APPLICATION OF PUSHOVER ANALYSIS TO RC BRIDGES

A Project Report

Submitted by

KALIPRASANNA SETHY

In partial fulfilment of the requirements for

the award of Degree of

MASTER OF TECHNOLOGY

In

STRUCTURAL ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING
NATIONAL INSTITUTE OF TECHNOLOGY, ROURKELA
ORISSA – 769 008

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Under the guidance of
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2011

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CERTIFICATE

This is to certify that the thesis entitled **"APPLICATION OF PUSHOVER ANALYSIS TO RC BRIDGES"** submitted by **Mr. Kaliprasanna Sethy** in partial fulfillment of the requirements for the award of Master of Technology Degree in Civil Engineering with specialization in Structural Engineering at the National Institute of Technology Rourkela is an authentic work carried out by him under my supervision.

To the best of my knowledge, the matter embodied in the thesis has not been submitted to any other University/Institute for the award of any degree or diploma.

Date: June 02, 2011

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Kaliprasanna Sethy

ABSTRACT

Keywords: *pushover analysis, RC Bridge, plastic hinge, target displacement, capacity curve*

After 2001 Gujarat Earthquake and 2005 Kashmir Earthquake, there is a nation-wide attention to the seismic vulnerability assessment of existing buildings. There are many literatures available on the seismic evaluation procedures of multi-storeyed 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. In order to address this problem, the aims of the present project was to carry out a seismic evaluation case study for an existing RC bridge using nonlinear static (pushover) analysis.

Bridges extends horizontally with its two ends restrained and that makes the dynamic characteristics of bridges different from building. Modal analysis of a 3D bridge model reveals that it has many closely-spaced modes. Participating mass ratio for the higher modes is very high. Therefore, pushover analysis with single load pattern may not yield correct results for a bridge model.

A 12-span existing RC bridge was selected for the case study. Standard pushover analysis using FEMA 356 (2000) displacement coefficient method and an improved upper bound pushover analysis method were used to analyse the building. Some of the analysis parameters were suitably modified to use in a bridge structure. The evaluation results presented here shows that the selected bridge does not have the capacity to meet any of the desired performance level.

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NOTATIONS

English Symbols

c	-	classical damping
C_0	-	factor for MDOF displacement
C_1	-	factor for inelastic displacement
C_2	-	factor for strength and stiffness degradation
C_3	-	factor for geometric nonlinearity
E_c	-	short-term modulus of elasticity of concrete
E_D	-	energy dissipated by damping
E_s	-	modulus of elasticity of steel rebar
E_S	-	maximum strain energy
EI	-	flexural rigidity of beam
f_{ck}	-	characteristic compressive strength of concrete
$\{f_s(t)\}$	-	lateral load vector
$\{f_{s,UB}\}$	-	force vector in upper bound pushover analysis
f_y	-	yield stress of steel rebar
F_y	-	defines the yield strength capacity of the SDOF
K_{eq}	-	equivalent stiffness
K_i	-	initial stiffness
m	-	storey mass
M_n^*	-	modal mass for n^{th} mode
N	-	number of modes considered
$P_{eff}(t)$	-	effective earthquake force
$q_n(t)$	-	the modal coordinate for n^{th} mode

R	-	normalized lateral strength ratio
S_a	-	spectral acceleration
S_d	-	spectral displacement
$\{s_n\}$	-	nth mode contribution in $\{s\}$
SR_A	-	spectral reduction factor at constant acceleration region
SR_V	-	spectral reduction factor at constant velocity region
T	-	fundamental natural period of vibration
T_{eq}	-	equivalent time period
T_i	-	initial elastic period of the structure
T_n	-	n^{th} mode natural period
$\{u\}$	-	floor displacements relative to the ground
$\ddot{u}_g(t)$	-	earthquake ground acceleration
$u_{n,roof}(t)$	-	displacement at the roof due to n^{th} mode
$u_{no,roof}$	-	peak value of the roof displacement due to n^{th} mode
$u_{roof}(t)$	-	roof displacement at time 't'
$u_{r,UB}$	-	target roof displacement in upper bound pushover analysis
V_c	-	shear strength of the concrete in RC section
V_y	-	yield shear strength

Greek Symbols

α	-	post-yield stiffness ratio
β_{eq}	-	equivalent damping
β_i	-	initial elastic damping
β_s	-	damping due to structural yielding
δ_t	-	target displacement

Δ	-	shear deformation
$\{\phi_n\}$	-	n^{th} mode shape of the structure
$\phi_{n,roof}$	-	value of the n^{th} mode shape at roof
φ_u	-	ultimate curvature
φ_y	-	yield curvature
Γ_1	-	1^{st} mode participation factor for the stepped frame
Γ_{ref}	-	1^{st} mode participation factor for the regular building without steps
μ	-	displacement ductility ratio
θ_y	-	yield rotation
ω_n	-	n^{th} mode natural frequency
ξ_n	-	n^{th} mode damping ratio
Γ_n	-	modal participation factor of the n^{th} mode

ABBREVIATION

2-D	Two Dimensional
3-D	Three Dimensional
ACI	American Concrete Institute
ADRS	Acceleration-Displacement Response Spectrum
ATC	Applied Technology Council
BS	British Standards
CP	Collapse Prevention
CSM	Capacity Spectrum Method
DCM	Displacement Coefficient Method
FEM	Finite Element Method
FEMA	Federal Emergency Management Agency
FSM	Finite Strip Method
IO	Immediate Occupancy
IS	Indian Standard
LS	Life Safety
MDOF	Multi Degree of Freedom
MPA	Modal Pushover Analysis
PGA	Peak Ground Acceleration
RC	Reinforcement Concrete

SAP	Structural Analysis Program
SDOF	Single Degree of Freedom
SPA	Standard Pushover Analysis
TLP	Triangular Load Pattern
UBPA	Upper Bound Pushover Analysis

CHAPTER-1

INTRODUCTION

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 north-eastern 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 2002. The magnitudes of the design seismic forces have been considerably enhanced in general, and the seismic zonation 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-storeyed 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 no seismic design provision at present. A large number of bridges are 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 case study for an existing RC bridge 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-storeyed building. So, in the present study an improved pushover analysis is also used to verify the results.

1.2 OBJECTIVES

Following are the main objectives of the present study:

a) To understand the standard pushover analysis procedures and other improved pushover analysis procedures available in literature.
b) To carry out a detailed case study of pushover analysis of a reinforced concrete bridge using standard pushover analysis and other improved pushover analyses.

1.3 METHODOLOGY

a) A thorough literature review to understand the seismic evaluation of building structures and application of pushover analysis.
b) Select an existing RC bridge with geometrical and structural details
c) Model the selected bridge in computer software SAP2000.
d) Carry out modal analysis to obtain the dynamic properties of the bridges and generate input parameters for pushover analysis from the modal properties of the bridge.
e) Carry out pushover analysis of the bridge model and arrive at a conclusion.

1.4 ORGANISATION OF THESIS

This introductory chapter presents the background; objectives and methodology of the project. The first part of Chapter 2 discusses details about pushover analysis procedures as per FEMA 356 and different improvements of this procedure available in literature. The second part of this chapter presents previous researches on seismic evaluation of RC bridges. Chapter 3 presents different issues on the bridge modelling including nonlinear hinge model used for pushover analysis. Chapter 4 presents the analysis results and different interpretations of the results. Finally, in Chapter 5 the summary and conclusions are given.

CHAPTER-2

LITERATURE REVIEW

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

The available literatures on pushover analysis of RC bridges are very limited whereas we can get a number of published literatures in pushover analysis of buildings. Hence the literature survey is presented here in two broad areas: (i) standard pushover analysis and its improvements and (ii) application of pushover analysis to bridges.

2.2 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 recognised 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.

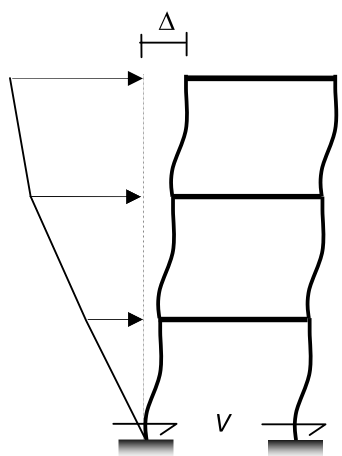
Pushover analysis is defined as an analysis wherein a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building shall be subjected to monotonically increasing lateral loads representing inertia forces in an earthquake 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 component 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 summarised as follows:

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

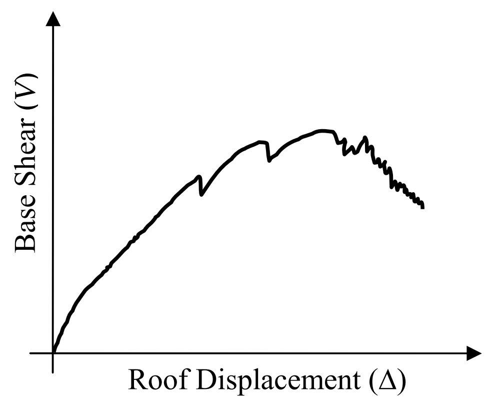
Pushover analysis delivers all these benefits for an additional computational effort (modelling nonlinearity and change in analysis algorithm) over the linear static analysis. Step by step procedure of pushover analysis is discussed next.

2.2.1 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. 2.1a). 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 (Fig. 2.1b).



a) Building model



b) Pushover curve

Fig. 2.1: 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 as explained in Section 2.2.3. 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 bi-directional 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 in Section 2.2.2. 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 behaviour.

2.2.2 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 behaviour 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).

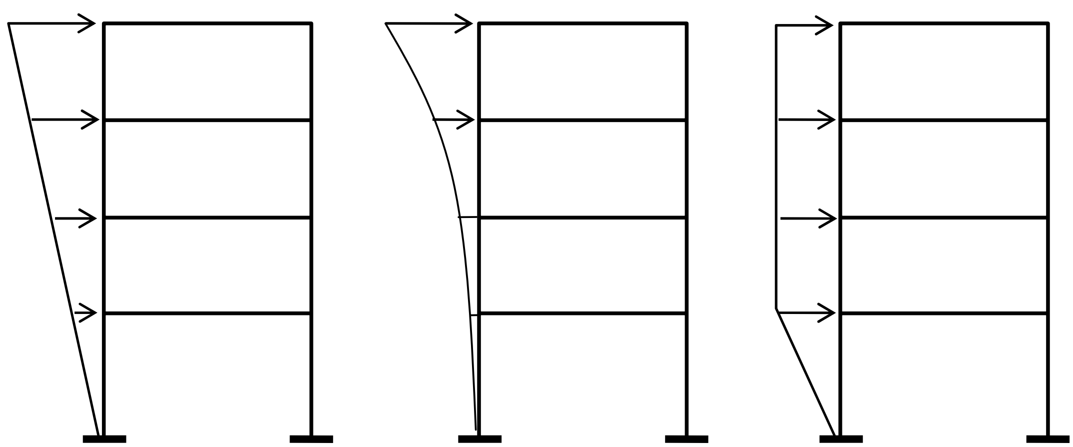
iii) A vertical distribution proportional to the story shear distribution calculated by combining modal responses from a response spectrum analysis of the building (sufficient number of modes to capture at least 90% of the total building mass required to be considered). This distribution shall be used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



(a) Triangular (b) IS Code Based (c) Uniform

Fig. 2.2: Lateral load pattern for pushover analysis as per FEMA 356 (considering uniform mass distribution)

2.2.3 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:

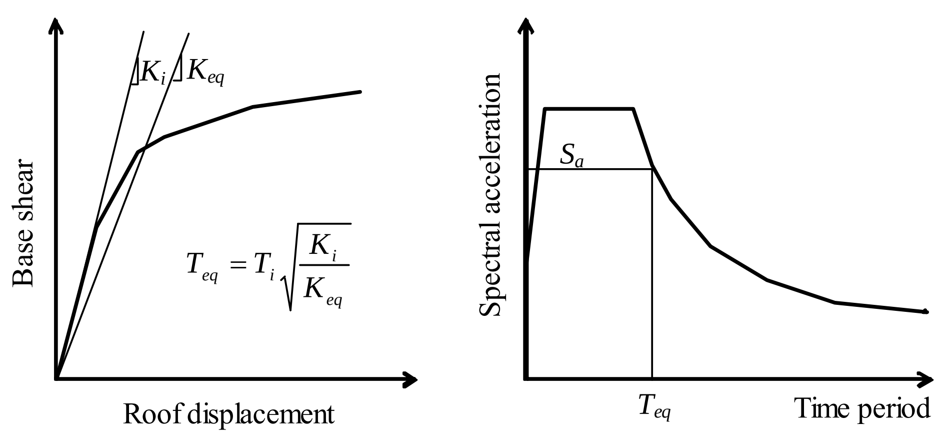
(a) Displacement Coefficient Method (DCM) of FEMA 356 and

(b) Capacity Spectrum Method (CSM) of ATC 40.

Both of these approaches 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.

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. 2.3: 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. 2.3a. An equivalent period (T_{eq}) is generated from initial period (T_i) 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. 2.3b).

$$S_d = \frac{T_{eq}^2}{4\pi^2} S_a \tag{2.1}$$

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

$$\delta_t = C_0 C_1 C_2 C_3 S_d = C_0 C_1 C_2 C_3 \frac{T_{eq}^2}{4\pi^2} S_a \tag{2.2}$$

C_0 = 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.

C_1 = the ratio of expected displacement (elastic plus inelastic) for an inelastic system to the displacement of a linear system.

C_2 = a factor that accounts for the effect of pinching in load deformation relationship due to strength and stiffness degradation

C_3 = 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.

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).

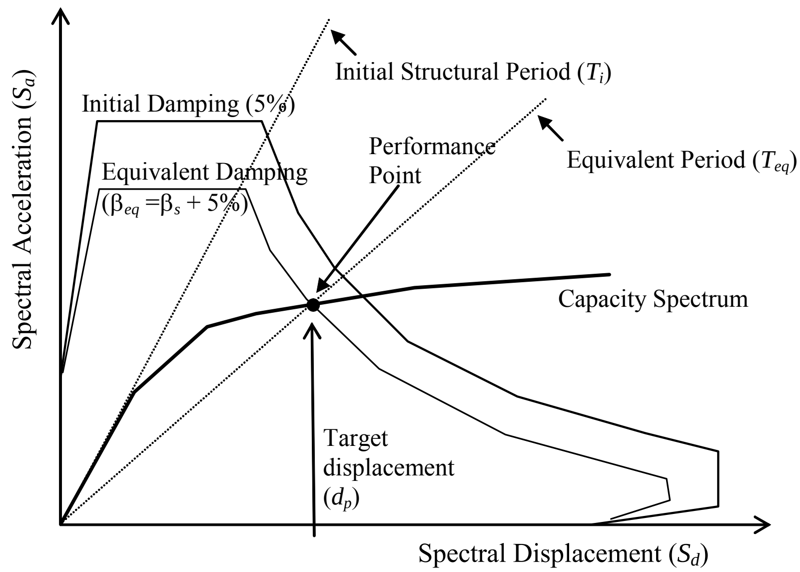


Fig. 2.4: Schematic representation of Capacity Spectrum Method (ATC 40)

The equivalent period (T_{eq}) is computed from the initial period of vibration (T_i) 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 (T_{eq}) and equivalent damping (β_{eq}).

$$T_{eq} = T_i \sqrt{\frac{\mu}{1 + \alpha\mu - \alpha}} \tag{2.3}$$

$$\beta_{eq} = \beta_i + \kappa \frac{2}{\pi} \frac{(\mu - 1)(1 - \alpha)}{\mu(1 + \alpha\mu - \alpha)} = 0.05 + \kappa \frac{2}{\pi} \frac{(\mu - 1)(1 - \alpha)}{\mu(1 + \alpha\mu - \alpha)} \tag{2.4}$$

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.

ATC 40 relates effective damping to the hysteresis curve (Fig. 2.5) and proposes three hysteretic behavior types that alter the equivalent damping level. Type A hysteretic behavior is meant for new structures with reasonably full hysteretic loops, and the corresponding equivalent damping ratios take the maximum values. Type C hysteretic behavior represents severely degraded hysteretic loops, resulting in the smallest equivalent damping ratios. Type B hysteretic behavior is an intermediate hysteretic behavior between types A and C. The value of κ decreases for degrading systems (hysteretic behavior types B and C).

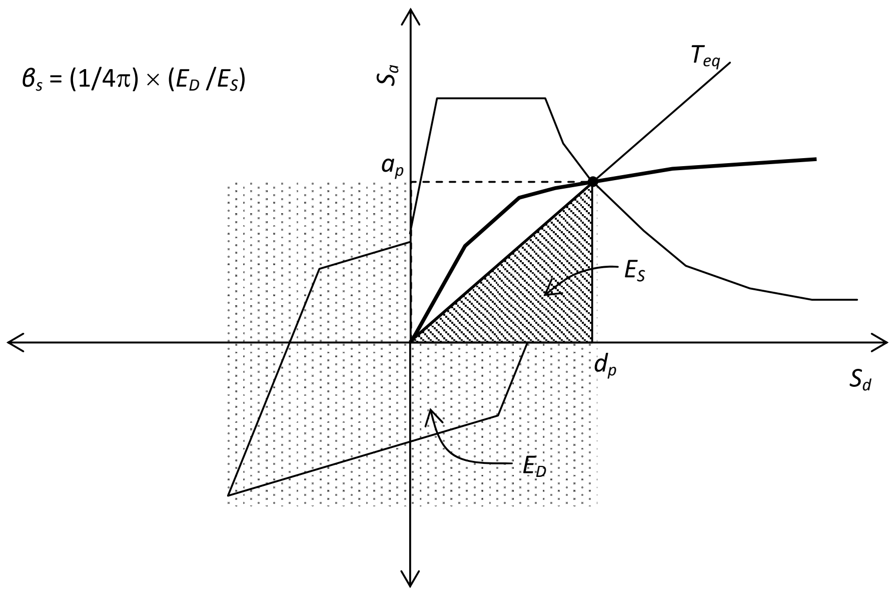


Fig. 2.5: Effective damping in Capacity Spectrum Method (ATC 40)

The equivalent period in Eq. 2.3 is based on a lateral stiffness of the equivalent system that is equal to the secant stiffness at the target displacement. This equation does not depend on the degrading characteristics of the hysteretic behavior of the system. It only depends on the displacement ductility ratio (μ) and the post-yield stiffness ratio (α) of the inelastic system.

ATC 40 provides reduction factors to reduce spectral ordinates in the constant acceleration region and constant velocity region as a function of the effective damping ratio. The spectral reduction factors are given by:

$$SR_A = \frac{3.21 - 0.68\ln(100\beta_{eq})}{2.12} \tag{2.5}$$

$$SR_V = \frac{2.31 - 0.41\ln(100\beta_{eq})}{1.65} \tag{2.6}$$

where β_{eq} is the equivalent damping ratio, SR_A is the spectral reduction factor to be applied to the constant acceleration region, and SR_V is the spectral reduction factor to be applied to the constant velocity region (descending branch) in the linear elastic spectrum.

Since the equivalent period and equivalent damping are both functions of the displacement ductility ratio (Eq. 2.3 and Eq. 2.4), it is required to have prior knowledge of displacement ductility ratio. However, this is not known at the time of evaluating a structure. Therefore, iteration is required to determine target displacement. ATC 40 describes three iterative procedures with different merits and demerits to reach the solution.

2.3 SHORT COMINGS OF STANDARD PUSHOVER ANALYSIS

Pushover analysis is a very effective alternative to nonlinear dynamic analysis, but it is an approximate method. Major approximations lie in the choice of the lateral load pattern and in the calculation of target displacement. FEMA 356 guideline for load pattern does not cover all possible cases. It is applicable only to those cases where the fundamental mode participation is predominant. Both the methods to calculate target displacement (given in FEMA 356 and ATC 40) do not consider the higher mode participation. Also, it has been assumed that the response of a MDOF system is directly proportional to that of a SDOF system. This approximation is likely to yield adequate predictions of the element deformation demands for low to medium-rise buildings, where the behaviour is dominated by a single mode. However, pushover analysis can be grossly inaccurate for buildings with irregularity, where the contributions from higher modes are significant.

Many publications (Aschheim, *et. al*., 1998; Chopra and Chintanapakdee, 2001; Chopra and Goel, 1999; Chopra and Goel, 2000; Chopra, *et. al*., 2003; Dinh and Ichinose, 2005; Fajfar, 2000; Goel and Chopra, 2004; Gupta and Krawinkler, 2000; Kalkan and Kunnath, 2007; Moghadam and Hajirasouliha, 2006; Mwafy and Elnashai, 2000; Mwafy and Elnashai, 2001; Krawinkler and Seneviratna, 1998) have demonstrated that traditional pushover analysis can be an extremely useful tool, if used with caution and acute engineering judgment, but it also exhibits significant shortcomings and limitations, which are summarised below:

a) One important assumption behind pushover analysis is that the response of a MDOF structure is directly related to an equivalent SDOF system. Although in several cases the response is dominated by the fundamental mode, this cannot be generalised. Moreover, the shape of the fundamental mode itself may vary significantly in nonlinear structures depending on the level of inelasticity and the location of damages.

b) Target displacement estimated from pushover analysis may be inaccurate for structures where higher mode effects are significant. The method, as prescribed in FEMA 356, ignores the contribution of the higher modes to the total response.

c) It is difficult to model three-dimensional and torsional effects. Pushover analysis is very well established and has been extensively used with 2-D models. However, little work has been carried out for problems that apply specifically to asymmetric 3-D systems, with stiffness or mass irregularities. It is not clear how to derive the load distributions and how to calculate the target displacement for the different frames of an asymmetric building. Moreover, there is no consensus regarding the application of the lateral force in one or both horizontal directions for such buildings.

d) The progressive stiffness degradation that occurs during the cyclic nonlinear earthquake loading of the structure is not considered in the present procedure. This degradation leads to changes in the periods and the modal characteristics of the structure that affect the loading attracted during earthquake ground motion.

e) Only horizontal earthquake load is considered in the current procedure. The vertical component of the earthquake loading is ignored; this can be of importance in some cases. There is no clear idea on how to combine pushover analysis with actions at every nonlinear step that account for the vertical ground motion.

f) Structural capacity and seismic demand are considered independent in the current method. This is incorrect, as the inelastic structural response is load-path dependent and the structural capacity is always associated with the seismic demand.

2.4 ALTERNATE PUSHOVER ANALYSIS PROCEDURES

As discussed in the previous Section, pushover analysis lacks many important features of nonlinear dynamic analysis and it will never be a substitute for nonlinear dynamic analysis as the most accurate tool for structural analysis and assessment. Nevertheless, several possible developments can considerably improve the efficiency of the method. There are several attempts available in the literature to overcome the limitations of this analysis. These include the use of alternative lateral load patterns, use of higher mode properties and use of adaptive procedures.

Modal Pushover Analysis (MPA), developed by Chopra and Goel (2002), is an improved procedure to calculate target displacement. Recent research shows that this procedure is capable of analysing buildings with plan asymmetry (Chopra and Goel, 2004) and some forms of vertical irregularity (Chintanapakdee and Chopra, 2004). However, a recent paper (Tjhin et. al., 2006) concludes that the scope of the applicability of multimode pushover analysis is not very wide and should be used with caution when analysing a particular category of buildings. Park et. al. (2007) presents a new modal combination rule (factored modal combination) to estimate the load profile for pushover analysis. This combination is found to work for frames with vertical irregularities (soft ground story and vertical mass irregularity)

Although the Modal Pushover Analysis (MPA) procedure explained in the previous paragraph estimates seismic demands more accurately than current pushover procedures used in structural engineering practice (Goel and Chopra 2004, Chopra and Chintanapakdee 2004), it requires multiple runs to arrive at the solution. Modified Modal Pushover Analysis, proposed by Chopra and Goel (2002), reduces the computational effort in MPA by simplifying the computation of the response contributions of higher modes by assuming the building to be linearly elastic.

To include the higher mode effects, this procedure suggests (Jan *et. al.*, 2004) a new load pattern to carryout pushover analysis. This is based on an upper-bound (absolute sum) modal combination rule. This can be explained from the fundamental structural dynamics theory. The following Section presents this upper-bound pushover analysis (UBPA) procedure in detail.

2.4.1 Upper-Bound Pushover Analysis

This procedure is developed based on the differential equations governing the response of a multi-story building subjected to an earthquake ground motion with acceleration, $\ddot{u}_g(t)$:

$$[m]\{\ddot{u}\}+[c]\{\dot{u}\}+[k]\{u\}=-[m]\{1\}\ddot{u}_g(t) \tag{2.7}$$

where $\{u\}$ is the floor displacements relative to the ground, $[m]$, $[c]$, and $[k]$ are the mass, classical damping, and lateral stiffness matrices of the system.

If we look at the solution of the differential equation (Eq. 2.7) governing the response of a MDOF system to an earthquake ground motion:

$$\{u(t)\}=\sum_{n=1}^{N}\{\phi_n\}q_n(t) \tag{2.8}$$

Now, the equivalent static forces can be expressed as:

$$\{f_s(t)\}=[k]\{u(t)\}=\sum_{n=1}^{N}[k]\{\phi_n\}q_n(t)=\sum_{n=1}^{N}\omega_n^2[m]\{\phi_n\}q_n(t) \tag{2.9}$$

At any instant of time *t,* these forces $\{f_s(t)\}$ are the external forces that produce the displacements $\{u(t)\}$ at the same time *t* and the roof displacement at time *t* due to the forces $\{f_s(t)\}$, $u_{roof}(t)$ can be expressed in the following form:

$$u_{roof}(t)=\sum_{n=1}^{N}\phi_{n,roof}\,q_n(t)=u_{1,roof}(t)\left[1+\sum_{n=2}^{N}\frac{\phi_{n,roof}\,q_n(t)}{\phi_{1,roof}\,q_1(t)}\right] \tag{2.10}$$

where $u_{1,roof}(t)=\phi_{1,roof}\,q_1(t)$, representing the roof displacement due to the first mode. If $\{\phi_n\}$ is normalized such that its value at the roof $\phi_{n,roof}$ = 1, then Eq. 2.10 can be simplified as

$$u_{roof}(t)=u_{1,roof}(t)\left[1+\sum_{n=2}^{N}\frac{q_n(t)}{q_1(t)}\right] \tag{2.11}$$

where $\sum_{n=2}^{N}\frac{q_n(t)}{q_1(t)}=\sum_{n=2}^{N}\frac{\Gamma_n D_n(t)}{\Gamma_1 D_1(t)}$, which is a combination of the displacement–response contribution ratio of all higher modes to that of the fundamental mode.

With this background, Jan *et. al*. (2004) explained that the first two modes alone provide a reasonably accurate prediction for the structural response to earthquakes, and the third or higher mode can be ignored. Thus, the authors assumed that the displacement response is mainly controlled by the first two modes, and choose the absolute sum (ABSSUM) modal combination rule to determine peak response, Eq. 2.9 and Eq. 2.11 can be reduced to

$$\{f_s\} = \omega_1^2[m]\{\phi_1\}q_1 + \omega_2^2[m]\{\phi_2\}q_2 = q_1\left[\omega_1^2[m]\{\phi_1\} + \omega_2^2[m]\{\phi_2\}\frac{q_2}{q_1}\right] \tag{2.12}$$

$$u_{roof}(t) = u_{1,roof}\left[1 + \frac{\Gamma_2 D_2}{\Gamma_1 D_1}\right] \tag{2.13}$$

Since $\{f_s\}$ is a spatial vector and increases monotonically from zero, Eq. 2.12 can be simply expressed as

$$\{f_s\} = \omega_1^2[m]\{\phi_1\} + \omega_2^2[m]\{\phi_2\}\frac{q_2}{q_1} \tag{2.14}$$

In Eq. 2.13, $u_{1,roof}$ is the roof displacement contributed by only the 1st mode which can be approximately taken as the target displacement as defined by FEMA 356 for simplicity.

$$u_{roof}(t) = \delta_t\left[1 + \frac{\Gamma_2 D_2}{\Gamma_1 D_1}\right] \tag{2.15}$$

where δ_t is the target displacement calculated as per FEMA 356 (Eq. 2.2)

The principle steps of upper-bound pushover analysis procedure are as follows:

i. Perform an eigen value analysis and find out the natural periods and mode shapes of the structure. Normalize the mode shape $\{\phi_n\}$ such that its value at the roof, $\phi_{n,roof} = 1$ for all the modes.

ii. Use the elastic response spectrum of the selected earthquake to determine the upper-bound of the 2nd mode contribution ratio, $(q_2/q_1)_{UB}$, as given by the following expression:

$$\left(\frac{q_2}{q_1}\right)_{UB} = \left|\frac{\Gamma_2 D_2}{\Gamma_1 D_1}\right|$$

where Γ_n (n = 1, 2) is the modal participation factor and D_n (n = 1, 2) is the displacement obtained from the elastic displacement response spectrum for n'th mode.

iii. Determine the lateral load distribution (height-wise) for pushover analysis using the following formula:

$$\{f_{s,UB}\} = \omega_1^2[m]\{\phi_1\} + \omega_2^2[m]\{\phi_2\}\left(\frac{q_2}{q_1}\right)_{UB}$$

where ω_n (n = 1, 2) is the natural frequency for the nth-mode.

iv. Determine the target roof displacement $u_{roof,UB}$ as given by the following relationship:

$$u_{roof,UB} = \delta_t\left[1 + (q_2/q_1)_{UB}\right]$$

where δ_t is the target displacement predicted by the pushover analysis as per FEMA 356.

v. The seismic demands of a given structure are determined by pushover analysis with a lateral load profile $\{f_{s,UB}\}$, and the forces are monotonically increased until the target displacement $u_{roof,UB}$ is reached or a collapse mechanism developed.

2.5 APPLICATION OF PUSHOVER ANALYSIS TO RC BRIDGES

Chiorean (2003) evaluated a nonlinear static (pushover) analysis method for reinforced concrete bridges that predicts behaviour at all stages of loading, from the initial application of loads up to and beyond the collapse condition. The author developed a line elements approach, which are based on the degree of refinement in representing the plastic yielding effects. The method has been developed for the purpose of investigating the collapse behaviour of a three span pre-stressed reinforced concrete bridge of 115m in total length.

Au, *et. al*. (2001) evaluated vibration analysis of bridges under moving vehicles. The authors reported that, vehicle-bridge interaction is a complex dynamic phenomenon, depending on many parameters which include the type of bridge and its natural frequencies of vibration, vehicle characteristics, vehicle speed, the number of vehicles and their relative positions on the bridge, roadway surface irregularities, etc. The authors finalized the interaction between the moving vehicles and the bridge is a nonlinear problem. And FEM is certainly the most versatile and powerful method, while FSM is particularly suitable for regular plate-type bridges.

Pinho, *et. al*. (2007) performed a pushover analysis subjecting the structure to monotonically increasing lateral forces with invariant distribution until a target displacement is reached. A pushover analysis of continuous multi-span bridge is carried out. The authors mentioned that, with respect to conventional pushover methods, these novel single-run approaches can lead to the attainment of improved predictions.

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. 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.

2.6 SUMMARY

This Chapter describes details procedure of standard pushover analysis as per FEMA 356 and ATC 40. This procedure, as explained in FEMA 356, is primarily meant for regular buildings with dominant fundamental mode participation. There are many alternative approaches of pushover analysis reported in the literature to make it applicable for different categories of irregular buildings. These comprise (i) *modal pushover analysis* (Chopra and Goel, 2001), (ii) *modified modal pushover analysis* (Chopra *et. al*., 2004), (iii) *upper bound pushover analysis* (Jan *et. al*., 2004), and (iv) *adaptive pushover analysis*, etc. However, none of these alternative methods have been tested for RC Bridges successfully. This second half of this chapter presents the previous research work available in literature on the seismic evaluation of RC bridges.

CHAPTER-3

STRUCTURAL MODELLING

CHAPTER 3

STRUCTURAL MODELLING

3.1 INTRODUCTION

The study in this thesis 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 modelling of the nonlinear properties of various structural elements is very important in nonlinear analysis. In the present study, piers were modelled with inelastic flexural deformations using point plastic model. This chapter also presents the properties of the point plastic hinges.

3.2 COMPUTATIONAL MODEL

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

3.2.1 Material Properties

M-25 grade of concrete and Fe-415 grade of reinforcing steel are used for all members of the bridge. Elastic material properties of these materials are taken as per Indian Standard IS 456 (2000). The short-term modulus of elasticity (E_c) of concrete is taken as:

$$E_c = 5000\sqrt{f_{ck}} \text{ MPa} \tag{3.1}$$

where f_{ck} ≡ characteristic compressive strength of concrete cube in MPa at 28-day (25 MPa in this case). For the steel rebar, yield stress (f_y) and modulus of elasticity (E_s) is taken as per IS 456 (2000).

3.2.2 Structural Elements

Piers and girders supporting deck are modelled by 3D frame elements. The girder-pier joints are modelled by giving end-offsets to the frame elements, to obtain the bending moments and forces at the beam and column faces. The girder-pier joints are assumed to be rigid (Fig. 3.1). The pier end at foundation was considered as fixed. All the pier elements are modelled with nonlinear properties at the possible yield locations. Deck is not modelled 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.

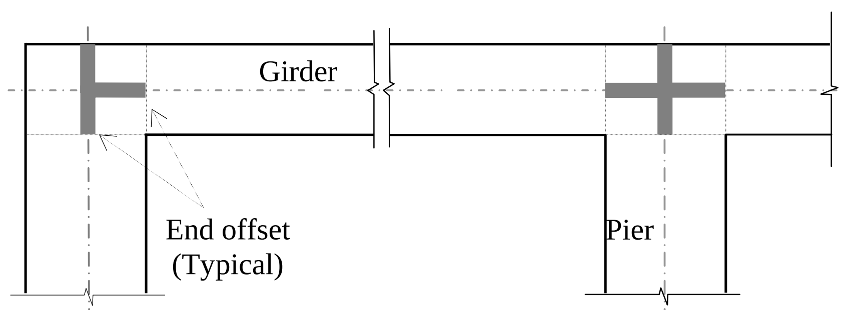


Fig. 3.1: Use of end offsets at pier-girder joint

3.2 BRIDGE GEOMETRY

The details of this bridge are obtained from literature (Muljati and Warnitchai, 2007). The bridge deck is supported by single-span pre-stressed concrete girders. Girders are placed on the concrete pier-head through the bearing and locked in the transverse direction. The supporting piers are in various heights, but in this study equal height of 7.7 m is selected. The width of the bridge is 10.5 m. Total 12 span with equal span length of 30 m. Fig. 3.2 shows a schematic diagram of the bridge in the longitudinal direction

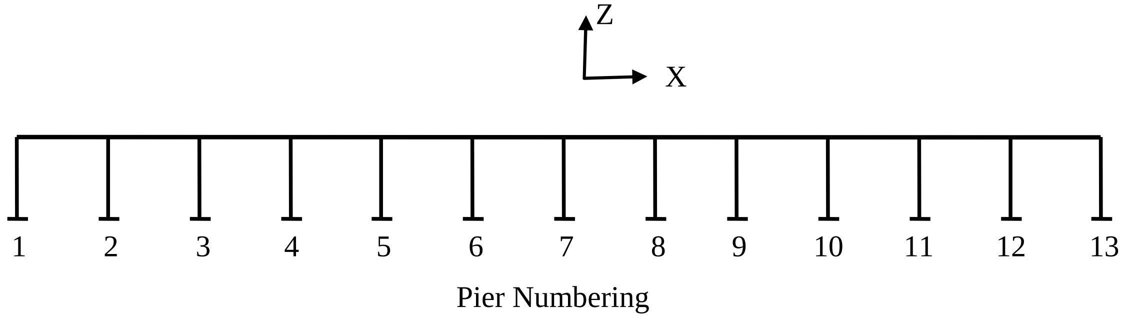


Fig. 3.2: Schematic diagram in longitudinal direction

Fig. 3.3 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 octagonal size as shown in Fig. 3.4

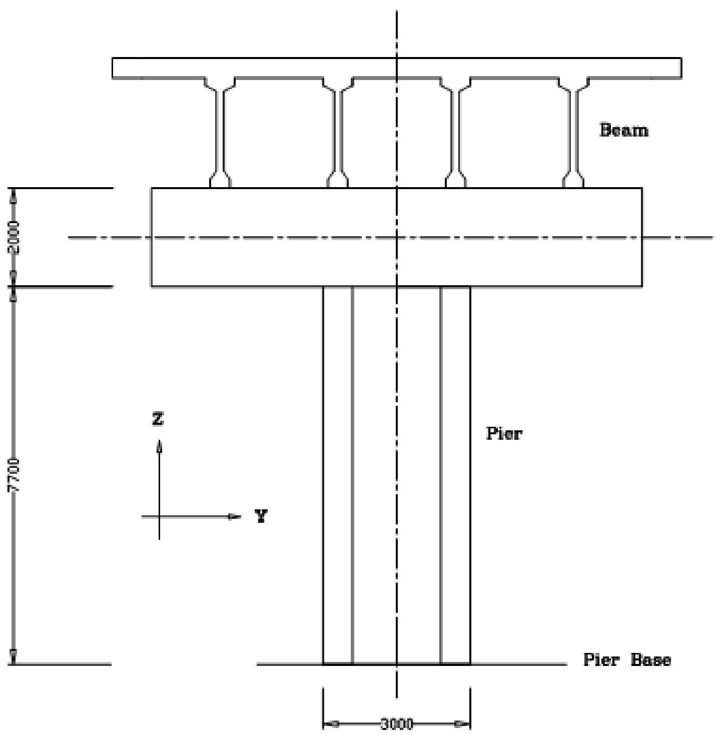


Fig. 3.3: Cross-sectional details of the bridge

The Bridge was modelled using commercial software SAP2000NL. A 3D computer model is shown in Fig. 3.5.

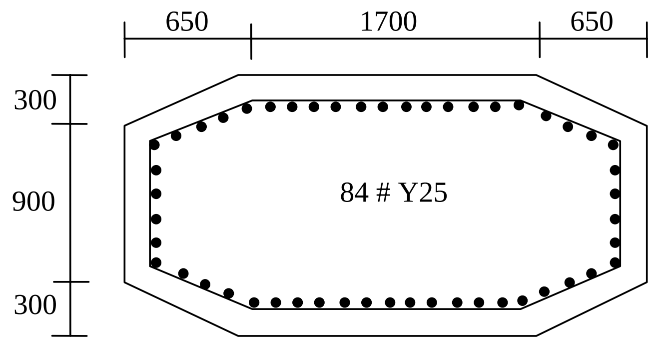


Fig. 3.4: Details of the pier section

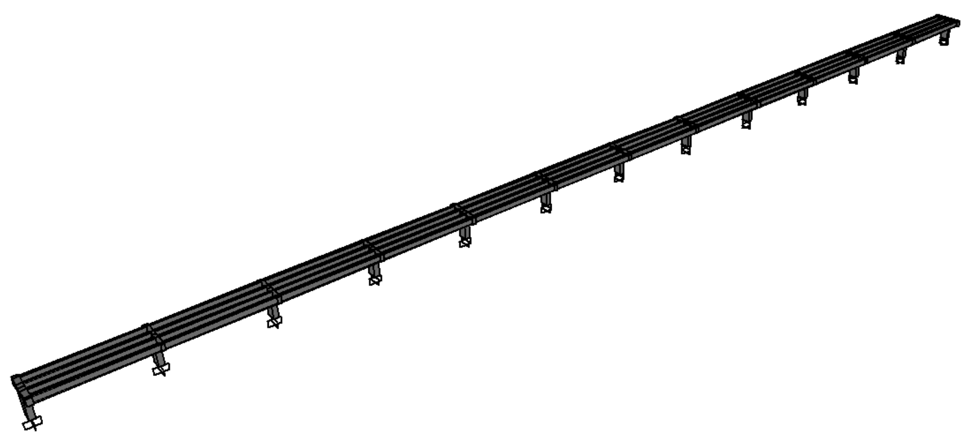


Fig. 3.5: 3D Computer model of the bridge

3.3 MODELLING OF FLEXURAL PLASTIC HINGES

In the implementation of pushover analysis, the model must account for the nonlinear behaviour of the structural elements. In the present study, 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. Piers in this study were 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.

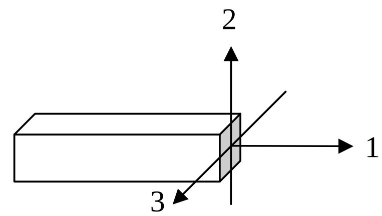


Fig. 3.6: The coordinate system used to define the flexural and shear hinges

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.

3.3.1 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 characterised 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 f_c corresponding to any given strain ε_c:

$$f_c = \frac{f'_{cc}\, x\, r}{r - 1 + x^r} \tag{3.5}$$

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

$$f'_{cc} = f'_{co}\left[1 + 3.7\left(\frac{0.5 k_e \rho_s f_{yh}}{f'_{co}}\right)^{0.85}\right] \tag{3.6}$$

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

$$\varepsilon_{cu} = 0.004 + \frac{0.6 \rho_s f_{yh} \varepsilon_{sm}}{f'_{cc}} \tag{3.7}$$

$$\varepsilon_{cc} = \varepsilon_{co}\left[1 + 5\left(\frac{f'_{cc}}{f'_{co}} - 1\right)\right] \tag{3.8}$$

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.

Fig. 3.7 shows a typical plot of stress-strain characteristics for M-20 grade of concrete as per Modified Mander's model (Panagiotakos and Fardis, 2001). 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.

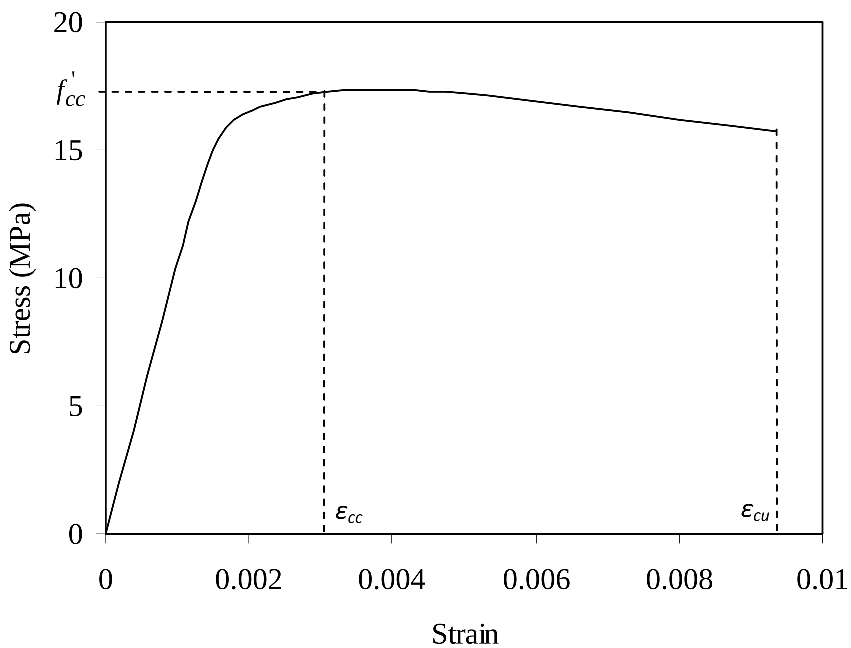


Fig. 3.7: Typical stress-strain curve for M-20 grade concrete (Panagiotakos and Fardis, 2001)

- 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 (*e.g.*, Pam and Ho, 2001).

3.3.2 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-415 grade of reinforcing steel (in tension or compression) are shown in Fig. 3.8.

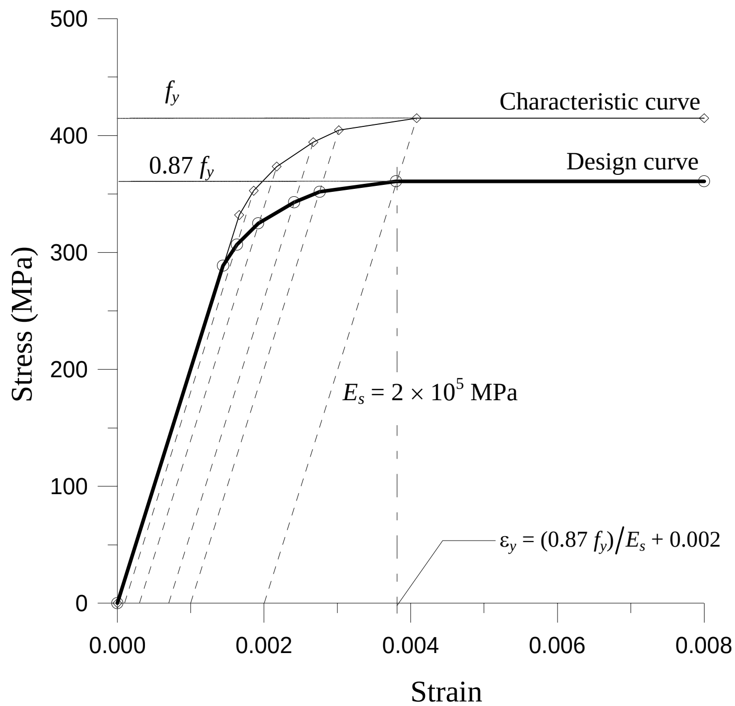


Fig. 3.8: Stress-strain relationship for reinforcement – IS 456 (2000)

3.3.3 Moment-Rotation Parameters

Moment-rotation parameters are the actual input for modelling the hinge properties and this can be calculated from the moment-curvature relation. The moment-rotation curve can be idealised as shown in Fig. 3.9, 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:

- The point ‘A’ corresponds to the unloaded condition.

- The point 'B' corresponds to the nominal yield strength and yield rotation θ_y.
- The point 'C' corresponds to the ultimate strength and ultimate rotation θ_u, 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, θ_u can be taken with that.
- The point 'E' defines the maximum deformation capacity and is taken as $15\theta_y$ or θ_u, whichever is greater.

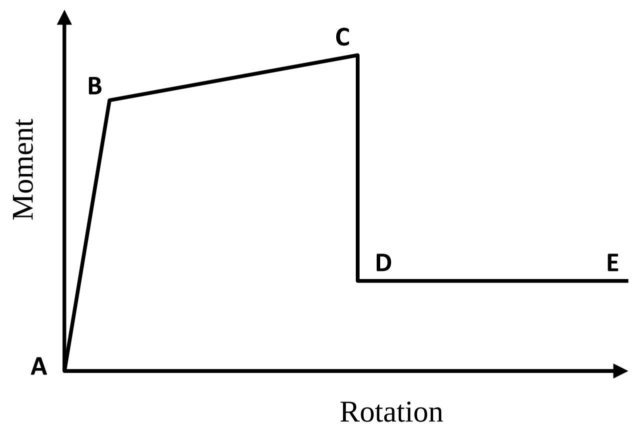


Fig. 3.9: Idealised moment-rotation curve of RC elements

3.4 SUMMARY

This chapter presents details of the basic modelling 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 modelling plastic flexural hinge.

CHAPTER-4

RESULTS AND DISCUSSIONS

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 INTRODUCTION

The selected bridge model is analysed using upper bound pushover analysis. 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.

4.2 MODAL PROPERTIES

Modal properties of the bridge model were obtained from the linear dynamic modal analysis. Table 4.1 shows the details of the important modes of the bridge in transverse direction (Y direction). The table shows that participating mass ratio in the first mode is only 56% cumulative mass participating ratio for first four modes is 65%. 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.

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

Mode	Period (s)	Frequency (Hz)	Eigen value (rad²/sec²)	UY*	Γ# (kN-s²)	$\frac{q_i}{q_1}$ **
1	0.600	10.47	109.71	0.56	136.6	1.00
2	0.598	10.50	110.26	0.06	-44.9	-0.33
3	0.595	10.56	111.46	0.02	-25.5	-0.19
4	0.590	10.64	113.30	0.01	15.9	0.12

* Mass Participating Ratio; # Modal Participation Factor; ** Refer Eq. 2.11

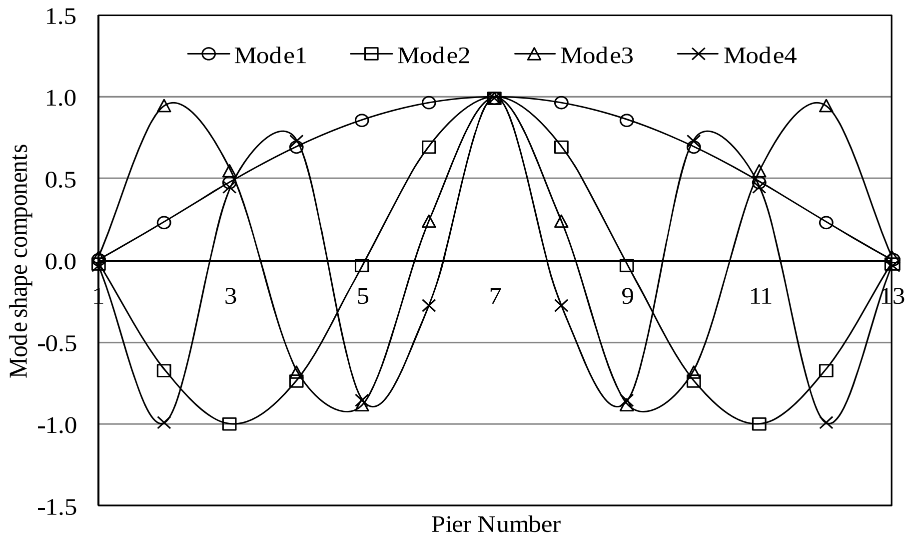


Fig. 4.1: First four modes of the bridge (normalised to Pier# 7)

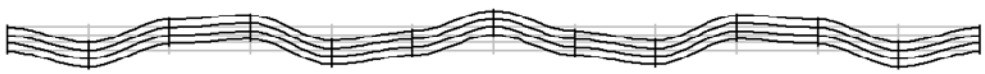
4.3 PUSHOVER ANALYSIS

Pushover analyses carried out using FEMA 356 displacement coefficient method as well as upper bound pushover analysis (UBPA) method. 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)

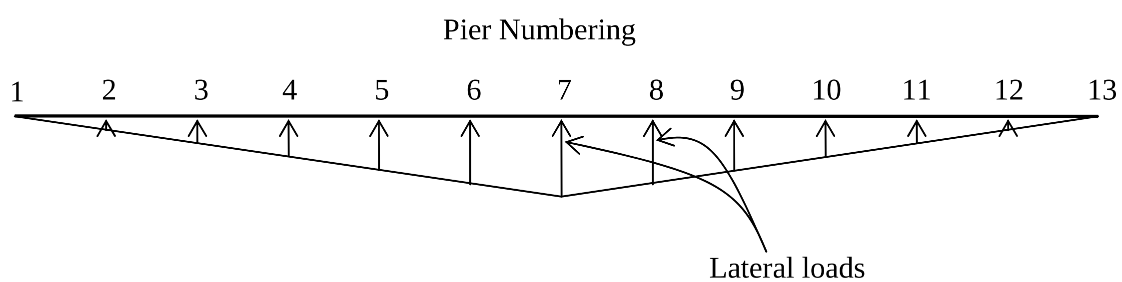


Fig. 4.3: Triangular load pattern used for standard pushover analysis

For UBPA, the load pattern for the analysis was calculated from the modal properties as discussed in Section 2.4.1. Sample calculation for determining the load profile for UBPA is presented in Table 4.2. Fig. 4.4 shows the load pattern for UBPA graphically and compares it with the triangular load pattern.

Table 4.2: Sample calculation for determining the load profile for UBPA

Pier No	Lumped Mass (kN)	Mode Shape Values		$f_{s,UB}$	$f_{s,UB}$ (normalised)
		M1 (2)	M2(4)		
1	6550	0.00	-0.02	8078	0.00
2	13100	0.24	-0.67	673287	0.39
3	13100	0.48	-1.00	892343	0.52
4	13100	0.70	-0.73	1328880	0.77
5	13100	0.86	-0.02	1724812	1.00
6	13100	0.96	0.70	1062131	0.62
7	13100	1.00	1.00	412655	0.24
8	13100	0.96	0.70	1062131	0.62
9	13100	0.86	-0.02	1724812	1.00
10	13100	0.70	-0.73	1328880	0.77
11	13100	0.48	-1.00	892343	0.52
12	13100	0.24	-0.67	673287	0.39
13	6550	0.00	-0.02	8056	0.00

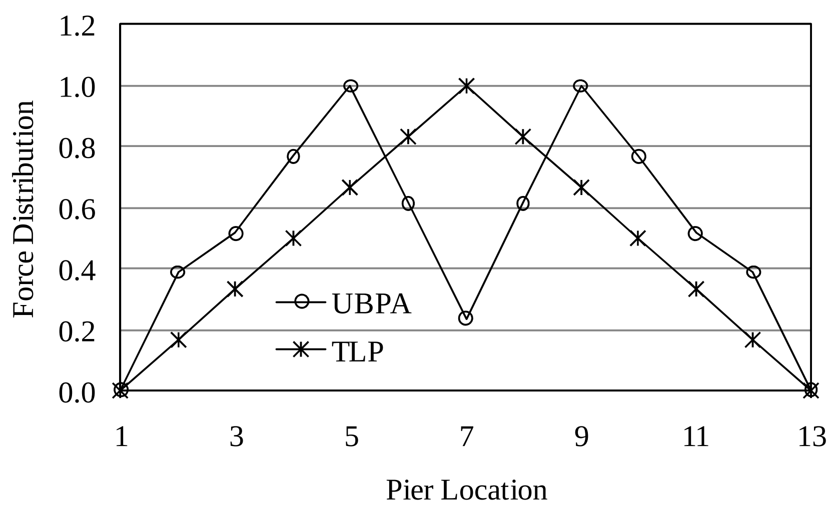


Fig. 4.4: Comparison of triangular and UBPA load pattern

4.3.1 Capacity Curve

Capacity curve of the bridge as obtained from the two pushover analyses (FEMA 356 with triangular load pattern and UBPA) are plotted and presented in Fig. 4.5. The definition of the capacity curve is discussed in Chapter 2.

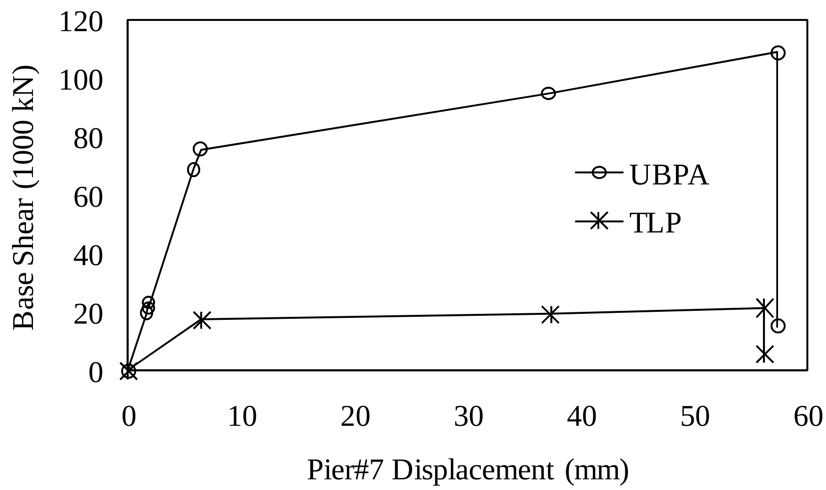


Fig. 4.5: Capacity curve of the bridge

Fig. 4.5 shows that UBPA estimates a very high base-shear capacity of the bridge in transverse direction as compared to the triangular load pushover analysis. However the estimated ductility is almost same for both of the two load patterns. This figure demonstrates the influence of load pattern on the capacity curve of the structure.

4.3.2 Target Displacements

Target displacements were calculated for different performance level as per the procedures discussed in Chapter 2. Table 4.3 presents the target displacement values calculated as per FEMA 356 displacement coefficient methods and that calculated as per UBPA procedures.

Table 4.3: Target displacements for different performance levels

Performance Level	IO	LS	CP
FEMA -356	80 mm	88 mm	96 mm
UBPA	106 mm	117 mm	128 mm

IO = Immediate Occupancy; LS = Life Safety; CP = Collapse Prevention

The results obtained from Pushover Analysis (both for FEMA-356 and UBPA) shows that the bridge collapses before reaching the Target Displacement. For FEMA-356, the failure is concentrated at the middle of the bridge whereas, for UBPA, the failure is distributed over the length of the bridges. Figs. 4.6 and 4.7 present the distribution of the plastic hinges in the bridge at collapse for the two pushover analyses.

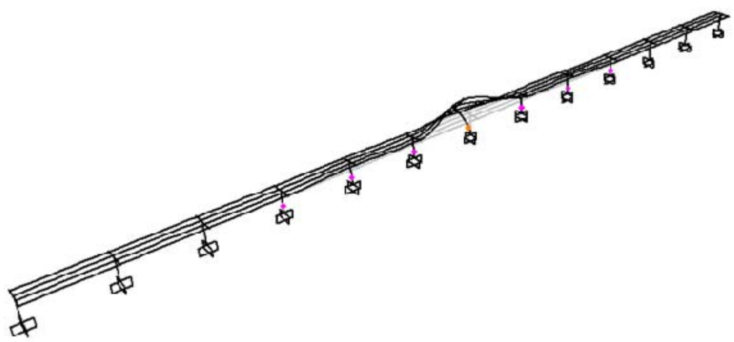


Fig. 4.6: Distribution of the plastic hinges as per FEMA 356

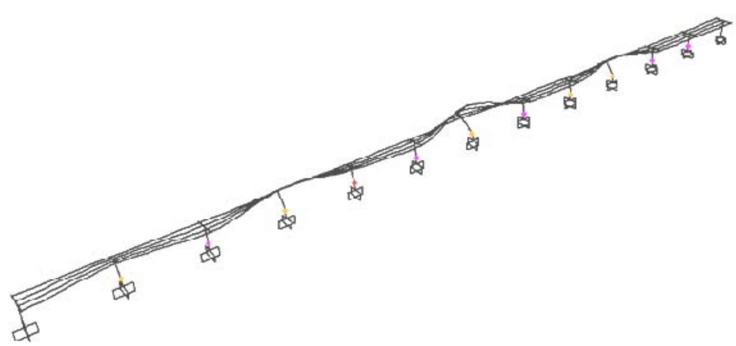


Fig. 4.7: Distribution of the plastic hinges as per UBPA

As the bridge could not achieve the target displacement in any of the pushover cases it can be concluded that the bridge is not safe for any performance limit state under the seismic demand corresponding Zone V. The distributions of the hinges are different for the two pushover analyses carried out in this study. For FEMA-356 loading hinges are concentrated at the middle of the bridges For UBPA loading, hinges are distributed over the entire length of the bridge. This bridge requires retrofitting for a desired performance.

CHAPTER-5

SUMMARY AND CONCLUSIONS

CHAPTER 5

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5.1 SUMMARY

After 2001 Gujarat Earthquake and 2005 Kashmir Earthquake, there is a nation-wide attention to the seismic vulnerability assessment of existing buildings. There are many literatures available on the seismic evaluation procedures of multi-storeyed 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. In order to address this problem, the aims of the present project was to carry out a seismic evaluation case study for an existing RC bridge using nonlinear static (pushover) analysis.

To achieve this, a multi-span RC bridge is selected from literature. The bridge was modelled using SAP2000 for nonlinear analysis. Nonlinear hinge properties were generated using improved stress-strain curve of concrete and reinforcing steel. The bridge is analysed using pushover analysis procedure as per FEMA 356 and Upper Bound Pushover Analysis procedure. Both of these two procedures are developed for multi-storeyed building. These procedures were 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 analysing the structure using 'Upper Bound Pushover Analysis' (UBPA) and FEMA-356 (TLP) pushover analysis, it was concluded that:

i) Here the performance of the bridge, according to FEMA-356 and UBPA, is not acceptable. Therefore it requires retrofitting.

ii) The distributions of the hinges are different for the two pushover analyses carried out in this study. For FEMA-356 loading hinges are concentrated at the middle of the bridges.

iii) For UBPA loading, hinges are distributed over the entire length of the bridge. However, the formation of hinges initiated from Pier# 5 and Pier# 10.

iv) Modal analysis of a 3D bridge model reveals that it has many closely-spaced modes.

v) Participating mass ratio for the fundamental mode is only 56%. Therefore, the contribution from the higher modes is very high (44%).

vi) Further investigation is required in order to make a generalised evaluation procedure for bridge structures with different configurations.

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