A MAJOR PROJECT THESIS
ON

'NON LINEAR STATIC ANALYSIS OF TYPICAL SHORT AND MEDIUM SPAN BRIDGES IN URBAN AREA'

submitted in the partial fulfillment for the requirement of the award of

MASTER OF ENGINEERING (M.E.)
IN
STRUCTURAL ENGINEERING

By

PARIMAL GODSE
M.E. (Structural Engineering)
Univ. Roll no.-10307

Under the Guidance of

PROF. D. GOLDAR
B.E.(Civil), M.Sc. Engg.(Structures), PhD (Structures), Steering Committee Member,
ACEM, MSEM(US), LMISCMS, LMISWE,
Former Principal, Delhi College of Engineering , Delhi-110042.

&

Dr. RAJEEV KUMAR GARG
Scientist (EII)
Deptt. of Bridge and Structures.
Central Road Research Institute (CSIR), Mathura Road, Delhi-110020.

Department of Civil and Environmental Engineering
DELHI COLLEGE OF ENGINEERING
UNIVERSITY OF DELHI
2007-2008

DEPARTMENT OF CIVIL & ENVIRONMENTAL ENGINEERING

DELHI COLLEGE OF ENGINEERING
UNIVERSITY OF DELHI

CERTIFICATE

This is to declare that the major project entitle **"Non-linear Static Analysis of Typical Medium and Short span Bridges in Urban Area"** is a bonafide record of work done by me for partially fulfillment of requirement for the degree of M.E., Civil Engineering (Structural Engineering) from Delhi College of Engineering.

This project has been carried out under the supervision of **Dr. D Goldar, Professor** and **Former Principal,** Delhi College of Engineering, Delhi and co-guided by **Dr. R.K. Garg, Scientist (EII),** Bridge and Structures, CRRI, New Delhi.

I have not submitted the matter embodied in this report to any other University or Institution for the award of any Degree or Diploma.

Name : **Parimal Godse**
Univ.Rolll No: 10307

Certificate:-

This is to certify that the above statement laid by the candidate is correct to best of our knowledge

Dr. Rajeev Garg
Project Co-guide
Scientist (EII), Bridges & Structures
CSIR, New Delhi.

Prof. D. Goldar
Project Guide
Delhi College of Engg.,
Delhi.

ABSTRACT

Non-linear static analysis is an effective analysis tool to evaluate the performance behavior using inelastic seismic study of Building and Bridges. Pushover technique uses non-linear static procedure where displacement control approach by capacity spectrum method and has been studied over the typical bridges in urban areas. For studying the performance behavior of bridges in urban area under high seismicity, without mentioning specific route typical structures resembling Metro piers i.e. mono-piers, multi bent structures like flyover and short span bridges have been studied. The seismic region of greater impact i.e seismic zone IV and V have been considered. Soil-structure interaction (SSI) has also been considered. Linear static (response spectrum) as well as nonlinear static procedure like push over analysis with parametric variation have been studied to understand their seismic performance. The evaluation of mode of failure in terms of hinge formation in structural components and estimating the capacity-demand relation under specified seismicity is carried out.

A linear static analysis like response spectra analysis is an elastic methodology to estimate the response of structure to the given earthquake. For forces and the displacement given by linear method the structure remains within elastic ranges. By introducing ductility in bridges, the load carrying capacity can be enhanced thus bridge is to be designed for lesser forces than obtained in elastic range, therefore, non-linear analysis (in-elastic range) is required. Pushover analysis is an effective tool to evaluate the actual behavior and consequent failure pattern for components wise. The response spectra analysis helps corroborating linear structural responses with nonlinear behavior.

In the present study, typical short and medium span bridge structure like a mono-pier, bent beam-pier frame (typical flyover) with and without elastic-foundation in the urban area (Delhi) are considered. Nonlinear push over analysis procedure by ATC-40 is adopted under various seismic demand. The hinge formation for expected performance level is noted, and compared for different boundary conditions in terms of different soil types using soil-structure interaction, ground acceleration input, and various values of ductility factors. The response parameters like base shear and roof displacement for each case studied are obtained. Evaluation of performance point (Sa, Sd) for the given

structure is considered (important parameter) as per capacity-demand methodology. Structural analysis has been carried out using software SAP2000 V.11.0.0. in the present study.

“THE CHANGE OF MOTION IS PROPORTIONAL TO THE MOTIVE FORCE IMPRESSED; AND IS MADE IN THE DIRECTION OF THE RIGHT LINE IN WHICH THE FORCE IS IMPRESSED.”

- SIR ISAAC NEWTON

To my Parents

ACKNOWLEDGEMENT

It gives me a great pleasure to submit the Thesis work on **'Non-Linear Static Analysis of Typical Short and Medium Span Bridges in Urban Area'** within the premises in the partial fulfillment of the requirements for the award of Degree of Master of Engineering with specialization in Structural Engineering from University of Delhi, Delhi.

I express deep gratitude towards my Project guide **Prof**. **Dr. D. Goldar,** Former Principal, Delhi College Of Engineering, without his consent and proper direction and guidance with his most valuable time , this thesis work could not have taken shape of the day.

In the same concern, I by heart most thankful to **Dr. Rajeev Kumar Garg** as a Co-guide from CRRI. This work is the reflection of his thoughts, ideas, concepts, sheer guidance, inspiration and above all his modest efforts. I am thankful to all the concern Scientists and staff at CRRI who contributed in the completion of my task.

I express gratitude to our Head of Deptt. **Prof. Awadhesh Kumar** for their Co-operation, facilitation and suggestions at time to time.

I would like to specially thanks Mr. Mahender Vashist, Mr. Kaushik, Mr. Susheel Kumar, Mr. Sanjay Arora, Mr. Puneet Sharma , recognized Bridge Engineers team, iBilt Technologies Ltd., Thapar House, New Delhi for the valuable basic guidance, suggestions and supports during this work.

A great and valuable help from Mr. Kandiyal, Maintenance Manager (DND Flyway), Mr. Dhananjay, Site Er, (DMRC),really makes my work verse with practicability application. I am thankful to them very much.

Above all I am deeply indebted to my parent for giving me motivation especially a moral support without which my task would never be turned into reality.

I would also like to express thanks towards all my friends Nilesh, Kanav,Durgesh whose assistance was invaluable. Last but not least, I am thankful to all of them who directly or indirectly helped me to complete this project work.

Parimal Godse

CONTENTS

List of Figures

List of Tables

1.1 General

A bridge collapse during earthquake may not result in many lives lost as compare to that in a building collapse, but the economic loss can truly substantial. Bridge often provides a vital link to earthquake ravaged areas as seen in the Bhuj earthquake and hence have a vital post-disaster purpose. Critical bridge must remain functional after the event to provide relief as well as for security and defense purpose.[1]

Bridge substructure includes variety of elements such as single or multiple column bents, reinforced concrete pier. The individual column may extend below the ground surface as a pile or caissons foundation or hey may be supported on a pile cap or spread footing. Much of the substructure damage in past earthquake has occurred at columns.[1]

The present Indian code particularly IRC:6 and IS:1893 and other related codes do represent the comprehensive knowledge and expertise available in India for earthquake engineering. Still the gap between knowledge gathered recently worldwide and codification needs to be bridged particularly for bridges. IRC:6 gives simplification of that earthquake force level static forces. In fact they are high magnitude dynamic force induced by mass inertia due to severe ground shaking.

The seismic performance of structure can be very well predicted by the pushover analysis. Bridges behaves differently than that of building under seismic condition due to its pattern of force transfer, redundancy and variable component behavior. Non-linear static analysis is effective technique to study seismic response of structure like bridges. For studying the performance behavior of bridges in urban area under high seismicity, without mentioning specific route typical structures resembling Metro piers i.e. mono-piers, multi bent structures like flyover and short span bridges have been studied. The seismic region of greater impact i.e seismic zone IV and V have been considered. Soil-structure interaction (SSI) has also been considered. The performance based design approach is need of the day.

1.2 Objective of the work

A short and medium span bridge structures has taken for the seismic analysis purpose. As from the past earthquake disaster, it has been seen that substructures are more vulnerable to damage which is serious issue. So , non-linear static analysis method like push over analysis is carried out over this structural components against critical seismic demand. A seismic behavior is studied under different parametric variations including the soil-structure interaction effect, various response reduction factor, zone factor, ground acceleration demand. Under such inputs, by using Nonlinear static procedure like capacity-spectrum method , the corresponding structural responses and failure pattern seen. The effectiveness of the methodology also check against the linear static analysis method like response spectra analysis. The effect of mono-pier with or without foundation on the performance of structure is to be studied. The limiting response in terms of base shear and roof displacement are to be check against the chosen performance level and corresponding ductile designing aspects suggests as per deficiencies. This is part of performance based design of structure.

1.3 Methodology of the work

A capacity spectrum method – ATC 40 with nonlinear structural analysis software SAP2000 is used in the study to evaluate the structural components response and failure pattern. In task undertaking, a short and medium span bridge structure in urban area (Delhi) is taken into consideration. A beam-element model with lumped mass at discretized node generated using SAP2000. A boundary condition taken into consideration like fixed base and base with elastic foundation. Then by using response spectra analysis as per IS 1893:2002 with all possible parametric variation with ground acceleration , ductility factor ‘R’ and different soil type carried out. The relation between base shear and roof displacement for each combination plotted and studied for the taken boundary condition. Next switch to Capacity spectrum method, through displacement controlled option, the seismic behavior for structure gives performance point and the corresponding location of hinges seen by using SAP2000 capability so ductile designing aspects can suggest.

The interpretation of results from linear and nonlinear static analysis for fixed and elastic foundation base component wise and for full bridge has been done for seismic performance objective. The comparison between linear and nonlinear analysis under similar condition has been done by using SAP2000 and MS Excel tool.

1.4 Layout of the work

In chapter 1, the introduction to the work, motivation behind step to work, problem taken and methodology adopted is described.

In Chapter 2, Literature review ,a concept of performance based design develop along with some structural behavior (ductile) property. A brief review of non-linear inelastic behavior of structures specially short and medium span bridges presented. The performance of bridge under inelastic condition has been predicted by various authors It clears the component wise behavior with and without soil-structure interaction. Need of push over analysis has been explained.

In Chapter 3, Structural analysis procedure both linear and nonlinear are introduced. The more emphasis given to linear static analysis procedure like response spectra analysis and non linear static procedure like pushover analysis- A displacement controlled based Capacity spectrum method (ATC 40). The methodology and significance for each procedure is briefly described. Also relevant features of software application like SAP2000 explained.

In chapter 4, three cases of short and medium span bridge structure considered from urban area .Their structural, material and geometric properties defined. The mathematical model is generated using SAP2000 features. This model under push over analysis along with linear elastic response spectra analysis presented with the parametric variation as per IS 1893:2002 and ATC 40 .In this section, procedure by Winkler for soil-structure modeling is explained.

In chapter 5, Results for each case presented and discussed. A chapter 6 draws a conclusion from overall work along with future study scope and chapter 7 enlists the references used for the study.

Performance Concept

The basic design criterion, which any Earthquake-resistant structure must satisfy, is the following:

Seismic demand ≤ Computed capacity

'Seismic demand' is the effect of the Earthquake on the structure. 'Computed capacity' is the structure's ability to resist that effect without failure. In short, the structure should not fall down. It should be noted that in the dynamic loading environment, the demand and capacity of a structure are very strongly coupled. At the same time, structure must meet all functional requirements at minimum economic cost. Unfortunately, it must be recognized that no structure can be completely safe. One, we cannot perfectly predict the seismic demand due to Earthquake loads; two**,** the computed versus actual capacity of a designed structure may not match perfectly; three**,** there could be human errors in design and construction. Earthquake loads are inertia forces resulting from ground movements and they impose certain demands on the structures related to strength, ductility and energy. The magnitudes of these demands are highly variable and are dependent on the seismicity of the region and the dynamic characteristics of the structure – which is why they cannot be predicted precisely and can be expressed only in probabilistic terms. Simplistically, it is graphically shown in Fig. 2.1, where probability density functions of demand and capacity are plotted. The design demand is the predicted maximum value of seismic demand for design purposes and actual distribution indicates that there is some probability that it would be exceeded. Similarly, the computed capacity is obtained by accepted methods of analysis and design. The distribution for capacity suggests that there is some probability that the actual as-built capacity may be less than the computed value. However, due to extra conservatism in design process, there is greater probability that it would be larger. The shaded area in Fig. 2.1 where both distributions overlap indicates that there is some probability of failure, where capacity is less than demand.

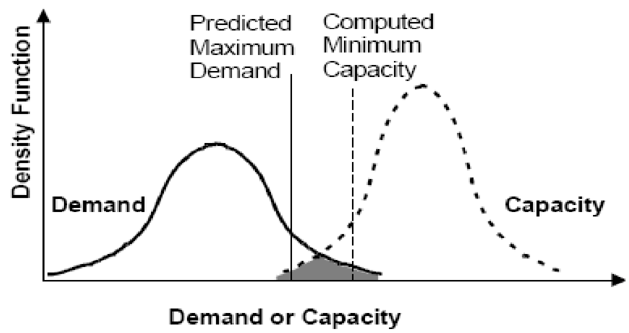


Fig. 2.1 Probability distribution for capacity and demand concept [6]

The inter-relationship between these two entities of the design process, i.e. demand and capacity is shown in Fig. 2.2 various quantities that determine demand and capacity and how design codes try to define them and specify a standard process for the design of a structure of acceptable performance level are also shown in Fig. 2.2. Various strategies for providing adequate capacity for the attenuation of the seismic response in a structure have been listed as well. Similarly, on the demand side, various factors characterizing the ground motion that determines the severity of the demand are listed.

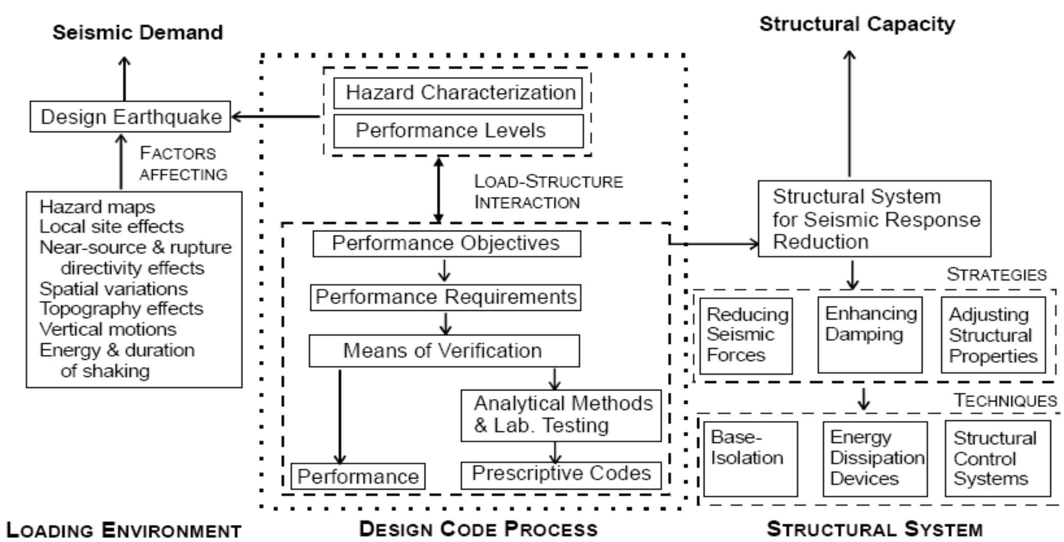


Fig.2.2 Inter-relationship between demand and capacity as applied to PBSD. [6]

Major efforts in Earthquake engineering research are directed towards reducing the level of uncertainties in predicting the ground motion at a site and the response of a structure due to that ground motion. Currently, structural responses can be predicted fairly confidently, but the prediction of ground motion is far from satisfactory. Many new devices, techniques and strategies have been continuously developed for the structural system to either reduce the seismic demand or to increase the strength, ductility or energy dissipation capacity.

Performance Objective

What is 'the level of acceptable risk' to be used in designing an Earthquake-resistant structure and who decides it? Risk is expressed in terms of *hazard* and *vulnerability.* In our context, an Earthquake is the hazard and susceptibility of structures to damage is the vulnerability. For co-relation with Capacity and Demand, hazard evaluates the seismic demand and vulnerability is the measure of capacity.

To engineers and designers (who, by the way, feel personal responsibility for the performance of every structure) a design that causes minimum loss of life and damage to structures is acceptable, even if the cost is high. The Structural Engineers Association of California (SEAOC) in their Vision 2000 document defines performance objectives. 'Expected performance level' can be one of the three damage states: immediate occupancy, life safety, and collapse prevention. These performance levels are combined with the expected ground motions at a particular site to determine the acceptability criteria for the structure. Hazard levels can vary from frequent to very rare occurrences of seismic events. In this framework, by specifying which performance objective is acceptable for various Earthquakes under consideration, a level of acceptable risk would be clearly indicated. Damage sustained by the structure while dissipating energy during an Earthquake is dependent on inelastic deformations (displacements) which the structure experiences. As a result, displacement parameters of a given structure provide the realistic evaluation of effects of Earthquake damage. Nonlinear Static Procedures (NSPs) of structural analyses are simplified numerical tools to obtain the structure's capacity curve, which relates an appropriate global deformation parameter to a global force parameter. For a given structure, a global displacement capacity limit *d*c for a specific performance level is based on prior experience of damage in terms of observed width and extent of concrete and masonry cracks or similar inelastic behavior. Similarly, displacement demands, *d*d, due to various levels of seismic hazards can be generated using NSPs in conjunction with an appropriate capacity curve. In Fig. 2.3, displacement demands for various hazard levels are plotted on the upper horizontal axis, whereas limits on displacement capacities for various performance levels are plotted on the lower horizontal axis. This combined plot provides a complete picture of the risk associated with a particular design of the structure. A structure meets a specific performance objective if the corresponding ratio, (*d*c / *d*d), of displacement demand and capacity is 1.0 or greater. In Fig. 2.3, the hypothetical structure does meet the performance objectives of immediate occupancy and life safety, but fails to meet the collapse prevention performance objective.

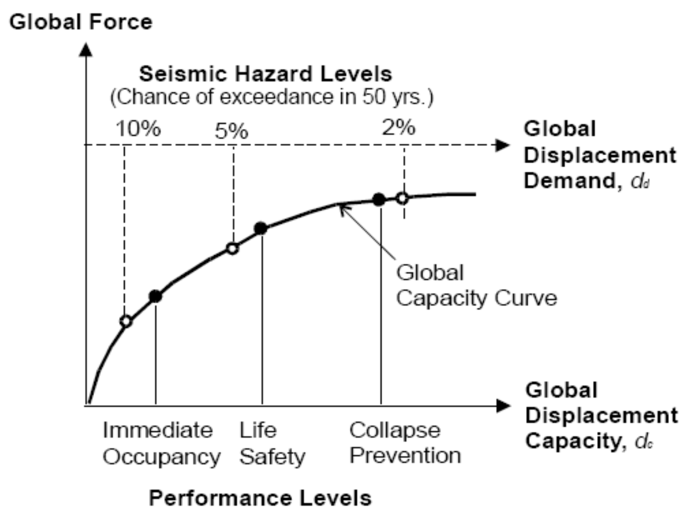


Fig.2.3 Global displacement demands and capacities [6]

Performance Approach

As in Figure 2.4, inelastic seismic demand is based on inelastic capacity of structure. As inelastic displacements increase, the period of structure lengthens, damping increases and demand reduces. The Capacity Spectrum Method generates Performance Point where displacement is consistent with the implied damping. Design is based on displacement corresponding to the Performance Point, which implies a unique damage stage related to a specific hazard level.

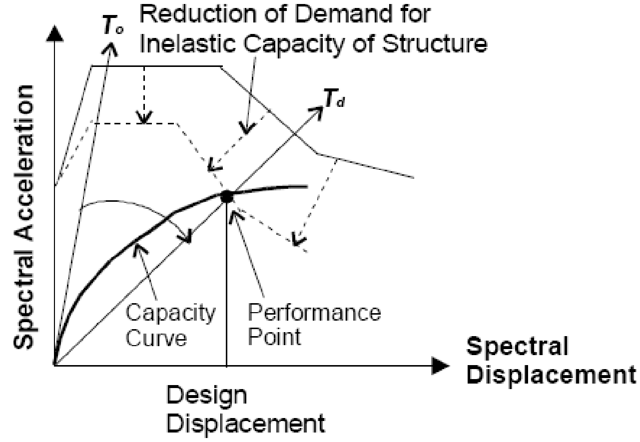


Fig.2.4 Evolution of seismic design procedure [10]

Ductility Factor for Performance evaluation

The ductility factor is the measure of global nonlinear response of framing system and the component of the system. Assuming for the moment that a multi story structure can be modelized single degree of freedom (SDOF) system and estimates of the global displacement available, relation between ductility factor and displacement ductility can be developed this relation for SDOF system has been subjected of much research in recent years.

The static lateral force method accounts for nonlinear response in seismic framing system by use of response modification factor (R) this factor first introduced in the ATC-3-06

In the late 1970's serves to reduce the base shear force (Ve) calculated by elastic analysis using 5% damped acceleration response spectrum (Sa) for the purpose of calculating design base shear (Vb)

$$V_b = \frac{V_e}{R} = \frac{S_a W}{R}$$

This R-factor describes as an empirical response modification (reduction) factor intended to account for both damping and ductility related to the structural system at displacements enough to approach the maximum displacement of the system.

The relation between the ductility factor and the global displacement ductility are well established for bilinear SDOF systems but should be extended to multiple degree of freedom system using a reduction factor.

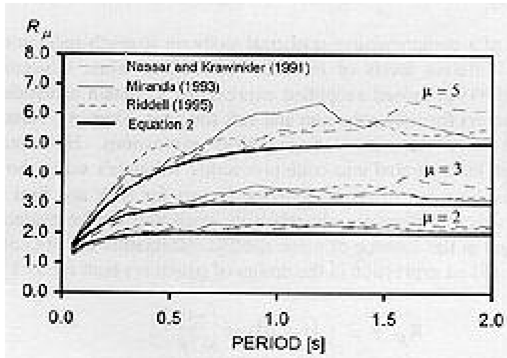


Fig.2.5 Relation between ductility factor and global displacement ductility

Performance Evaluation for Bridges under Nonlinear Static Procedure

Bridge researchers and engineers are currently investigating concepts and procedures to develop simplified procedures for performance based seismic evaluation of bridges (Dutta, 1999: Shinozuka, 2000).

Fenves and Ellery (1998) studied the three-dimensional nonlinear model of the multiple frame highway bridge failed in 1994 Northridge earthquake using DRAIN-3DX computer program. The objective was to ascertain the cause of failure by comparing the capacities and demands of various components in the bridge, and to examine the earthquake modeling and analysis recommendation for highway bridges. Nonlinear static pushover analysis was conducted in modal pattern to determine the capacity of the piers, superstructure and intermediate hinges to understand the failure criteria. To validate the nonlinear static procedure, especially displacement coefficient method and capacity spectrum method for bridges. Al Ayed (2002) analyzed the three span bridge using nonlinear time-history and pushover analysis. The spine model of bridge using frame elements with lumped mass was used to evaluate the force and displacement. The displacement, base shears and rotation of plastic hinges from pushover analysis were compared with nonlinear modal time history analysis to get the response similar or close to the actual seismic response. Jeremic (2004) studied the influence of soil foundation structure interaction on seismic response of viaduct and found that Soil foundation-structure (SFS) intersection can have both beneficial and detrimental effects on structural behavior and is dependent on the characteristics of the earthquake motion.

Paraskeva et al. 2006 studied the seismic behavior of bridge by modal pushover analysis procedure taking the higher mode effects into consideration. In their study the pushover analysis are carried out separately for each significant mode and the contribution from individual modes to calculate response quantities (displacement, base shear etc.) are combined using an appropriate combination rules like SRSS and CQC. The result have compared with the result of load pattern resulting from statistical combination of modal loads and nonlinear time history analysis. The modal pushover results were found to be closer to nonlinear time history analysis.

3.1 Linear Static and Dynamic Procedure

The basis, modeling approaches and acceptance criterion for the Linear Dynamic Procedure (LDP) is similar to those for Linear Static Procedure (LSP). The main exception is that the response is obtained from either a linearly elastic response spectrum or a time-history analysis. As with LSP, LDP will produce displacements that are approximately correct, but will produce inertial forces that exceed those that would be obtained in a yielding structure.

The Response spectrum method uses peak modal responses calculated from an eigen value analysis of a mathematical model. The time history method involves a time-step by step evaluation of the structural response using a discredited record or synthetic record as base motion input. In both the methods, only modes contributing significantly to the response need to be considered. In the response spectrum analysis, modal responses are combined using rational methods to estimate total building response quantities.

3.1.1 Response Spectrum Method

All significant modes must be included in the response spectrum analysis such that at least 90% seismic mass participation is achieved in each of the structures principle directions. Modal damping must reflect the damping inherent in the structure at the deformation levels less than yield deformation.

In IRC 6 code ,standard response spectra curve for Bridges is given which is shown in Fig. 3.1 used in response spectra analysis of structure. Its behavior changes according to soil type. A simple procedure is given in the standard for calculating a design base shear and corresponding roof displacement so one can predict the seismic behavior. The response evaluated by this method are well within the elastic range by considering response reduction factor ‘R’. This factor very much co-relate the ductile behavior of structure. Its affects the structural responses and ultimately the ductile design approach. Predicting and assigning an accurate ductility to structure is matter under intense research.

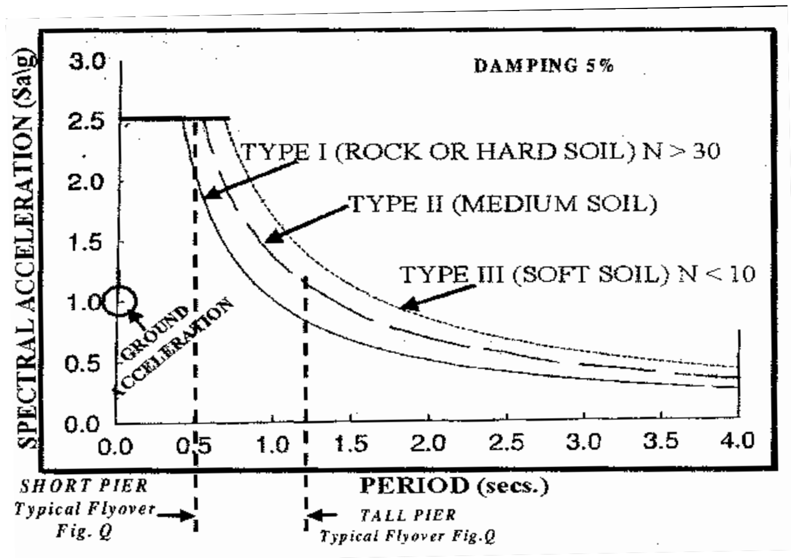


Fig. 3.1 Proposed IS: 1893 Response Spectra for Bridges [8]

The peak member forces, displacements story forces, shears and base reactions for each mode should be combined using SRSS (square root sum of squares) or CQC (complete Quadratic combination). It should also be noted that the directivity of the forces is lost in the response spectrum analysis and therefore the combination of forces must reflect this loss. Multidirectional effects should also be investigated when using the response spectrum analysis.

3.1.2 Time History Method

Al1 the requirements for response spectrum analysis are also identical for the time history analysis. Response parameters are computed for each time history analysis. If three pairs of time histories are used, the maximum response of the parameter of interest shall be used for the design. If seven or more pairs of time histories are used, the average response (of the maximum of each analysis) of the parameter of interest is to be used.

Multidirectional effects can be accounted by using a three dimensional mathematical model and using simultaneously imposed pairs of earthquake ground motions along each of the horizontal axes of the building.

3.2 Nonlinear Static Procedure

In the Nonlinear Static Procedure (NSP) the nonlinear load-deformation characteristics of individual elements and components area modeled directly. The mathematical model of a the structure is subjected to monotonically increasing lateral load until a target displacement is reached or the structure collapses. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake.

The nonlinear effects are directly included in the model and therefore the calculated inertial forces are reasonable approximations of those expected during the design earthquake.

The target displacement can be calculated by any procedure that accounts for nonlinear response on displacement Amplitude as well as damping effects at the performance point. One such procedure called the Displacement Coefficient Method (FEMA 273). ATC-40 also includes this method as an alternative method of finding the performance point. The important method giving in ATC 40 is Capacity spectrum method which use spectral acceleration vs spectral displacement relation to study the structural behaviour.

The modeling requirements for NSP are similar to those described in ATC-40. The pushover analysis is performed and a curve relating the base shear force and the lateral displacement of the control node are established between 0 and 150% of the target displacement, δi. Acceptance criterion is based on the forces and deformation corresponding to the displacement of the control node equal to δi.

The analysis model must be sufficiently discredited to represent the load-deformation response of each element or component. Particular attention needs to be paid to identifying locations of inelastic action along the length of element or component. Thus, local models of elements or assemblages of elements need to be studied before embarking on the global models.

Simplified Nonlinear procedures are used for practical applications and they have been found to be most rationalized methods. The different simplified nonlinear procedures used to implement the pushover analysis are:

(i) Capacity Spectrum Method (CSM) (ATC-40, 1996)

(ii) Displacement Coefficient Method (DCM) (FEMA-273, 1997)

(iii) The secant method and

(iv) Modal Pushover Analysis (MPA) (Chopra 2001)

The different methods used for evaluating the Non linear static procedure (NSP) may lead to similar results in most of the analysis but they differ in respect to simplicity, transparency and clarity of theoretical background. Non linear static procedure (NSP) is a powerful tool for evaluating the inelastic seismic behavior of structure.

3.2.1 Incremental Load Technique

The conditions of equilibrium for a given structure are satisfied by solving the structural stiffness equations for the unknown generalized (global) displacements and a known applied loading. The most suitable approach to analysis is by applying the total load in a series of small finite-sized increments. For each load increment the resulting increment of displacement is determined from the incremental stiffness equations where the stiffness parameters are evaluated to reflect the instantaneous state of the total displacement, total stress and material characteristics that exists just prior to the application of the load increment. The total displacement after the load increment is evaluated by adding the computed displacement increment to the total displacement that exists prior to the application of the load increment. This type of solution is piecewise linear solution, a physical representation of which is illustrated in Fig.3.2. This figure shows three load displacement curves for a single degree-of-freedom system. Curve A represents the linear behavior which result by solving the governing stiffness equation for the total load applied in one increment; curve B is the piecewise linear solution which would result by applying the total load in several increments and curve C represents the exact nonlinear behavior. It is clear that as the size of the load increment approaches zero (or the number of load increments approaches infinity), the piecewise linear curve approaches the true curve. Since load increments of infinitesimal order are impossible to achieve, a reasonable number of moderately sized load increments in applied.

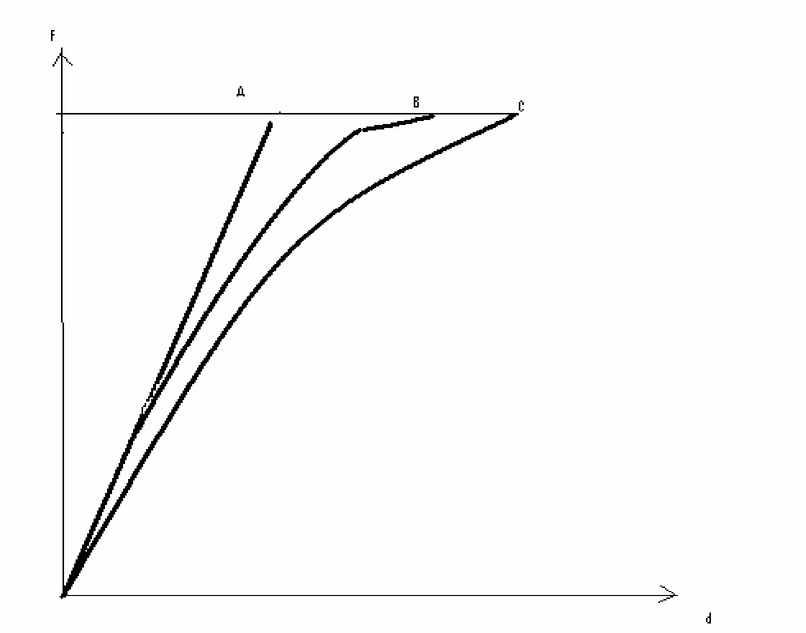


Fig. 3.2 Incremental Load Technique for SDOF

3.2.2 Push Over Analysis

Nonlinear static analysis also called as pushover analysis is used to determine displacement capacity of structures and also to estimate available plastic rotational capacities to ensure satisfactory seismic performance. Seismic demands in pushover analysis are estimated by establishing the capacity curve for a structure by monotonically increasing the displacement at a control node until a prescribed displacement is reached or the structure collapses. Control node is a node which is used to monitor the displacement of the structure and it should satisfy two conditions,

i) It should have a maximum of displacement

ii) Its deflection should reflect the behavior of the structure.

In case of longitudinal pushover analysis any node may be selected as a control node. In case of transverse direction of bridges, since the bridge is restrained at both the ends, the center of mass can be considered as a control node, if the bridge is symmetric. In case of non-symmetric, maximum displacement point may be considered as a control node.

The distribution of lateral inertia forces varies continuously during earthquake response. Loading pattern is the most important factor affecting the capacity curve, which in turn affects the target displacement. Different load patterns such as uniform pattern, Modal pattern and Spectral pattern are recommended by FEMA-273 and ATC-40 to represent the load intensity produced by earthquake.

i. Uniform pattern is one which is widely used and it is based on lateral forced that are proportional to the total mass assigned to each node. In buildings, the uniform load pattern is applied based on the lateral forces that are proportional to the total mass at each floor level. In bridges it can be directly taken as

$$\mathbf{F_i = m_i a_g}$$

ii. In Modal pattern, monolithically incremental displacement is applied in the mode shape of the structure and can be represented as

$$\mathbf{Fi = (mi*Qi / \sum_{i=1}^{n} mi*Qi) * V}$$

where, Fi is the lateral force at node i (i=1,2,3,…….n), n is the number of nodes, ag is the ground acceleration, m_i is the mass assigned to i_{th} node, Q_i is the amplitude of the fundamental mode at i_{th} node, and V is the base shear. This pattern may be using in the fundamental mode having maximum total mass participation. The value of V is optional since the distribution of forces is important while the values are increased incrementally until reaching the prescribed target displacement or collapse.

iii. Spectral pattern is used when the higher mode effects are deemed to be important (Jangid and Datta 1993). This load pattern is based on modal forces combined using Square root of Sum of the Squares (SRSS) or Complete Quadratic Combination (CQC) method, it can be represented as

$$F_i = \left(m_i \delta_i \Big/ \sum_{i=1}^{n} m_i \delta_i \right) V$$

Where, δ_i is the displacement of node i resulted from response spectrum analysis.

The ATC-40 and FEMA-273 and 356 have developed the acceptance criteria for pushover analysis using two different methods such as Capacity Spectrum Method (CSM) and Displacement Coefficient Method (DCM) to find out the performance point or target displacement of the structure.

3.2.3 Capacity Spectrum Method (CSM) - ATC 40

The procedure for the CSM has been developed by ATC-40. In CSM, the design curve shown in Fig.3.3 is reduced by using spectral reduction factors to intersect the capacity curve shown in Fig.3.4 to find the performance point. The performance point indicated the seismic capacity of structure which will be equal to seismic demand imposed in structure by ground motion. In pushover analysis, the performance point or target displacement is based on the assumption that the fundamental mode or uniform mode of vibration is the predominant response of the structure and mode shapes remain unchanged until collapse occurs.

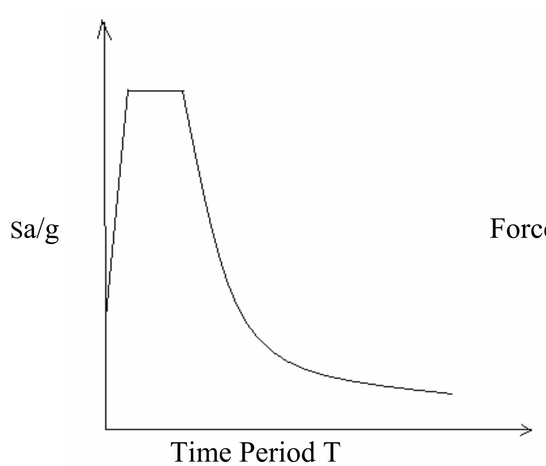


Fig. 3.3 Demand curve

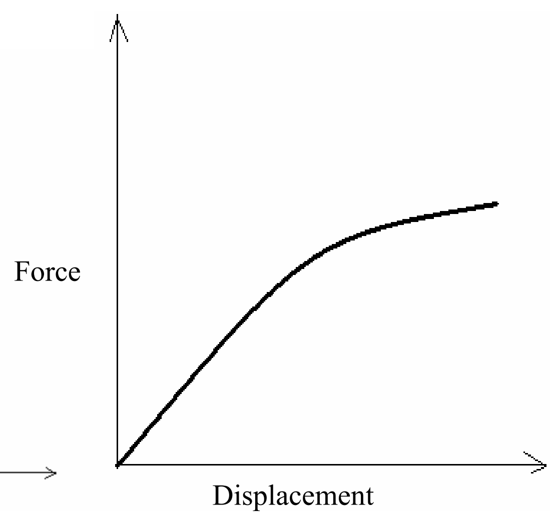


Fig. 3.4 Capacity curve

The performance point must satisfy two relationships

a) The point must lie on the capacity spectrum or capacity curve in order to represent a structure at given displacement.
b) The point lie on the spectral demand curve, reduced from the elastic 5 percent-damped design spectrum.

The structure to satisfy the above two relationships the spectral acceleration of structure and spectral acceleration of the response spectra should be same and the performance point requires a trial and error method to satisfy the above condition. ATC-40 proposed three procedures 'a', 'b', 'c' to determine the performance point. Procedure 'a' and 'b' are analytical and 'c' is graphical procedure. Step-by-step procedure for 'a', 'b' and 'c' are explained in ATC-40. ATC simulates three categories of structural behavior A, B and C to consider the damping modification. 'A' represents reasonably full hysteresis loops, 'B' represents moderate reduction in hysteresis area and 'C' represents poor hysteric behavior.

The bilinear representation of the capacity spectrum is shown in Fig.3.5. The damping that occurs in the inelastic range of structural behavior is a combination of viscous damping associated with hysteresis damping can be represented by (Kumar and Paul, 2007)

$$\beta eff = k * \beta o + 0.05$$

$$\beta o = (1/4\pi)\,(E_D/Es)$$

E_D is the energy dissipated by damping or area enclosed in a single hysteresis loop of capacity curve, shown in Fig. 3.5

$$E_D = 4\,(Vy*Di - Dy*Vi)$$

Es is the maximum strain energy = Area of triangle ODiB in Fig. 3.5

$$Es = ViDi/2$$

For structures which are not typically ductile, the eq. for βo over estimates the equivalent viscous damping. Imperfect hysteresis loop are taken care by multiplying the effective viscous damping using a damping modification factor, k (ATC-40).

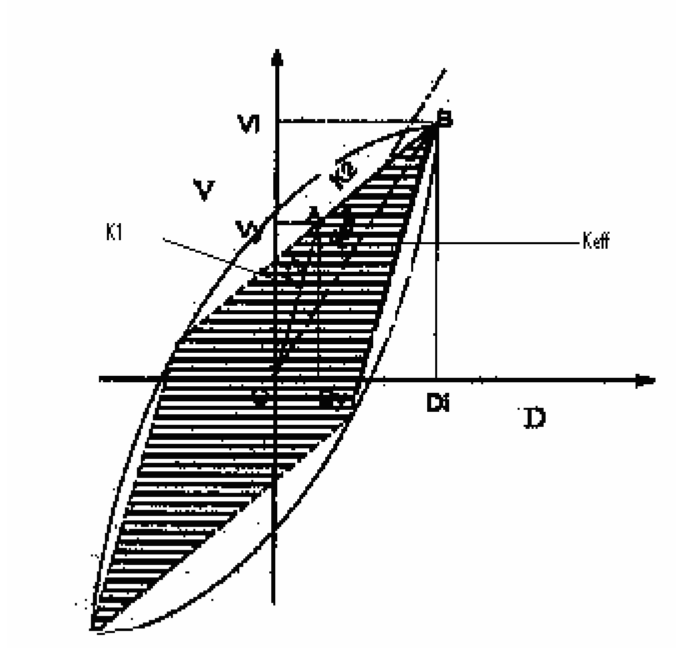


Fig.3.5 Hysteresis behaviour of structure from capacity curve.[10]

The design spectrum in CSM is reduced using spectral reduction factor which is a function of effective damping with capacity curve of the structure. Spectral reduction S_{RA} and S_{RV} as per ATC-40 are given by

$$\mathbf{S_{RA} = 3.21 - 0.68 \ln(\beta eff) / 2.12}$$

$$\mathbf{S_{RV} = 2.31 - 0.41 \ln(\beta eff) / 1.65}$$

This reduced demand spectra intersect with capacity spectra gives the co-ordinates of performance point.

CHAPTER 4 STUDY OF TYPICAL SHORT AND MEDIUM SPAN BRIDGES IN URBAN AREA (DELHI)

For studying the Performance behavior of bridges in urban area under seismicity, without mentioning specific route typical structures resembling Metro piers i.e. mono-piers, multi bent structures like flyover and short span bridges have been studied. The seismic region of greater impact i.e. seismic zone IV and V have been considered. Soil-structure interaction (SSI) has also been considered. Static (response spectrum) as well as nonlinear static procedure like push over analysis with parametric variation have been studied to understand their seismic performance. The structural details do not pertain to the actual bridges rather their relevance is considered during seismic performance study.

The modeling includes :

1) A mono-pier model.

2) A frame model-multibent (flyover/short span bridge).

3) A Full short span bridge model

4.1 Study of Mono-Pier bridge

4.1.1. Structural Details and Modeling

Circular pier , Diameter- 1.8m, Height= 9m, Centre of Gravity (C.G.) of super structure= 2m from pier top. (Fig.4.1)

Concrete: M45 grade, Steel: FE500 grade

Adopt , Reinforcement : 1.35% gross area (As per Priestley [11])

Longitudinal Reinforcement : 36no. of 36mm with 40mm clear cover.

Transverse Reinforcement : 20mm Diameter spiral at 115mm c/c spacing.

In Soil-structure Interaction (SSI) model, Pile with same diameter as 1.8m is taken up to 10m depth below ground level with adopted reinforcement value.

Loading- 200kN/m (gravity), span= 40m.

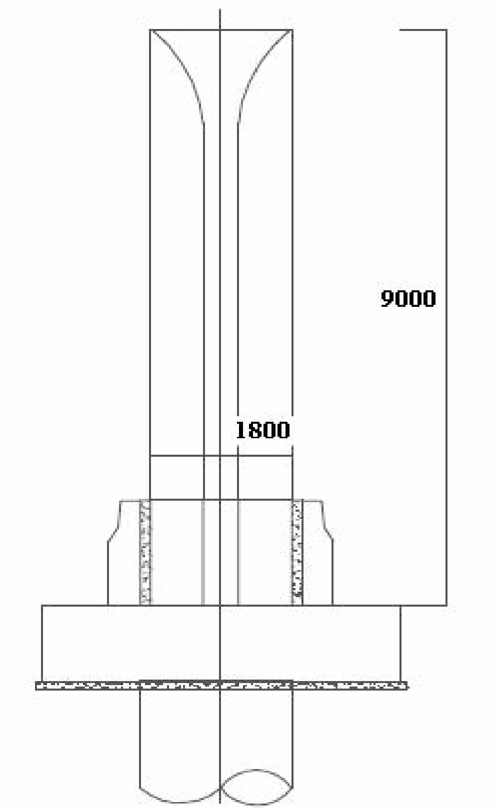


Fig.4.1 Sketch of typical mono-pier (as Delhi Metro Pier)

Modeling is carried out using SAP2000 v.11.0.0. A simple beam element model with discretized node at 1m spacing is generated using the tool. Mass is lumped at the nodes at C.G. of superstructure. Two cases of base fixity are considered. The base is fixed at ground and base with elastic foundation. A section assign to element with given material and loading properties. (Fig. 4.2)

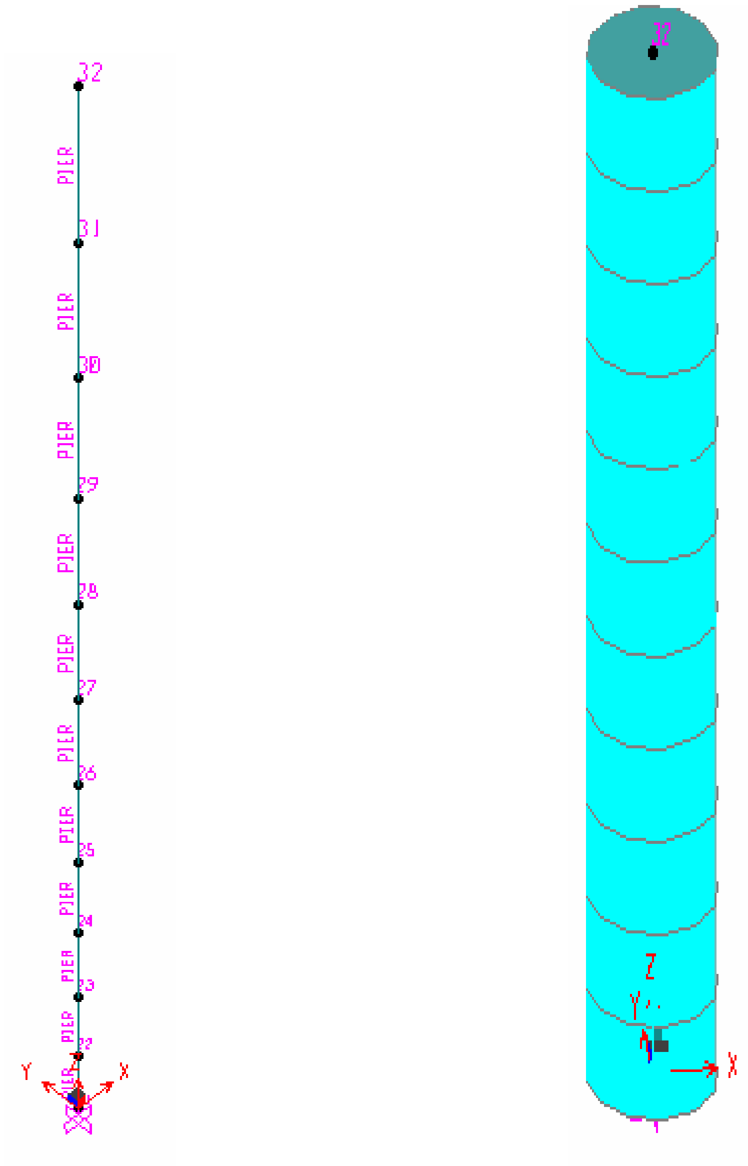


Fig.4.2 Fixed Base pier model with 3D view [16]

The same model is considered with soil-structure effect .In which a Cast-in-drill-hole (CIDH) pile with diameter as of pier modeled up to 10m depth below ground level for studying the influence of soil-structure interaction (SSI). (Fig. 4.3)

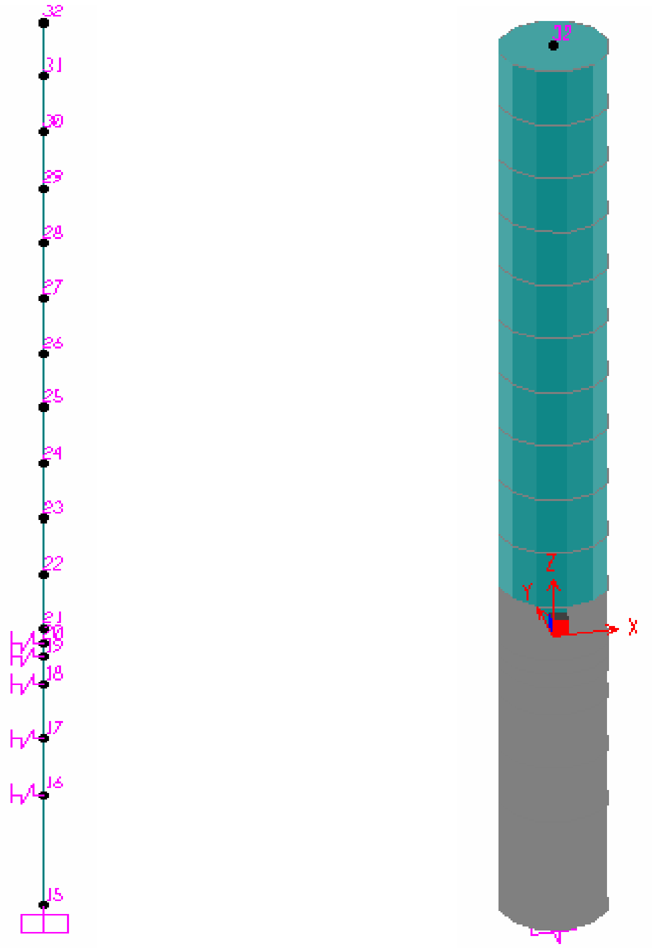


Fig.4.3 Soil structure (SS) model with 3D View [16]

Three type of soil are taken into consideration. The three soil conditions of IS1893/IRC6 are represented by relative subgrade modulus as given in the Table 1.

Sr. No.	Nature of Soil	Designation	Modulus of Subgrade Reaction , ks (MN/m^3)
1	Rocky soil (coarse crushed stone)	Type I	225
2	Medium soil (very well compacted sand and clays soil with sand)	Type II	90
3	Soft soil (Fine or slightly compacted soil)	Type III	15

Table 1 Modulus of Subgrade Reaction for different type of soil [14]

Several level of sophistication can be used in modeling the stiffness of foundation soil. A method of approach based on Winkler springs [11] (Fig. 4.4) is most reliable for general purpose dynamic analyses. At the first stage of study a simple spring approach was adopted for modeling the soil stiffness. In the elastic range the discrete soil springs with the stiffness k and is dependent on soil type, depth z (effective influence length) and pile diameter D was specified.

The Winkler soil reaction modulus or spring constant 'k_s' along the length of well foundation of diameter 'D' is determined as follow:

k=D. k_s **Equation 4.1**

ki= k_s *** Zi * Bi * D** **Equation 4.2**

where Bi= $(Z_{i+1} - Z_{i-1})/2$

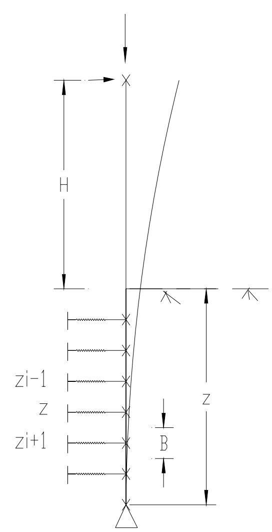


Fig. 4.4 Soil stiffness Model

In the modeling of soil flexibility the portion of the pile below the bed level is first divided into no. of elements. The spring with spring constant proportionate to the depth of location of the node below bed level calculated from (Equation 2) is assigned to the node to represents the soil stiffness. They are given in Table 2.

	i (node)	Z (m)	Bi (m)	ki (kN/mm)
Type I	20	0.25	0.25	25.31
	19	0.5	0.5	101.25
	18	1	0.75	303.75
	17	2	1	810
	16	3	1.5	1822

	i (node)	Z (m)	Bi (m)	ki (kN/mm)
Type II	20	0.25	0.5	22.5
	19	1	1	180
	18	2	1	360
	17	3	1	540
	16	4	1.5	1080
	15	6	2	2160
	14	8	2	2880

	i (node)	Z (m)	Bi (m)	ki (kN/mm)
Type III	20	0.25	0.5	3.38
	19	1	1	27
	18	2	1	54
	17	3	1	81
	16	4	1.5	162
	15	6	2	324
	14	8	2	432

Table 2 Calculation of soil stiffness for three soil types [11]

4.1.2. Linear Static analysis using Response spectra method

A Response Spectra analysis is carried out using IS1893/IRC6:2002 [2,3,4] spectra and methodology with varying values of response reduction factor, R. The Zone factor (Z) and Soil Type I (Rocky soil), Type II (Medium soil) and Type III (Soft soil) as per IS 1893:2002 are and modeled with the SAP2000 analysis tool. The model is acted upon by different intensity earthquake and corresponding roof displacement and the base shear are assessed.

Parametric variation

Response Reduction factor (R)	: 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0.
Soil type (IS 1893 : 2002)	: I, II, III
Zone factor (Sa/g)	: 0.1, 0.16, 0.24, 0.36
Base fixity	: Pier with fixed foundation at ground and Pier with pile foundation (SSI)

IS 1893:2002 response spectra (Demand spectra) has been used for all cases.

4.1.2. Non-Linear Static analysis using Capacity Spectrum method

In this method, the nonlinear static analysis of mono-pier with fixed base and with pier-pile foundation has been carried out by using capacity spectrum method as per SAP2000 for different values of response reduction factor and different ground acceleration. The hinges are assign to the mono-pier model at the bottom of fixed base and throughout the pile depth below ground level adjacent to springs . The Life safety performance level are chosen as a limit of failure. That step noted and the respective performance point in terms of base shear and roof displacement are set up as limiting criteria for performance based design. The typical capacity curve for fixed base and mono pier-pile foundation model are discussed in next chapter, Result and Discussion.

4.2 Seismic Performance of multi bent bridge

A multiple bent bridge structure as shown in the Fig. 4.5 and Fig. 4.6 are considered.

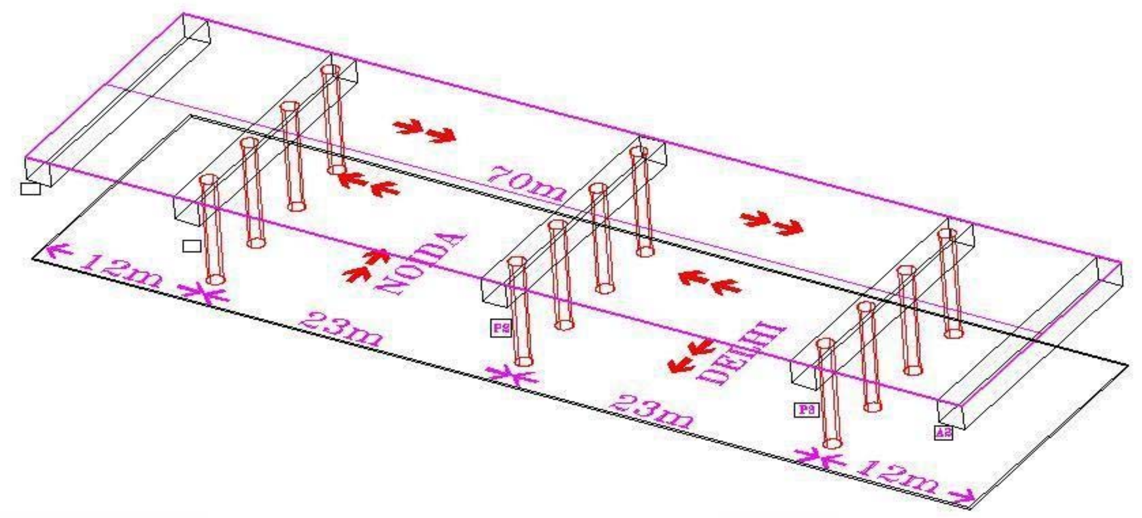


Fig. 4.5 Isometric sketch Approach Bridge near DND flyover

Ths structure may have resemblance with typical short span bridge as Approach Bridge near DND flyover port 3 (Sarai-Kalekha end) having four span with precast I-girder bridge lying enroute Delhi to Noida , 8 km away from Ashram chowk. A structural dimensions and geometry clearly shown in the Fig. 4.5, Fig. 4.8.

Fig. 4.6 Pictorial view ofApproach way near DND flyover

Fig. 4.7 Pictorial view of Bent beam-Pier system

The following geometrical and material properties has been considered.

Bent beam :

1.0 m x 1.5m rectangular RCC beam

Concrete M25 grade and Steel FE 415 grade.

Pier :

1.3 m diameter circular RCC section, Concrete M45 grade and Steel FE 415 grade

Longitudinal Reinforcement 25 nos. of 25mm diameter bar.

Transverse reinforcement 12mm dimeter spiral at 115mm c/c spacing.

Superstructure details :

Precast I-Girder section, there are 27 no. of girders each having 0.6 m^2 cross sectional area. Deck slab 150mm thick. Dead weight from crash barrier, meridian and wearing course also considered.70 R 2-lane live load taken.

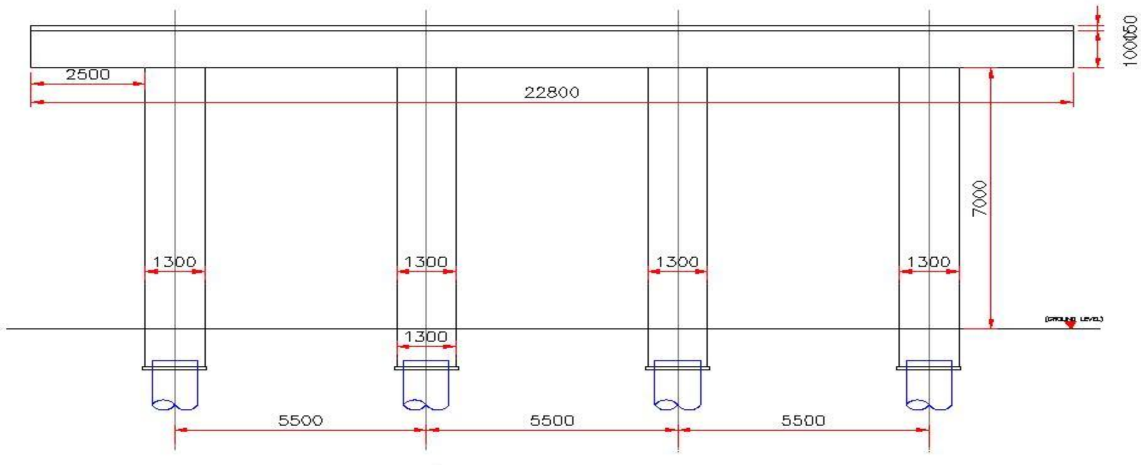


Fig. 4.8 Cross sectional sketch of Bent Beam-Pier frame System

For non-linear static analysis purpose, a simple interior Bent beam- pier frame is modeled by using SAP2000. Mass of 300 ton is lumped at each top node of beam-pier connection in three Global axes.

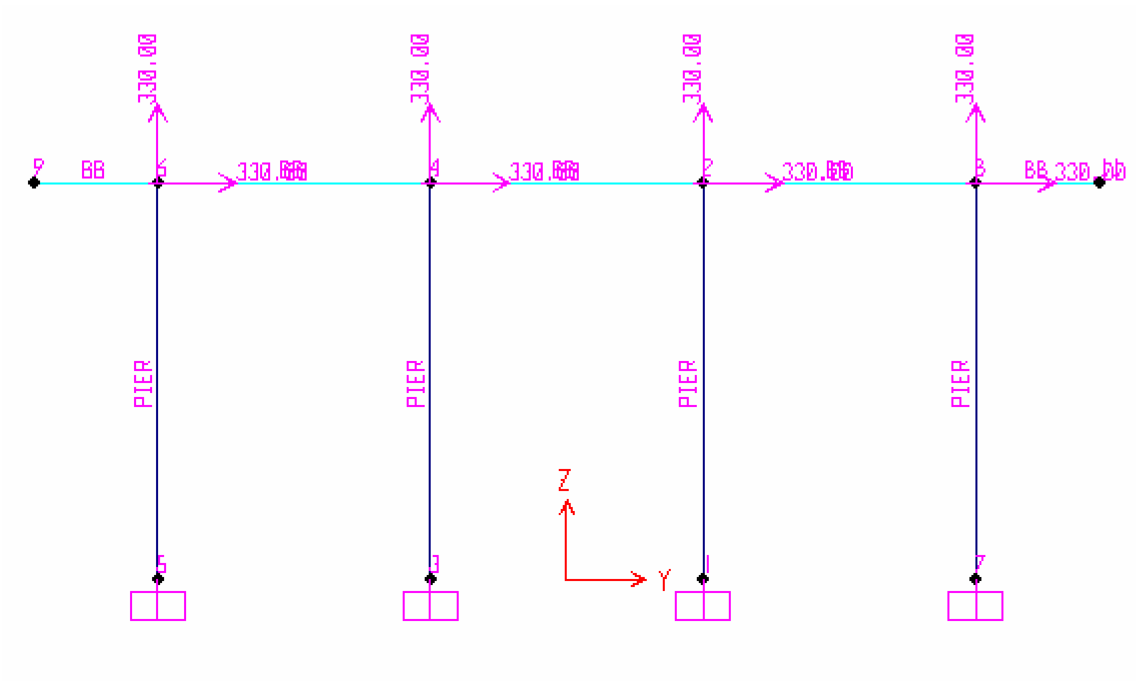


Fig. 4.9 A Beam-Pier Frame model [16]

Nonlinear static analysis (push over) using ATC 40's capacity spectrum method is carried out to find out the performance point. Typical capacity curve etc. is discussed in next chapter- Result and Discussion.

First Bent beam-pier model is studied for nonlinear analysis and later the full bridge. Capacity spectrum method by ATC-40 is used to evaluate structural performance under seismic demand. A displacement based push technique is used of 0.24g and 0.36g for ground acceleration for soil Type II and Type III in both longitudinal and transverse directions. In the case demand spectra from IS1893:2002 used with varying value of Response reduction factor(R).

Hinges are assigned to Pier base and bent beam-pier joint vicinity as this location are more prone to failure. A case of auto hinge generation with default properties is adopted in the study for more precise failure evaluation. Two performance objective chosen as per MCEER studies are given in Table 3 and performance objective chosen is Life safety.

Parameter	Life Safety	Immediate Occupancy
Column plastic hinge rotation	0.035 rad	0.01 rad
Vertical offset in girders	0.2 m	0.03 m

Table 3 Proposed damage states for given parameter according to required objective (MCEER 2001)

4.3 Seismic Performance of full span bridge

Nonlinear static analysis of full span bridge studied. The dimensional details shown in Fig. 4.10 A Bridge wizard module in SAP2000 is used to model complete bridge. A push over analysis carried over whole bridge and results get obtained.

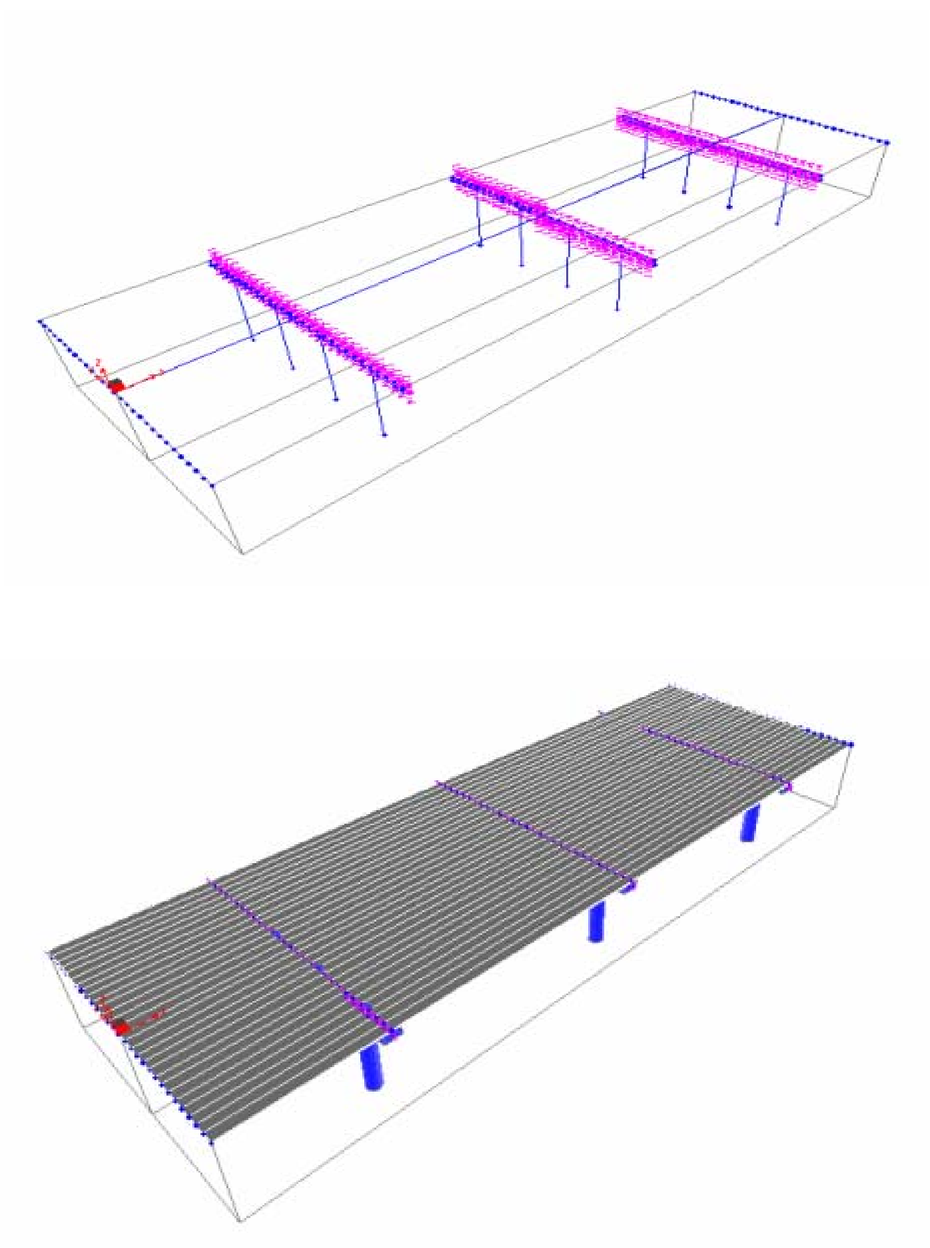


Fig. 4.10 A full bridge model using SAP2000 [16]

Based on numerical study carried out on typical short and medium span bridges, important results are presented and discussed.

5.1 Linear static analysis (Response spectra) results for mono-pier system

The relations between Base shear and roof displacement has been obtained from Linear static analysis and is presented in the Fig. 5.1.

For the Pier model with same material properties, by varying R base shear reduces but at the same time Ductility of material in terms of roof displacement is varies in 25-30%. (Fig. 5.1)

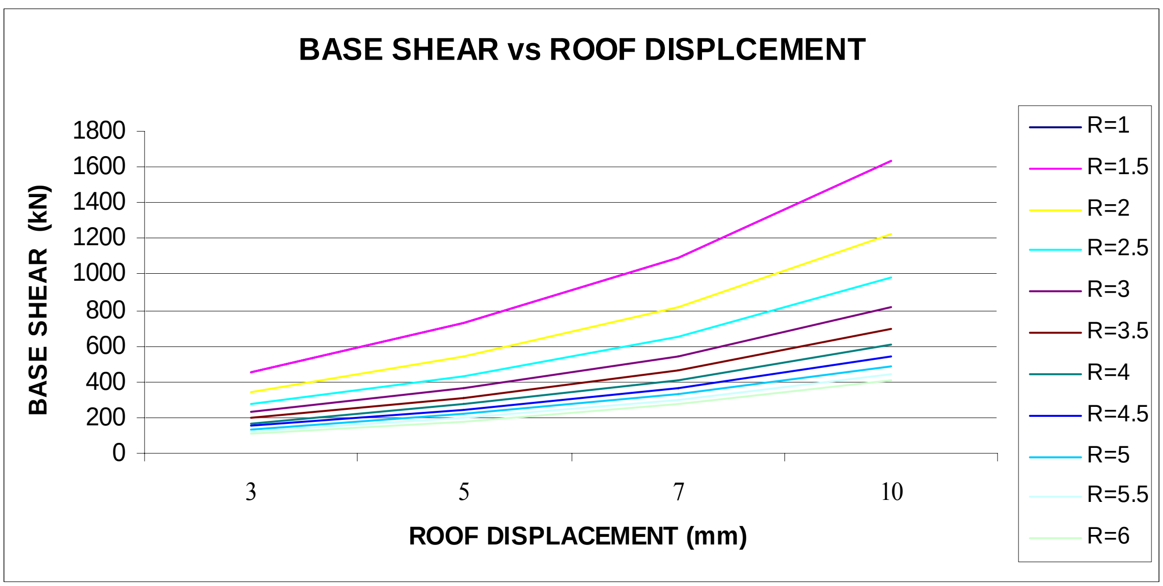


Fig. 5.1 Base shear vs Roof displacement (Response reduction factor, R varies)

For the different Earthquake ground motion for particular type of soil, it observed that (Fig.5.2) as increase in the R value increases the roof displacement reducing considerably. As R value increases there is corresponding reduction in the base shear also relative to particular earthquake intensity.

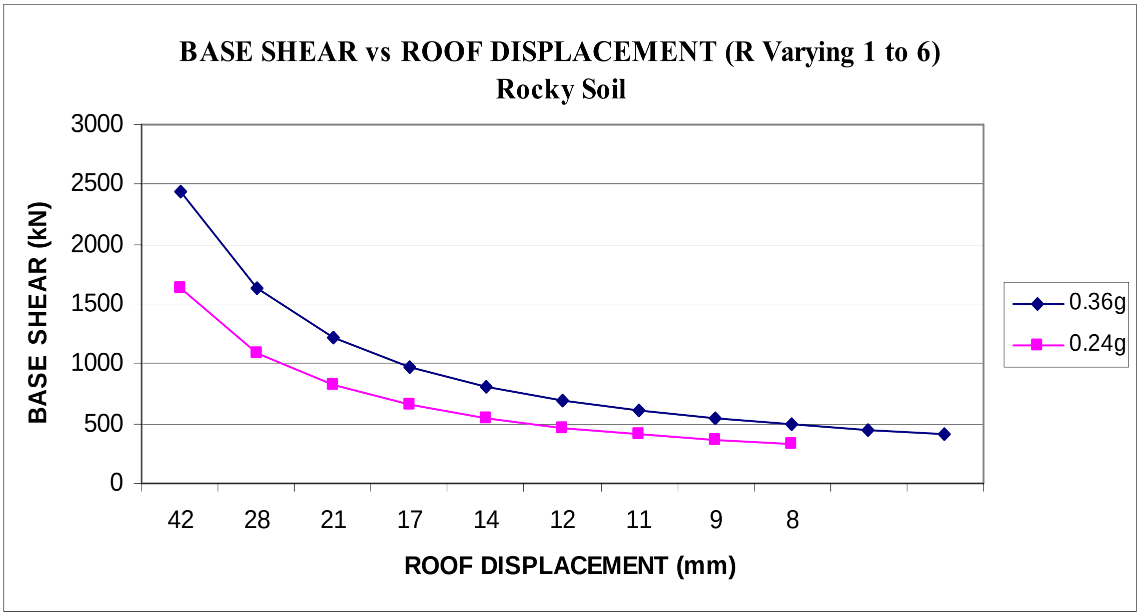


Fig. 5.2 Base shear vs Roof displacement (Ground acceleration varies)

For different soil types, Type III soil (soft soil) is quite flexible, it causes the maximum base shear and roof displacement in other similar conditions (Fig. 5.3 and Fig. 5.4) . As response reduction factor R increases, base shear reduces and indicate weak influence of soil types for higher 'R'.

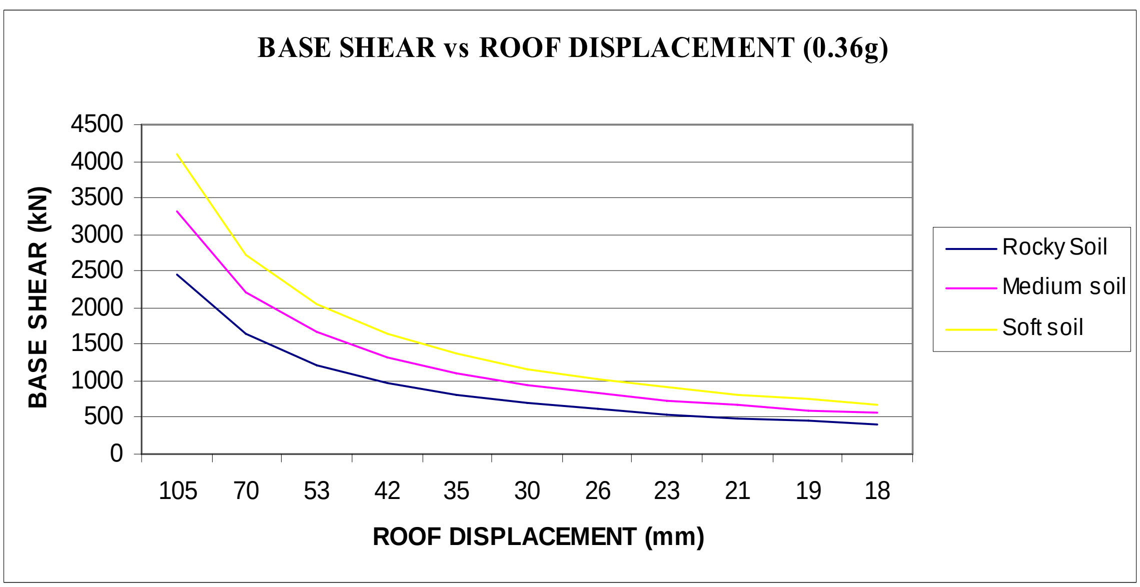


Fig. 5.3 Base shear vs Roof displacement (Soil type varies)

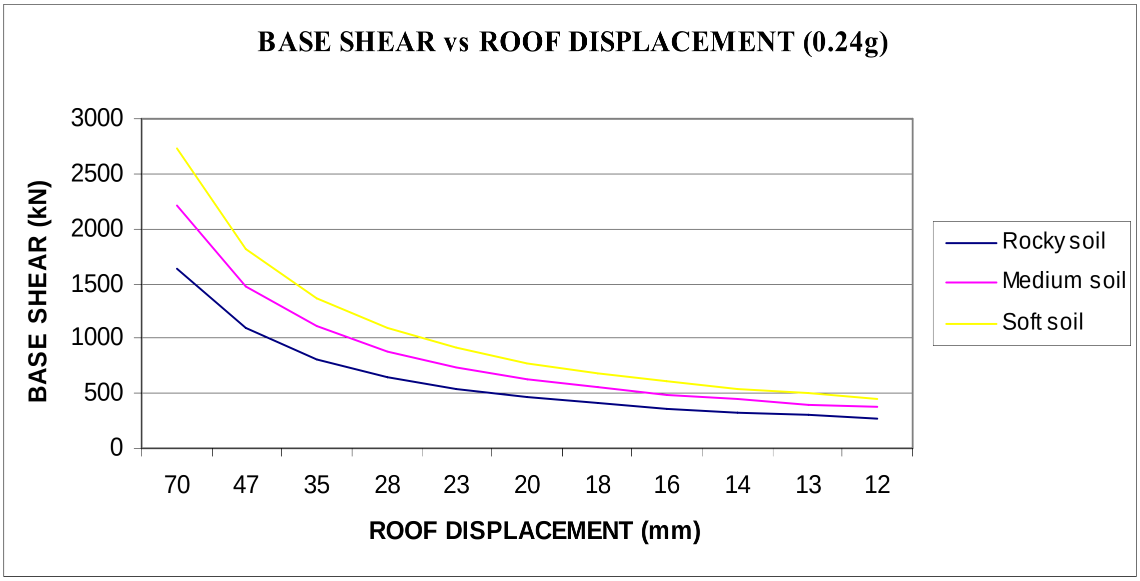


Fig. 5.4 Base shear vs Roof displacement (0.24g , Soil type varies)

5.2 Linear static analysis (Response spectra) results for mono-pier with elastic foundation system

A same structure is taken with soil-structure modeling. A spring with translational stiffness is assigned the pile up to its depth for studying the behavior of structure.

For particular value of R , an adopted response spectra produces effect in both longitudinal and transverse direction as the soil acts in flexible mode. (Fig.5.5)

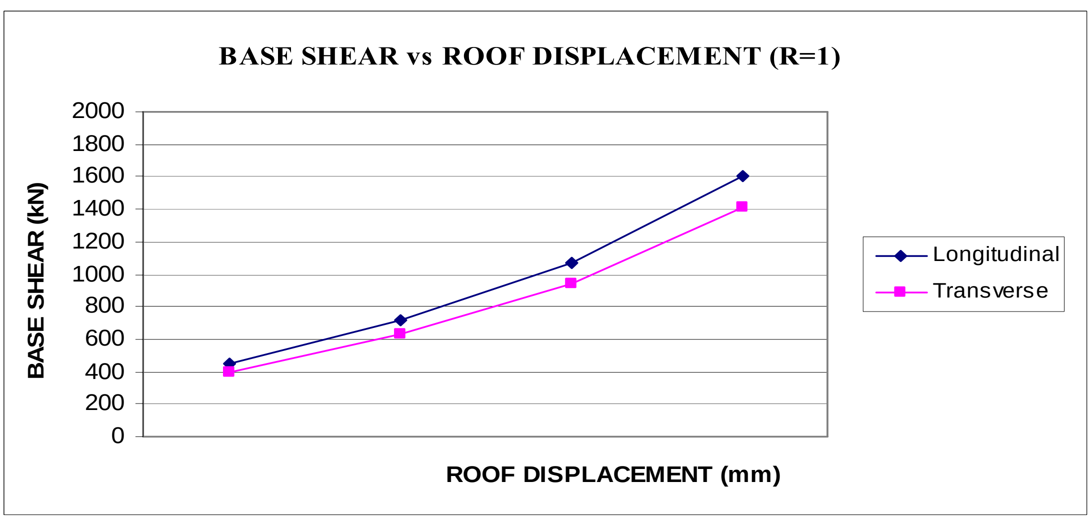


Fig. 5.5 Base shear vs Roof displacement in transverse and longitudinal direction

5.3 Comparison of mono-pier with fixed foundation and with elastic foundation

The following output for the different soil with similar/same ductility level, geometric and material properties, shows that for the given earthquake soil-structure interaction (SS) model gives less base shear but higher displacement for the same compared to fixed base model (LL) and are given in Table 4

SOIL I-LL	R=2.5	SOIL-I-SS	R=2.5
Base Shear	Roof displacement	Base Shear	Roof displacement
272.177	6.999	178.081	10.7
435.484	11.199	284.93	17.119
653.226	16.799	427.396	25.67
979.838	25.198	641.093	38.52

(A)

SOIL I-LL	R=2.5	SOIL-I-SS	R=2.5
Base Shear	Roof displacement	Base Shear	Roof displacement
272.177	6.999	157.129	12.21
435.484	11.199	251.407	19.45
653.226	16.799	377.111	29.31
979.838	25.198	565.666	43.97

(B)

Table 4 Relation for (A)longitudinal and (B)transverse direction for Mono-pier foundation effect (soil type I)

The ratio of longitudinal roof displacement to transverse one is nearly about 0.87 showing that the mass is excited in both longitudinal and transverse direction (shown in Fig. 5.6 and Fig. 5.7). This parameter further depends upon the pile with elastic foundation system flexibility.

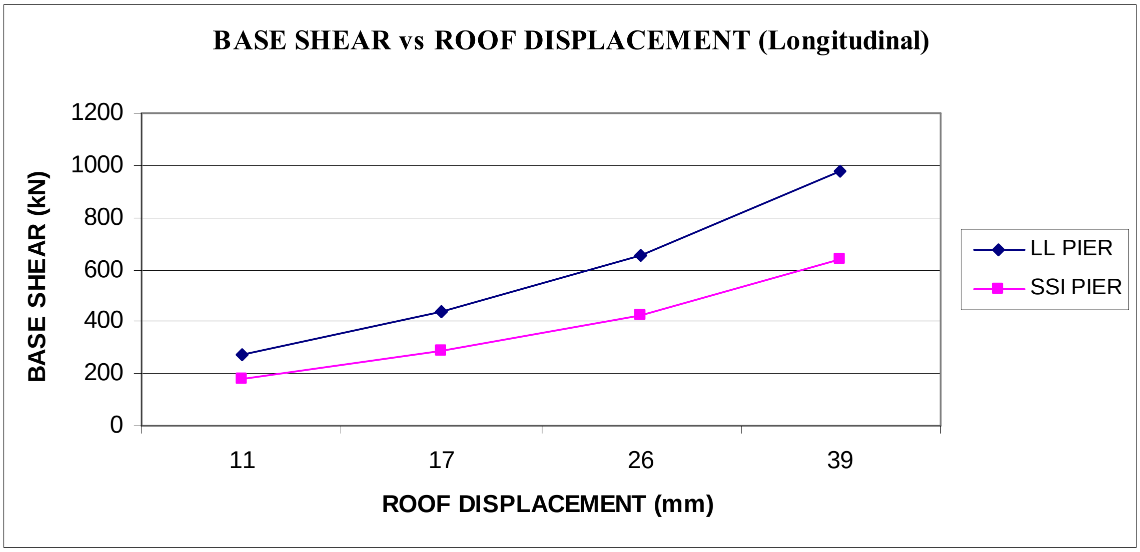


Fig.5.6 Base shear vs Roof displacement (Soil-structure interaction-Longitudinal direction)

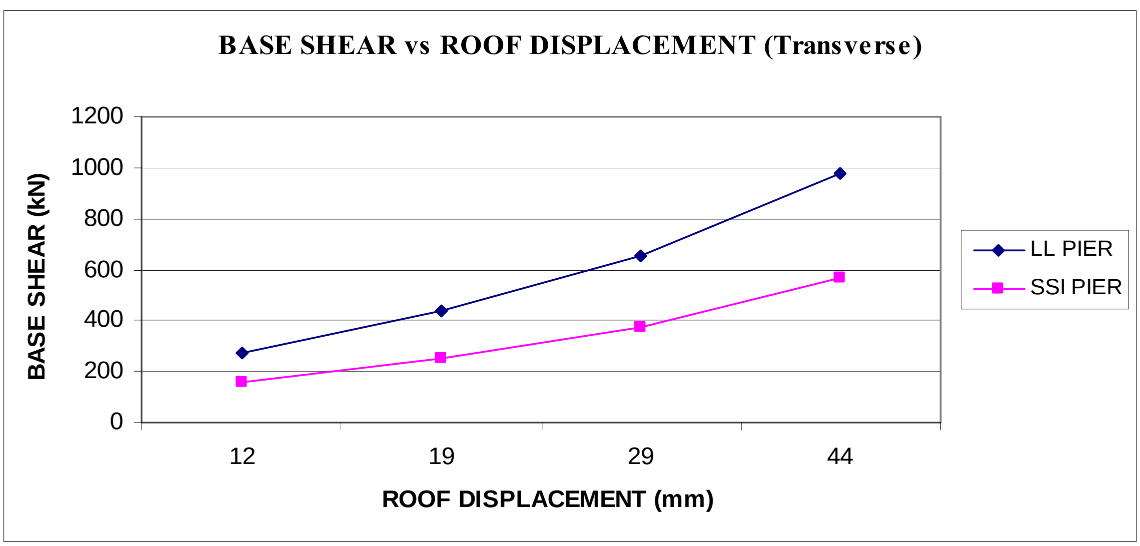


Fig. 5.7 Base shear vs Roof displacement (Soil-structure interaction-Transverse direction)

For the given ground motion and soil type , when fixed base (LL) and soil-structure (SS) model compared with varying R factor, it shows that for a higher R value , the base shear in SS model is less than that of LL model , at the same time roof displacement is more. (Fig. 5.8 and Fig. 5.9)

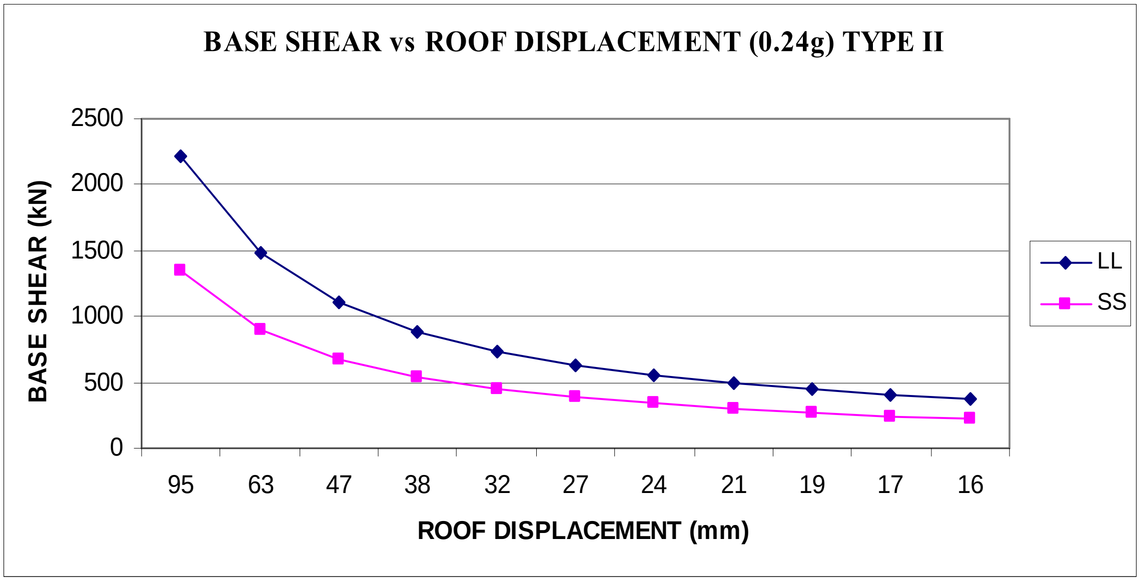


Fig. 5.8 Base shear vs Roof displacement (Soil-structure interaction effect for 0.24g)

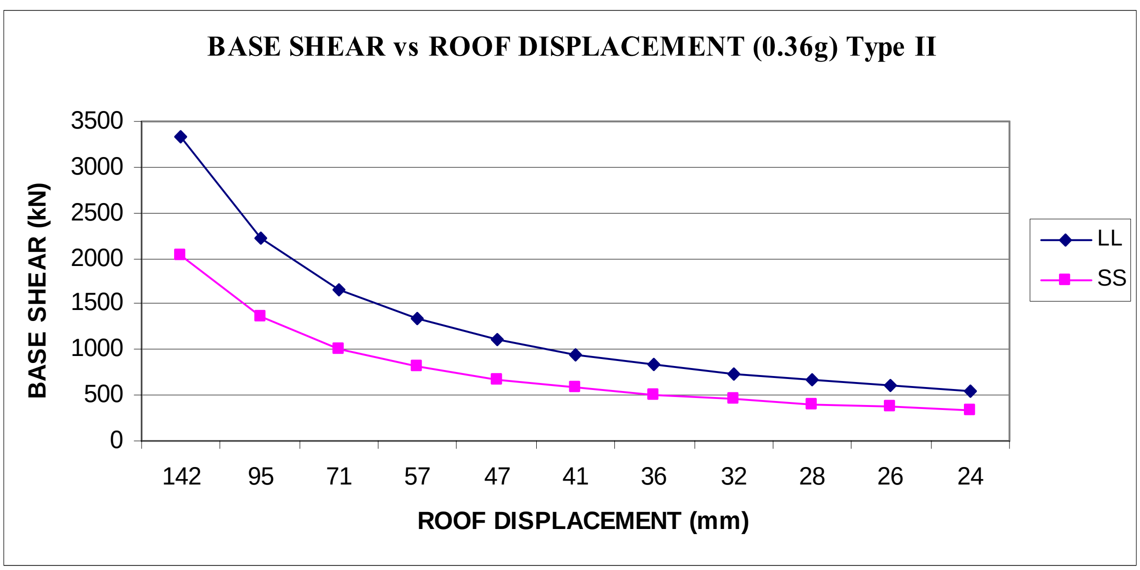


Fig. 5.9 Base shear vs Roof displacement (Soil-structure interaction effect for 0.36g)

5.4 Nonlinear static analysis of mono-pier model using capacity spectrum method

Non-linear static analysis (Push) has been compared with response spectrum for mono-pier with fixed base (Soil type II) under similar site condition for different values of R and a respective Base shear and roof displacement and is shown in the Table 5. It, however, shows that the respective values are nearly equal.

Push in Longitudinal direction for Soil Type II				
Seismic demand	**0.24g**		**0.36g**	
R	Base Shear (kN)	Roof displacement (mm)	Base Shear (kN)	Roof displacement (mm)
1	1278.71	46.000	1289.67	65.114
2.5	912.46	22.213	1271.03	33.350
4	566.63	14.110	849.95	21.210
5	452.69	11.240	679.04	17.001
6	375.71	9.100	563.55	14.155
Response spectra in Longitudinal direction for Soil Type II				
Seismic demand	**0.24g**		**0.36g**	
R	Base Shear (kN)	Roof displacement (mm)	Base Shear (kN)	Roof displacement (mm)
1	1218.19	47.057	1227.21	65.586
2.5	888.18	22.846	1332.27	34.269
4	555.30	14.284	832.95	21.425
5	443.63	11.411	665.45	17.117
6	370.00	9.517	555.00	14.276

Table 5 Comparison of Base shear vs Roof displacement by Response spectra and Capacity spectrum method for mono-pier with fixed base

Non-linear static analysis (Push) has been carried out for mono-pier with elastic foundation under similar site condition for different values of R, and a respective Base shear and roof displacement compared with the results by linear static-response spectrum analysis is shown in the Table. 6 It shows that the respective values are quite similar. The percentage variation is about 40%.

Push in Longitudinal direction for Soil Type II				
Seismic demand	**0.24g**		**0.36g**	
R	Base Shear (kN)	Roof displacement (mm)	Base Shear (kN)	Roof displacement (mm)
1	1249.00	85.000	1260.23	114.000
2.5	912.46	36.000	852.233	55.000
4	566.63	23.000	529.00	34.000
5	452.69	18.000	422.81	27.000
6	375.71	15.000	350.92	22.000
Response spectra in Longitudinal direction for Soil Type II				
Seismic demand	**0.24g**		**0.36g**	
R	Base Shear (kN)	Roof displacement (mm)	Base Shear (kN)	Roof displacement (mm)
1	1350.856	94.720	2026.285	142.08
2.5	540.894	37.920	811.341	56.89
4	338.174	23.710	507.26	35.57
5	270.171	18.940	405.257	28.41
6	225.327	15.800	337.99	23.7

Table 6 Comparison of Base shear vs Roof displacement by Response spectra and Capacity spectrum method for mono-pier with pile foundation. (soil II)

Results of Nonlinear analysis of pier with fixed base compared with pier with elastic foundation with similar site and demand condition with varying response reduction factor, R, it shown that respective values of base shear and roof displacement are higher for pier with elastic condition as compared to fixed one .

The roof displacement for elastic foundation system shown to be 75.2% higher than that of pier with fixed base. The base shear for pier with elastic foundation system is found to be less by 30-38% than that of pier with fixed base with increasing R values.

Pushover analysis for soil type II under 0.36g demand for mono-pier with fixed base		
Seismic demand	**0.36g**	
R	Base Shear (kN)	Roof displacement (mm)
1	1289.67	65.114
2.5	1271.03	33.350
4	849.95	21.210
5	679.04	17.001
6	563.55	14.155

Pushover analysis for soil type II under 0.36g demand for mono-pier with pile foundation		
Seismic demand	**0.36g**	
R	Base Shear (kN)	Roof displacement (mm)
1	1260.23	114.000
2.5	852.233	55.000
4	529.00	34.000
5	422.81	27.000
6	350.92	22.000

Table 7 Comparison of Base shear vs Roof displacement by Capacity spectrum method for mono-pier with fixed base(LL) and mono-pier with elastic foundation (SS)system.

For checking the reliability of Results by SAP2000, the manual calculation for mono-pier system by IRC method done and it found that a very much resemblance in the results.

Case: Mono-pier system with fixed base.

Soil type II (Medium soil) , Zone factor (Z) - 0.24, Response reduction factor (R) – 1.5

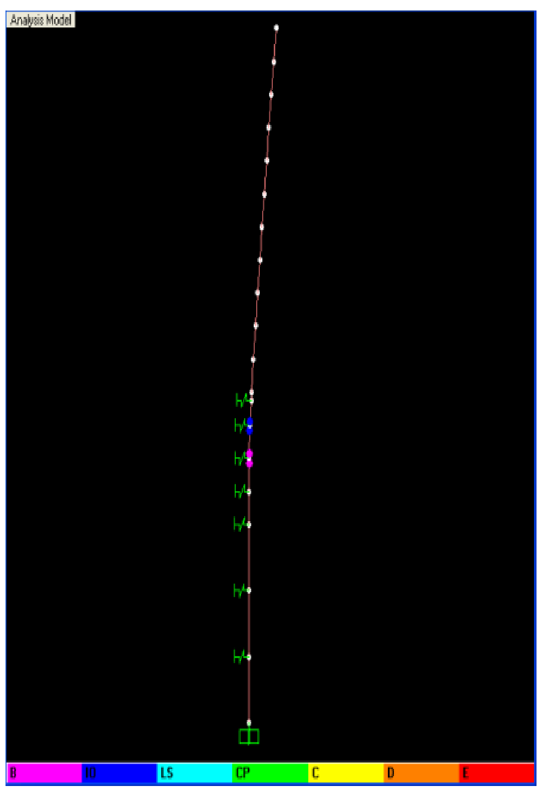
Time period obtained using SAP2000 modeling is 0.91 sec. and from IRC is 0.81 sec.

Base Shear obtained is 1098 kN by SAP2000 and 1160 kN by IRC method.

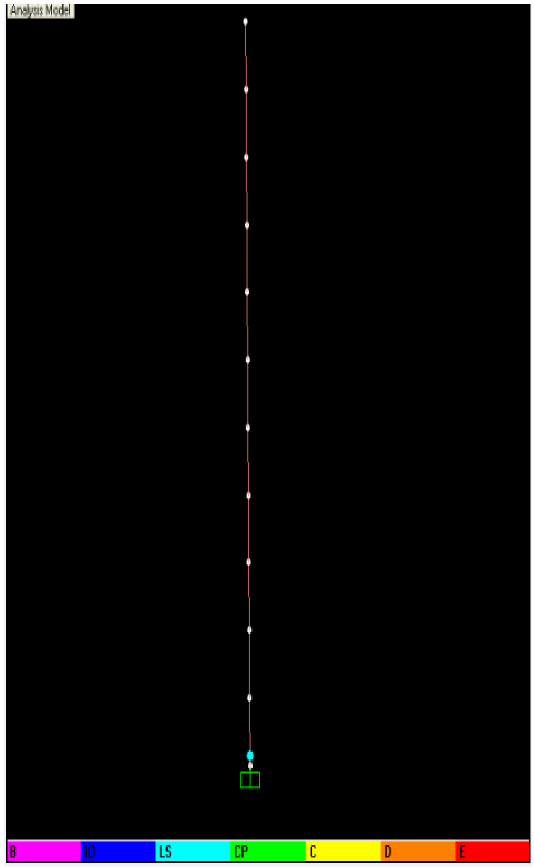
5.5 Hinge Formation

Push over analysis for the two system shows that the Life safety (LS) as a performance level chosen for pier achieved in step 23 and Immediate occupancy (IO), a performance level for foundation achieved in step 11 shown in Fig. 5.10, which indicate Mono pier – pile system has more ductile behavior than that of fixed base system provided proper reinforcement detailing adopted.

Also the hinge formation are more liable to occurs near ground level but in elastic pile first than fixed base pier .This highlighting need of modeling soil with structure.



(A)



(B)

Fig. 5.10 Hinge formation in (A) Mono-pier with elastic foundation (B) Mono-pier with fixed base

Performance level : LS- life safety shown in sky colour dot.

IO- immediate occupancy shown in dark blue colour dot.

5.6 Capacity curve for Mono-pier fixed base and Mono-pier with elastic Foundation

Under the similar condition , the mono-pier with fixed base gives higher base shear but lower roof displacement at performance point compared to pier with elastic foundation is shown in Fig. 5.11 and Fig.5.12

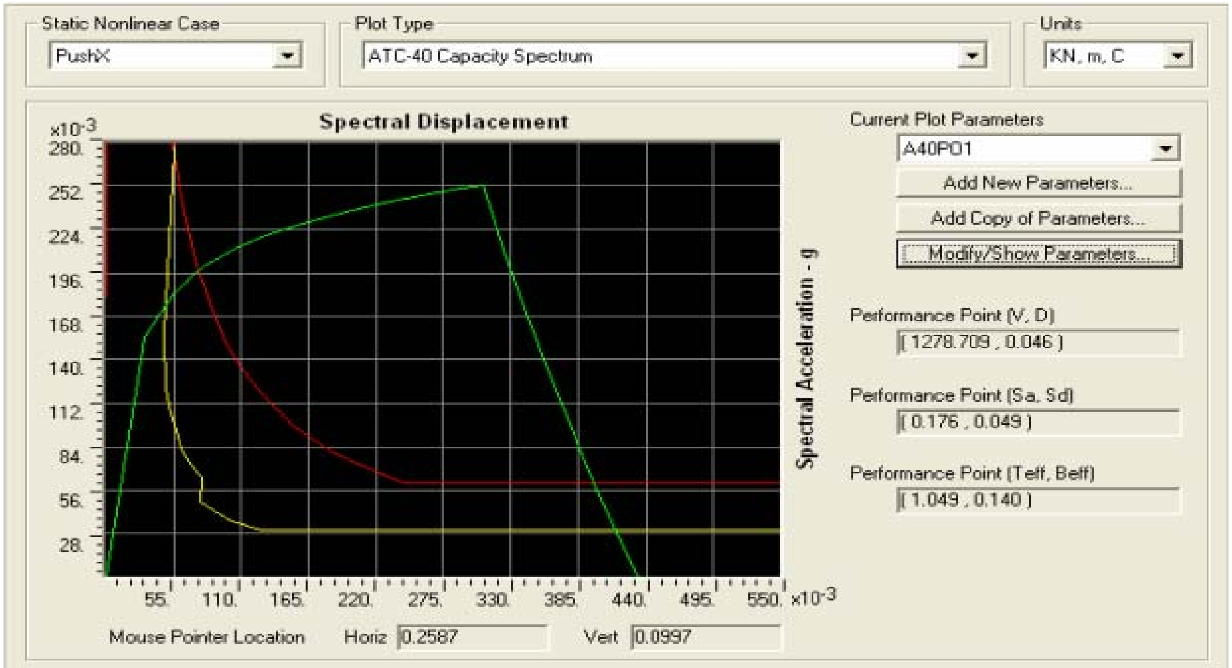


Fig. 5.11 Push over curve for mono-pier Model with fixed base

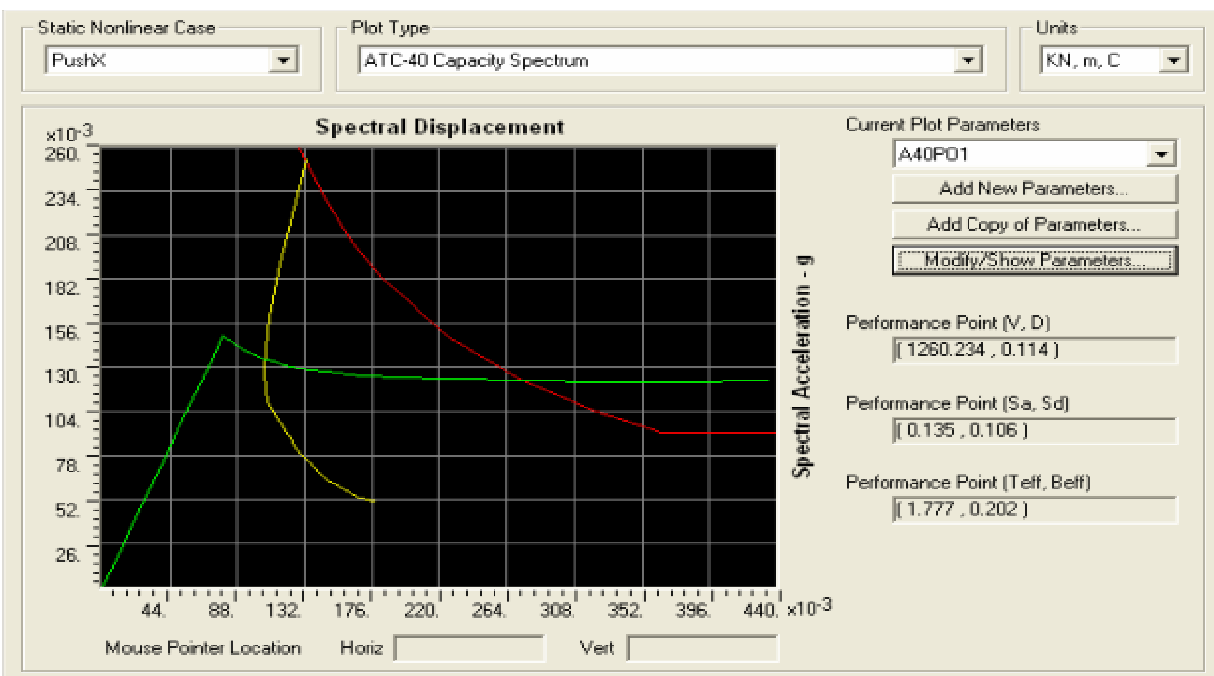


Fig. 5.12 Push over curve for mono-pier Model with elastic foundation

(Notation: Green – Capacity spectra, Red- Demand curve , Yellow- ADRS demand spectra, performance point- intersection of capacity and demand spectra)

5.6 Nonlinear static analysis of bent beam – pier frame model

This values are observed to be similar to the values given by the response spectra analysis. Also the results indicate here that at the transverse mode structure has to carry large amount of shear forces for comparatively small roof displacement compared to single column pier. The results show in Fig. 5.13

Push in Longitudinal direction for Soil Type II				
Seismic demand	**0.24g**		**0.36g**	
R	Base Shear (kN)	Roof displacement (mm)	Base Shear (kN)	Roof displacement (mm)
1	1849.85	43	2093	83
2.5	1687.95	17	1725.96	23
4	1375.88	12	1684.94	17
5	1070	9.3	1605	14
6	888.13	7.65	1332	11

Push in Transverse direction for Soil Type II				
Seismic demand	**0.24g**		**0.36g**	
R	Base Shear (kN)	Roof displacement (mm)	Base Shear (kN)	Roof displacement (mm)
1	3537.44	26	3647.53	47
2.5	2605.92	8	3093	13
4	1721.91	5	2499.5	7
5	1380.85	4	1979.52	6
6	1165.86	3.36	1673	5

Fig. 5.13 Values of Base shear and roof displacement by capacity spectrum method applied in longitudinal and transverse direction.

For the longitudinal push case , the first hinge formation seen to be at intermediate pier as shown in Fig. 5.14 and Fig. 5.15 . It indicates that the first hinge and the Life safety level achieved first at intermediate pier so they are weakest one in this case.

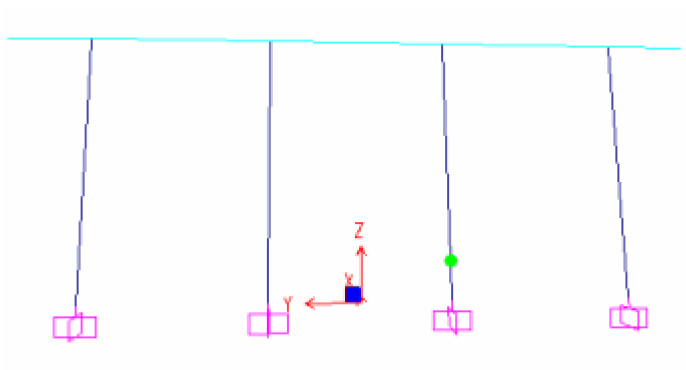


Fig. 5.14 First hinge location for Longitudinal Push case

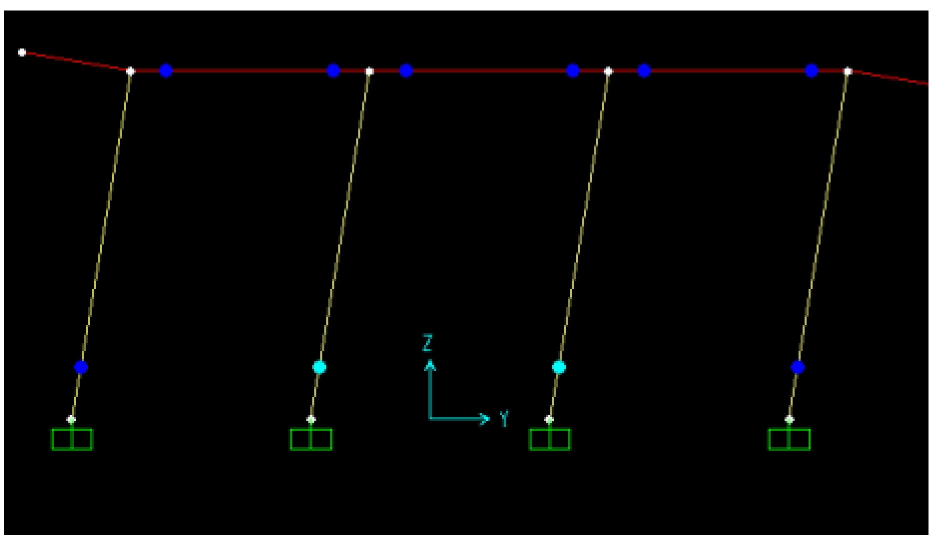


Fig. 5.15 Life safety level location (sky blue colour dots) for Longitudinal Push case

For the transverse push case , the first hing formation seen to be at outer cap-beam portion adjacent to pier-bent beam exterior joint pier as shown in Fig. 5.16 but the life safety level achieved first by the intermediate piers as that of in longitudinal case.

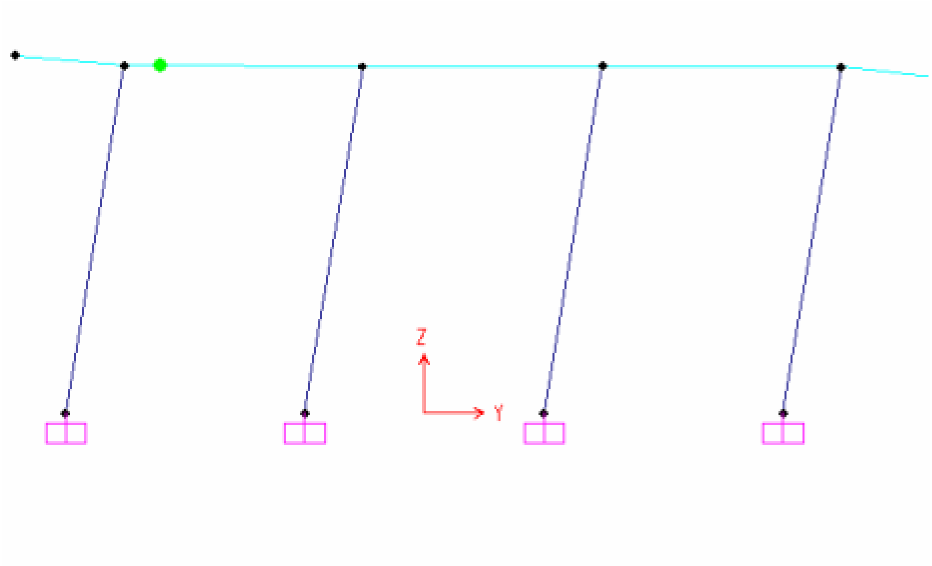


Fig. 5.16 First hinge location for Transverse Push case

For full bridge, under longitudinal push over, the exterior column were seen to be weak as hinges formed in the initial steps. Also the trasverse push does not effecting much on a structural capacity. It has higher strength for transverse push. (Fig. 5.17)

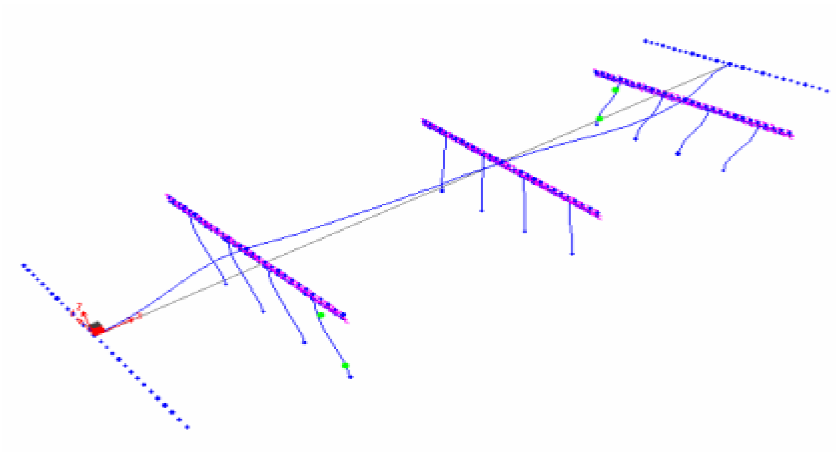


Fig. 5.17 First hinge location for longitudinal Push case for full bridge condition

6.1 Conclusion

The seismic study carried out on mono-pier, multibent and full bridge system with and without foundation, the important conclusions can be highlighted as follows:

1. To allow large displacement, greater base shear can be permitted.
2. The induced base shear is smaller for a higher response reduction factor ‘R’, however, its choice is based on appropriate (ductile) detailing pertaining to performance objectives.
3. Consideration of elastic foundation suggests that greater responses at base are induced in case of soft soil and smaller responses in case of rocky soil. The fixed base condition may also underestimate the magnitude of responses.
4. In case of multiple bent , there is difference of response in longitudinal and transverse direction suggesting possible optimization to reduce the cost.
5. Assessment of capacity and demand using in-elastic displacement approach provides an insight of modeled bridge and its components comparable to stated performance goals.
6. Pushover analysis can be used successfully to study progressive collapse by obtaining formation of hinges (in ductility design) and their sequence of occurrence helps in achieving stated performance goals of bridges.
7. As the number of bents increases, the complexity of modeling also increases which however requires appropriate choice of parameters even in standard analytical tools e.g., SAP2000. Component wise modeling as well as complete bridge modeling including soil-structure is recommended for clear understanding of the bridge system performance.
8. Many features of complex time history dynamic analysis can be captures by relatively easy technique of pushover.

6.2 Future Study

1. The study can be extended to consider various boundary conditions of bridges like rocker-roller vs integral bridges.
2. Influence of base isolation on seismic demand vs capacity of bridge piers.
3. Influence of non-linear or variable damping on progressive collapse behavior.

1. Tondon Mahesh, 'Earthquake Resistant Design of Bridges', Journal of Indian Road Congress, Volume 62-3, paper no.479,December,2001.
2. IS:1893-1984, Criteria for Earthquake Resistant Design of Structures, Bureau of Indian standards, New Delhi.
3. IS:1893-2002, Criteria for Earthquake Resistant Design of Structures, (fifth revision),Bureau of Indian standards, New Delhi.
4. IRC:6-2000, Standard Specifications and Code of Practice for Road Bridges, Section II- Loads and Stresses, (Fourth Revision), Indian Road Congress, New Delhi,1985.
5. Agrawal Pankaj and Shrikhande Manish, 'Earthquake Resistant Structures' A Book from Prentice Hall of India Private Limited, New Delhi., 2006.
6. Prakash Vipul, 'Whither Performance-based Engineering in India?', ISET Journal of Earthquake Technology, Paper No. 447, Vol. 41, No. 1, March 2004, pp. 201-222.
7. www.nicee.org IIT, Kanpur.
8. Murthy C.V.R. and Jain S.K., 'A proposed draft for Indian code provision on seismic design of Bridges-Part I: Code,', Journal of structural Engineering,vol.26, Jan 2000.
9. Chen Y. (1996), Modeling and Analysis methods of Bridges and their effect on Seismic Responses, Computer & Structures, vol. 59, pp 81-98.
10. ATC(1996), 'Seismic Evaluation and Retrofit of Concrete Building', Report ATC 40,volume-I, Applied Technology Council, Redwood city, U.S.A.
11. Priestley M.J.N., Seible F., Calvi G.M., 'Seismic Design and Retrofitting of Bridges',John Wiley & Sons Inc.,pp.249-255.
12. Naeim Farzad, The Seismic Design Handbook, Kluwar academic publisher,USA.
13. Abheysinghe Ranjit, Gavaise Evgenia, Rosingnoli Marco, 'Pushover Analysis of Inelastic seismic behaviour of Greveniotikos Bridge' ASCE Journal.
14. Hem slay J.A., 'Design application of raft foundation', Thomas telford,2000.

15. Seneviratna G.D.P.K., Krawinkler H., ' Pros and cons of a pushover analysis of seismic performance evaluation', Engineering structures, vol. 20, Nos 4-6, pp.452-464, 1998.
16. SAP2000: Integrated software for structural Analysis and Design'(http://csiberkeley.com), Computer and Structures, Berkeley, CA, U.S.A