With continuous advancement and advent of software technology, pushover approach has been adopted internationally. This study aimed at applying latest pushover analysis method to bridges with required modification to suit bridge analysis and compare the analysis results with design demands as per existing Indian Codes.

In order to achieve it two series of bridge models, with different span and pier height are modeled using SAP2000 v18.0.1 for nonlinear analysis. Nonlinear hinge properties were generated using improved stress-strain curve of concrete and reinforcing steel. The bridges are analyzed using procedures as per Displacement Coefficient Method (FEMA 356), Capacity Spectrum Method (ATC 40), Displacement Modification Method (FEMA 440) and Equivalent Linearization Method (FEMA 440). These procedures are suitably modified to use for multi-span bridges.

5.2 Conclusions

Bridges extends horizontally with its two ends restrained and that makes the dynamic characteristics of bridges different 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. The Trapezoidal Pattern, which is based on lateral forces that are proportional to the total mass assigned to each node, estimates a very high base-shear capacity of the bridge in transverse direction as compared to other load pattern.

ii. 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%. These values are permissible as per FEMA 440 guidelines.

iii. 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).

iv. 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.

v. The Target deflection in transverse direction for longest span bridge is 106mm and highest bridge is 83mm.

vi. 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.

vii. Possibility of plastic hinge formation in an extreme seismic event is not accounted for in the design procedure outlined in IRC codes; capacity design is not performed.

viii. 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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