PERFORMANCE BASED SEISMIC ASSESSMENT OF RCC BRIDGE PIER ADOPTING MODIFIED CAPACITY SPECTRUM METHOD

THESIS SUBMITTED FOR PARTIAL FULFILMENT OF THE
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BY

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CERTIFICATE OF RECOMMENDATION

This is to certify that the thesis entitled **"Performance Based Seismic Assessment of RCC Bridge Pier Adopting Modified Capacity Spectrum Method"** has been prepared by **KAHINI KUNDU (**Examination Roll No. – **M6CNE19003**, Registration No. **- 137339 of 2016 - 2017**) for partial fulfilment of the requirement for the award of Master Degree in Construction Engineering (MCE) in Structural Repair and Retrofit Engineering, is a record of research work carried out under my Guidance and Supervision. I hereby approve this thesis for submission and presentation.

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DECLARATION OF ORIGINALITY AND COMPLIANCE OF ACADEMIC ETHICS

I hereby declare that this thesis contains a literature survey and original research work by an undersigned candidate, as part of his **Master of Technology** in **CONSTRUCTION ENGINEERING** (Specialization in Structural Repair and Retrofit Engineering).

All information in this document has been obtained and presented in accordance with academic rules and ethical conduct.

I also declare that, as required by thesis rules and conduct, I have fully cited and referenced all materials and results that are not original to this work.

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Abstract:

Over the past few decades, earthquake engineering has seen a paradigm shift in the aseismic design of new structures and assessment of existing structures, from the traditional force-based design (FBD) procedures to the performance-based design (PBD) procedures. The newly developed performance-based earthquake engineering (PBEE) provides a framework to quantitatively assess the seismic risks of a structure and explicitly consider its seismic performance in the design process.

Existing bridges are generally not conforming to the current stipulations of seismic design and detailing rules and thus are exposed to a significant risk when subjected to an earthquake or earthquake sequence. Collapse of a bridge would not precisely endanger human lives, but it could significantly delay the post-earthquake mitigation and may cripple the economy of a state or country depending upon the severity of earthquake. The modern seismic code for bridges was introduced relatively recent in India. Therefore, a large number of bridges in India are very vulnerable to design level shaking complying to the current design standard seismic provisions. A systematic evaluation of all the bridges according to their importance in local and national level are required to be carried out. Subsequently a development methodology to improve the seismic resilience of the bridges and overall road network in India is quite significant.

The present study evaluates the seismic performance of reinforced concrete bridge piers in the context of performance-based earthquake engineering. Since the Indian bridge design standards do not explicitly provide any performance requirements against any earthquake hazard, current international practices are followed to define the performance requirements in this study. The 2001 Bhuj earthquake was instrumental to embrace the modern seismic design from age-old seismic design practice. The objective of the present study is to evaluate the seismic performance of RCC bridge piers of different heights designed using post 2000 standard. In order to show the limitations in seismic performance, the same piers designed in accordance with the pre 2000 seismic design standards were also assessed. A parametric study was undertaken to understand the seismic performance of a tall and a short pier. This thesis considered the most commonly used circular pier columns sections. However, the observations of

the present study in general may be are applied to other types of piers sections, namely rectangular section, portal piers, inverted L-piers with minor deviations.

A nonlinear static pushover analysis based seismic assessment was employed to determine the seismic performance of the piers using Modified Capacity Spectrum method in accordance with FEMA 440[22] guidelines which was adopted later in ASCE 41[6]. The member capacities were estimated based on the probable material strengths without any partial safety factors to eliminate conservatism associated in the new designs. The deformation capacity of piers was estimated based on the actual reinforcement detailing and strain limits related to various modes of failure in accordance with FHWA[23] Seismic Retrofitting Manual for Highway Structure for bridges.

Results indicate that columns designed in accordance with the pre-2000 Indian standard, the likely failure mode would be excessive spalling of concrete followed by buckling of longitudinal bars. These columns also show a significant P-delta effect in the capacity curve and thus lead to instability after a design level earthquake. Considering the excessive damage in the column following a design level shaking, repair might not be economically feasible. The columns designed in accordance with the current standard are likely to perform better and could withstand a shaking significantly higher than a design level shaking. The likely failure mode for the modern piers would be low cycle fatigue fracture of longitudinal bars. It is also expected that the modern pier columns would suffer nominal damage under a design level shaking and can be reinstated relatively quickly.

In conclusion, there is a need for seismic assessment of bridges in a more holistic approach to ensure a better seismic resiliency in the major highway routes in India.

Keywords: *Bridge piers, Performance Based Seismic Engineering, Non-linear static procedure, Modified Capacity Spectrum method, Pushover Analysis.*

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ABBREVIATIONS

ABBREVIATION	DESRIPTION
AASTHO	American Association of State Highway and Transportation Officials
ADRS	Acceleration Displacement Response Spectrum
ASCE	American Society of Civil Engineers
ATC	Applied Technology Council
CALTRANS	California Department of Transportation
CEN	European Committee for Standardization
CSM	Capacity Spectrum Method
CHBDC	Canadian Highway Bridge Design Code
DCM	Displacement Coefficient Method
DBD	Displacement Based Design
DBE	Design Basis Earthquake
DDBD	Direct Displacement Based Design
DL	Dead Load
EQ	Earthquake Load
EQGM	Earthquake Ground Motion
FBD	Force Based Design
FEMA	Federal Emergency Management Agency
eSDOF	Equivalent Single Degree of Freedom
IRC	Indian Road Congress
IS	Indian Standard
MADRS	Modified Acceleration Displacement Response Spectrum
MCE	Maximum Considered Earthquake
MCSM	Modified Capacity Spectrum Method
MDOF	Multiple Degree of Freedom
NEHRP	National Earthquake Hazards Reduction Program
NSP	Non-Linear Static Procedure
PACT	Performance Assessment Calculation Tool
PBD	Performance Based Design
PBEE	Performance Based Earthquake Engineering

ABBREVIATIONS

ABBREVIATION	DESRIPTION
PBSA	Performance Based Seismic Assessment
PBSD	Performance Based Seismic Design
PBSE	Performance Based Seismic Engineering
POA	Pushover Analysis
PSA	Pseudo-Spectral acceleration
RC	Reinforce Concrete
SD	Spectral Displacement
SDOF	Single Degree of Freedom
SEAOC	Structural Engineers Association of California
SIDL	Superimposed Dead Load
SSI	Soil Structure Interaction

NOTATIONS

NOTATION	DESRIPTION
α	Horizontal seismic coefficient
α	Post yield stiffness ratio
α_{pi}	Initial performance maximum acceleration
α_y	Yield acceleration
β	Coefficient depending on soil foundation
β_{eff}	Equivalent viscous damping ratio
β_0	Initial damping value
λ	Coefficient depending on importance of bridge
δ	Displacement
ε_{ap}	Plastic strain amplitude
ε_b	Buckling strain in the longitudinal reinforcing steel
ε_{cu}	Ultimate compression strain of the confined core concrete
ε_{su}	Strain at the maximum stress of the transverse reinforcement
ε_y	Yield strain
μ	Ductility ratio
θ_p	Plastic hinge rotation
θ_u	Ultimate (total) drift
θ_y	Elastic drift at yield
ρ_s	Volumetric ratio of transverse steel
ρ_t	Volumetric ratio of the longitudinal reinforcement
Φ_p	Plastic curvature
Φ_u	Ultimate (total) curvature
Φ_y	Nominal yield curvature
Δ_y	Yield displacement
$a_{SDOF,j}$	Acceleration of the equivalent SDOF system at the i^{th} pushover step
d_{bl}	Diameter of the longitudinal reinforcement
d_{pi}	Initial performance maximum displacement
$d_{SDOF,j}$	Displacement of the equivalent SDOF system at the i^{th} pushover step

NOTATIONS

NOTATION	DESRIPTION
d_y	Yield displacement
f'_{cc}	Confined concrete strength
f'_{ce}	Ultimate expected compressive strength of concrete
f_u	
f_{ye}	Expected yield strength of longitudinal reinforcement
f_{yh}	Yield stress of the transverse hoops
f_y	Yield stress
k_{50}	Stiffness at which 50% of the ultimate soil resistance is mobilized
m_j	Lumped weight at i^{th} node
p_{ult}	Ultimate soil resistance
$u_{i,j}$	i^{th} node displacement at the j^{th} pushover step
t_{ult}	Ultimate skin resistance of pile
y_{50}	Displacement at which 50% of the ultimate soil resistance is mobilized in horizontal direction
z_{50}	Displacement at which 50% of the ultimate soil resistance is mobilized in vertical direction
A_{cc}	Area of confined concrete core
A_g	Gross section area
A_h	Horizontal seismic coefficient in Interim provision
A_{st}	Area of steel
A_v	Area of transverse shear steel
B	Width of the section
D	Overall depth of section.
E_c	Elastic modulus of concrete
E_s	Elastic modulus of steel
F_{eq}	Seismic force to be resisted
I	Importance Factor
H	Height of the structure
K	Strength enhancement factor
L	Length of the structure

NOTATIONS

NOTATION	DESRIPTION
L_c	Length from the critical section to the point of contraflexure in the member
L_p	Plastic hinge length
L_{sp}	Strain penetration length
M	Modification factor
M_p	Plastic strength capacity
P	Axial load on member
R	Response Reduction Factor
Sa/g	Average response acceleration coefficient
Sa	Spectral Acceleration
Sd	Spectral Displacement
T	Fundamental natural period
T_0	Initial elastic time period
T_{eff}	Effective time period
T_{sec}	Secant period
$V_{b,j}$	Base shear at the j^{th} pushover step
W	Seismic Weight
Z	Zone Factor

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1.0 INTRODUCTION:

1.1 General

Bridges are lifeline facilities that must remain functional even after major earthquake shaking. Its performance during and even after an earthquake event is quite crucial for socio-economic considerations. A bridge must remain functional even after the seismic event is over to provide relief as well as for security and defence purpose.

Bridge foundation is not easily accessible for inspection and retrofitting after an earthquake, and any inelastic action or failure of the superstructure renders the bridge dysfunctional for a long period. Connection failure is generally brittle in nature and hence avoided. Therefore, the substructure is the only component where inelasticity can be allowed to dissipate the input seismic energy and that too in flexural action. In addition, a flexural damaged pier can be more easily retrofitted. However, much of the substructure damage in past earthquake has occurred at pier columns. Failures of the bridge piers in the past major earthquakes attracted attention towards prevalent design practices for bridges.

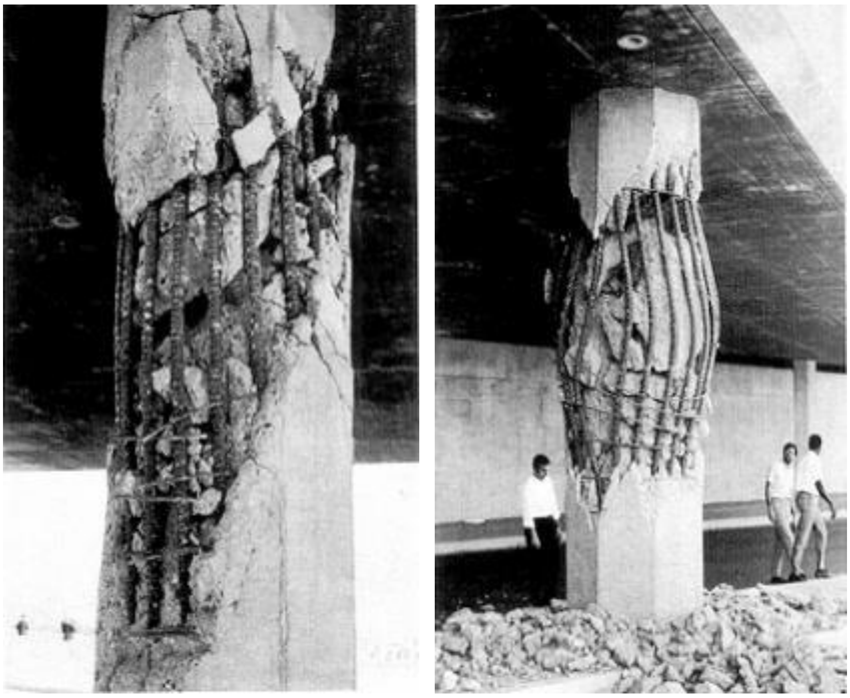


Figure 1.1 : Bridge Pier Failure

Seismic provisions in the design codes have undergone many changes over the years. In the year 1958, seismic provisions were introduced for the first time, for bridge design in IRC: 6, wherein the country was divided into 4 regions based on the damage likely to occur. Meanwhile, IS: 1893 came up with a different map with five seismic zones, which was introduced in IRC: 6 during 1981. Further changes were made in IRC:6, for computation of seismic force, horizontal seismic coefficient, importance factor and a coefficient to account for different soil and foundation system as given in IS:1893:1970[36]. With the increasing frequency of earthquakes, more attention was paid to the seismic development programs.

After the devastating Bhuj Earthquake in 2001, radicle changes were brought in the seismic design methodology in IS: 1893[37] Part 1: 2002 and the same were adopted in IRC:6 in the year 2003. For computation of seismic force, force-based approach was adopted, and mandatory provisions were included to prevent dislodgment of superstructure and ductile detailing of piers in line with IS: 13920[38] to minimize the damage, especially in seismic zones IV and V. Till the year 2011, the bridges were being designed based on working stress approach. Meanwhile there had been rapid developments in state-of-the-art in the area of seismic resistant design of bridges, like performance-based design approach, which have been incorporated in many international standards of countries like Japan, USA, New Zealand and Eurocode. Similarly, the bridge design codes in India also witnessed a major change with the introduction of Limit State Design approach for design in IRC: 6-2014[32], IRC: 112-2011[34].

Thus, with the change in seismic provisions in the same design standard, a bridge designed in the earlier days will vary considerably with the recent ones. The same pier section with same constituent materials, located at the same region will have different rebar, detailing and thus performance for seismic demand. Hence the assessment of the existing bridge piers based on performance is more useful, rather than force based only to address the problem.

In recent years, there is a new focus in seismic assessment on the performance of reinforced concrete bridges. In a performance-oriented environment, a bridge is assessed to meet specified performance levels. Many of the international design codes have adopted different approaches to achieve required performance objectives where plastic deformation in members is considered as the demand parameter.

However, current Indian seismic design standards for reinforced concrete bridges do not address adequate performance design requirements in details. Although future earthquakes may result in damage to existing reinforced concrete bridges, the quantification of the damage with respect to different hazard level is not indicated clearly. We are yet to develop robust methodologies and quantifications of engineering demand/capacity parameters for performance-based design methodology.

There are many literatures available on the seismic evaluation procedures of buildings. But there is no much effort available in literature for seismic evaluation of existing bridges although bridge is a very important structure in any country. The attention for existing bridges is comparatively less. A large number of bridges are even designed and constructed without considering seismic forces. The performance level of such existing bridges for different seismic hazard levels, quantification of damage, development of more reliable analytical procedures are areas which need further attention. Therefore, it is very important to evaluate the capacity of existing bridges against seismic force demand.

This paper conducts investigation on the seismic performance of existing bridge piers. It also outlines one of the most widely accepted method of Performance Based Seismic Assessment and discuss it in the context of traditional force-based seismic design and earlier design approaches.

1.2 Objective of Present Study

The objective of the present study is to evaluate and compare the seismic performance two typical existing reinforced concrete bridge piers (one short and one long pier column) designed as per seismic provisions in IRC:6-2000[30] and IRC:6-2014[32], using Pushover Analysis. The assessment is based on Modified Capacity Spectrum in accordance to ASCE-41[6], (2017), which can be used for the evaluation of both rehabilitation as well as design of new project.

1.3 Scope of Work

The present study has been planned and outlined as below to fulfil the objective of the work:

- Design of two existing pier columns: one short and one long, as per provisions in IRC:6-2000[30] and IRC:6-2014[32].
- Comparison of seismic response of the different substructures analysed for the mentioned technique.
- Static non-linear displacement-based pushover analysis of bridge substructure using SAP2000 (CSI 2009) software to assess displacement and lateral load capacity.
- Evaluation of spectral demands, performance points from seismic force obtained from these simplified procedures
- against the results (median values) obtained using a Pushover Analysis (POA) method.
- Evaluation of global responses e.g. displacement at pier top, shear demand, yield strain, ultimate strain.
- Behaviour analysis of these concrete bridge piers for using- Capacity spectrum approach
- Comparison of performances of the piers designed in different era against different level of seismic demands.

1.4 Thesis Outline

The thesis is organized as follows:

Chapter 1- Introduction: the background and incentive for this study is described, the objectives of the research are outlined.

Chapter 2- Literature review: Previous work and researches pertinent to this study are briefly addressed. Discussion on certain gaps identified from the review is also discussed.

Chapter 3- Performance based seismic assessment: The state of art of practice of performance-based seismic assessment is described, including the philosophy and assessment criteria and methods. Detailed discussion on static non-linear Pushover analysis theory and its applications in context of the codal provisions are also presented.

Chapter 4- Numerical study: Case studies of seismic performance of existing RCC bridge substructures, designed with the guidelines provided in the country's bridge design code, are presented.

Chapter 5- The major findings from the study conducted together with the results and discussions are presented.

Chapter 6- Conclusions and future work: A discussion of the significance of this study for performance based seismic assessment is presented. Possible scope of future research areas is identified and outlined in this chapter.

2.0 LITERATURE REVIEW:

2.1 General

Extensive review of various literature of the specified topic has been carried out to understand the present state-of-art prior to the actual work of the present study to identify the potential areas where further research study can be carried out. The study has been conducted in two parts: (i) Research papers and journals; (ii) Design standards and codes.

(i) Research papers and journals

- M J N Priestley[48] (2000), in his paper outlined the three mentioned methods of determination of seismic performance- the capacity spectrum approach, the N2 method and direct displacement-based design and compares them in the context of traditional force-based seismic design and earlier design approaches which contained some elements of performance-based design. Factors defining different performance states are also mentioned. Emphasis is placed on soil-related problems, and the incorporation of soil/structure interaction into performance-based design. It points out that the main differences from the N2 method appear to be that damage indices are not specifically referenced in the Capacity Spectrum approach, and that seismic demand is expressed in terms of response spectra set plotted with acceleration on the vertical axis, and displacement on the horizontal axis. Period is determined by radial lines from the origin. The paper discusses implications of performance based seismic design of building structures like: influence of building height, influence of seismic intensity on base shear, incorporation of foundation flexibility effects into performance based seismic design.

- Bento et. al.[11] (2004) in their paper discussed about non-linear static procedures applicable for RCC building structure. Three methodologies have been developed for the performance evaluation; The Displacement Coefficient Method – DCM (FEMA-273[18]); The Capacity Spectrum Method – CSM (ATC-40[8]); The N2 Method (adopted in EC8[26]). They tried to evaluate and compare the response of two reinforced concrete building systems: Three non-linear static procedures are used for the seismic assessment of a four-story reinforced concrete structure and the N2 method chosen as the non-linear static procedure for the seismic assessment of the eight-story building. It is observed from their study that the response of the structure is sensitive to the shape of the lateral load distribution. Moreover, the uniform load pattern seems to indicate conservative results regarding the base shear evaluation but they may be misleading in some cases. They concluded that because of some of the pushover analysis's limitations referred previously, sometimes it is necessary to use the non-linear dynamic analysis (time-history analysis) as a verification tool at this developmental stage.

- Freeman[24] (2004) in his paper reviewed the development of the capacity spectrum method which compared the capacity of the structure (in the form of a pushover curve) with the demands on the structure (in the form of response spectra). In order to account for non-linear inelastic behaviour of the structural system, effective viscous damping values were applied to linear-elastic response spectra similar to inelastic response spectra. The most important aspect of the paper is that the need of conventional iterations in CSM to find out the intersection point with demand curve and provided could be avoided using a simple graphical method.

- Lehman et. al.[15] (2004), studied about performance-based seismic design of reinforced concrete bridge columns where an experimental study was conducted to quantify performance measures and examine one aspect of detailing for reinforced concrete bridge columns. Columns with various longitudinal reinforcement ratios and aspect ratios subjected to a constant axial lateral loading were tested to characterize the response of modern bridge columns. It was observed that the sequence of damage was similar for all columns. The most notable observations, in sequence of first occurrence, were concrete cracking, longitudinal reinforcement yielding, initial spalling of the concrete cover, complete spalling of the concrete cover, spiral fracture, longitudinal reinforcement buckling, and longitudinal reinforcement fracture. Results from the experimental and analytical investigation were used to develop and evaluate the important aspects of structural damage.

- Goswami and Murty[28] (2005) in a study compared the seismic design provisions for reinforced concrete (RC) bridge piers given in Indian codes. They investigated the adequacy of strength design provisions as per Interim IRC:6-2002[31] for most commonly used solid and hollow RC piers of circular and rectangular cross-sections. The effect of parameters such as presence transverse reinforcement, column slenderness, applied axial load were studied in this regard. They modelled 12 columns with varying parameters and performed POA on each of them to obtain the comparative results. The study results revealed that piers with solid cross-sections with design transverse shear reinforcement have better post-yield behaviour in the form of enhanced deformability and displacement ductility. In the piers, with only nominal transverse reinforcement, buckling of longitudinal reinforcement occurred, resulting in sudden loss of load carrying capacity. Further they showed that slender piers exhibit a ductile behaviour. With the increase in slenderness, the shear demand reduces and the deformability increases. The investigation on effect of axial load shows that with increase in axial load level, ductility reduces while the shear demand increases. In their study they have pointed out the drawbacks in the Indian code provisions and suggested certain changes to be urgently brought into the IRC provisions to improve the structural design of the bridges.

- Qinghua et al.[50] (2008) published the paper with a purpose to provide a comprehensive basis of plastic hinge model in the application of performance-based bridge seismic design method and to promote the development of bridge seismic design codes. They employed five commonly used plastic hinge models such as Priestley-Park model, Chang-Mander model, Japanese code model, Esmaeily-Xiao model 1 and Esmaeily-Xiao model 2 to study the feasibility and main factor for seismic damage evaluation of twelve RC bridge pier specimens of low-cycle loading test by comparing the results of numerical analysis and test data. The study results show that force-displacement curves and residual deformation calculated match the experiment with adequate accuracy, but the strain of longitudinal steel is overestimated, and the strain of core concrete is underestimated. The computed ultimate curvature is lower than experiment results when shear span ratio is not less than 8. The study also recognized that under the same loading control displacement, loading path hardly affects the aforesaid damage indices.

- K.A. Korkmaz[42] (2008) reviewed the procedures of Capacity Spectrum Method and Displacement Coefficient Method in details in his paper. New and previous Capacity Spectrum Method and Displacement Coefficient Method are taken into consideration as a performance-based analysis methodology. After definition of the methodologies, an analytical application is realized for the selected structure. He selected a 4-story reinforced concrete building in a high seismicity region for study. After the methods are applied, the time history analysis was realized for the same sample structure for control. After the analyses results, he concluded that the new capacity spectrum method gives closer results to the time history results regarding with the new and previous Displacement Coefficient Method and previous Capacity spectrum method.

- Kappos et. al.[12] (2009) in their publication reported modal pushover analysis as a means for seismic assessment of bridge structures with an aim to investigate the extension of the modal pushover method to bridges. It was found that main advantage of nonlinear static (pushover) analysis was lower computational cost compared to nonlinear dynamic time-history analysis. Extension of the pushover approach to consider higher modes to match as closely as possible to the results of nonlinear time history analysis was explored. The paper applied the modal pushover analysis to a bridge and by comparing the results from standard pushover, modal pushover and nonlinear time history analysis. It was found that modal pushover was the best method to apply.

- Kevin R. Mackie[41]. (2012), in his study of performance-based seismic design and assessment of bridges emphasized on defining performance using damage, loss, and sustainability metrics, probabilistic quantification of performance. Differentiation between performance-based assessment and design is also presented in this study. Examples are presented for performance-based design using a case study, followed by performance-based assessment of an integrated bridge-ground system. The objective of this paper is to demonstrate both PBSA and PBSD of typical concrete highway bridges specifically considering potential losses as the measures of performance. PBSD is illustrated using a model purposely kept simple to facilitate analysis, followed by PBSA of a typical highway overpass using a more complex integrated bridge-foundation-ground model. The performance metrics presented are repair costs and repair times.

- Abey et al.[3] (2013) reviewed the different provisions in the present Indian codes IRC: 112-2011[34] and IRC:6-2014[32] for seismic design in his journal. The comparison of IS/IRC code with AASHTO LRFD[1] Bridge Design Specifications-2007 is also done for certain provisions. Through this paper the need for bringing modification in the present code provisions is emphasized with reference to a performance based seismic design approach. The paper highlights the importance of ductility and energy dissipating capacity of a structure in resisting seismic forces and critically evaluates the limitations of the current Indian code provisions on seismic resistant design.

- Andreas J. Kappos[40] (2015), provided a brief overview of available PBD/DBD methods for bridges, focusing on the new contributions. The methods available for bridges are critically reviewed and a number of critical issues are identified, which arise in all procedures. A PBD procedure based on elastic analysis is presented and the use of the secant stiffness approach and 'over-damped' elastic spectra, i.e. the 'direct displacement-based design approach' is extended with a view to making it applicable to a broad spectrum of bridge systems, including those affected by higher modes, and also introducing additional design criteria not previously used in this method. The study also summarizes the current trends worldwide in seismic assessment of bridges and applies the more rigorous assessment procedure, i.e. nonlinear dynamic response-history analysis, to assess the performance of bridges designed to the procedures. Moreover, comparisons are made between these performance–based designed bridges and similar ones designed to Eurocode 8.

(ii) Design standards and codes

- SEAOC[51], (1959) first published the Recommended Lateral Force Requirements, also known as the Blue Book. Thus, began SEAOC's role in the development of seismic provisions for incorporation into building code regulations, as well as its role in providing a commentary on existing building code seismic provisions to assist design engineers. The Blue Book[51] was widely viewed as a preliminary version of future seismic code provisions and a commentary on the UBC. The basic formula for base shear, V_w was developed in it. The Blue Book[51] commentary also clarified that recommendations were limited to the provision of life safety-minimum standards to assure public safety. Some of the Blue Book recommendations that were adopted into the UBC seismic design provisions from 1970 through 1994 are presented below.

UBC Edition	SEAOC Recommended Enhancement
1973	Direct positive anchorage of masonry and concrete walls to diaphragms
1976	Seismic Zone 4, with increased base shear requirements
	Base shear dependence on site conditions through coefficient S
	Occupancy Importance Factor I for certain buildings
	Interconnection of individual column foundations
1985	Diaphragm continuity ties
1988	Requirements for columns supporting discontinuous walls
	Separation of buildings to avoid pounding
	Design of steel columns for maximum axial forces
	Restrictions for irregular structures
	Ductile detailing of perimeter frames
1991	Revisions to site coefficients
	Revision to spectral shape
	Increased wall anchorage forces for flexible diaphragm buildings
	Limitations on b/t ratios for braced frames
1994	Ductile detailing of piles
1997	Near-fault zones and corresponding base shear requirements
	Redundancy requirements, Increase in wall anchorage requirements

UBC Edition	SEAOC Recommended Enhancement
	Requirements to consider liquefaction
	More realistic evaluation of design drift

- ATC 40[8], (1996) was published with a primary purpose to provide an analysis and design methodology for use in seismic evaluation and retrofit of existing concrete buildings in California. Although the focus is specifically on concrete buildings, the document provides information on emerging techniques applicable to most building types. The conceptual basis of the procedures is performance-based design using nonlinear static structural analysis. The ATC 40[8] document comprises a practical guide to the entire evaluation and retrofit process. The evaluation and retrofit design criteria are expressed as performance objectives, which define desired levels of seismic performance when the building is subjected to specified levels of seismic ground motion. Topics include performance objectives, seismic hazard, determination of deficiencies, retrofit strategies, quality assurance procedures, nonlinear static analysis using the capacity spectrum method, modelling recommendations, foundation effects, and response limits. Analytical methods or techniques for detailed investigations to assess seismic capacity are also presented in this report. The report provides guidance for the review of existing conditions, preliminary determination of deficiencies, formulation of retrofit strategies. Although it is not intended for the design of new buildings, the analytical procedures are applicable.

- FEMA 440[22], (2005) is the final and principal product of the ATC-55[2] Project which records in detail an effort to assess current nonlinear static procedures (NSP) for the seismic analysis and evaluation of structures. The document has three specific purposes: (1) to provide guidance directly applicable to the evaluation and design of actual structures by engineering practitioners; (2) to facilitate a basic conceptual understanding of underlying principles as well as the associated capabilities and limitations of the procedures; and (3) to provide additional detailed information used in the development of the document for future reference and use by researchers and others. The document presented a comprehensive analysis program to develop the improved versions of both the capacity spectrum method (CSM) as well as the displacement coefficient method (DCM). This investigation consisted of determining the peak nonlinear displacements of a large number of SDOF systems subjected to a large number of real ground motion records. For this purpose, the SDF systems with initial natural periods between 0.05s and 3s and having nine different levels of lateral strengths were used. They are modelled with four different hysteretic behaviours (elastic perfectly plastic, stiffness-degrading, strength- and stiffness-degrading, and nonlinear elastic behaviour). Based on the results obtained from this comprehensive parametric study, various improvements in both the CSM and DCM were proposed.

- FHWA[23], (2006) published a manual which represents the most current state-of-practice in assessing vulnerability of highway structures to the effects of earthquake. In this manual, a performance-based retrofit philosophy is introduced similar to that used for the performance-based design of new buildings and bridges. It gives a complete overview of the retrofitting process including the philosophy of performance-based retrofitting, the characterization of the seismic and geotechnical hazards, the assignment of the Seismic Retrofit Category, and summaries of recommended screening methods, evaluation tools, and retrofit strategies. Performance criteria are defined for four performance levels and are given for two earthquake ground motions with different return periods, 100 and 1000 years. Criteria are recommended according to bridge importance and anticipated service life, with more rigorous performance being required for important, relatively new bridges, and a lesser level for standard bridges nearing the end of their useful life. Minimum recommendations are made for screening, evaluation and retrofitting according to an assigned Seismic Retrofit Category. It concludes that the bridges in Category A need not be retrofitted whereas those in Categories B, C and D require successively more rigorous consideration and retrofitting as required. Various retrofit strategies are described, and a range of related retrofit measures are explained in detail, including restrainers, seat extensions, column jackets, footing overlays, and soil remediation.

- ASCE-41[6], (2017) represent a new state of the practice in seismic evaluation and retrofit of existing buildings. The new standard combines seismic evaluation and retrofit into one document and brings consistency to the process. It describes deficiency-based and systematic procedures that use performance-based principles to evaluate and retrofit existing buildings to withstand the effects of earthquakes. The standard presents a three-tiered process for seismic evaluation according to a range of building performance levels by connecting targeted structural performance and the performance of non-structural components with seismic hazard levels. The deficiency-based procedures allow evaluation and retrofit efforts to focus on specific potential deficiencies deemed to be of concern for a specified set of building types and heights. This standard establishes analysis procedures and acceptance criteria and specifies requirements for foundations and geologic site hazards; components made of steel, concrete, masonry, wood, and cold-formed steel; architectural, mechanical, and electrical components and systems; and seismic isolation and energy dissipation systems. Checklists are provided for a variety of building types and seismicity levels in support of the Tier 1 screening process. It introduces revisions to the basic performance objectives for existing buildings and to the evaluation of force-controlled actions. It revises the nonlinear dynamic procedure and changes provisions for steel and concrete columns, as well provisions as for unreinforced masonry.

2.2 Critical Observations

The referred literatures were highly informative and gave an insight of the existing assessment procedures of a structure. The critical observations that arose from the literatures referred are as below:

The present Indian code provisions are based on Force-based method of design. The load-deformation pattern of various components clearly shows that, damage is equally or more dependent on deformation than the force. Even though a general performance objective of Life Safety level in minor and moderate seismic events and Collapse prevention in major seismic events are mentioned, no provisions are given in Indian Standard codes for assessing it. The codes do not elaborate on the relation between the response reduction factor and the ductility of a member, and the method of achieving a higher ductility. They are also silent about deformations, the relation between deformation and seismic force and the demand and capacity of deformation of various bridge members.

PBEE and PBSA, as applied to buildings, have seen rapid development and adoption recently (e.g., ATC-58[9] and ATC-63[10]). However, in the bridge and infrastructure arena, there have been fewer attempts (e.g., Mackie et al. 2008) at rigorous development of the data necessary for PBEE or packaging the tools in a form that allows rapid PBEE-based evaluation and assessment such as PACT in ATC-58[9]. However, assessment of an inventory of bridges in a network is unreliable using generic class fragilities due the variability in structural configurations, site conditions, and hazard.

Nonlinear static analysis has no rigorous theoretical foundation. It is based on the premise that response of an MDOF structure can be obtained via an equivalent SDOF system. This implies that the estimated responses are controlled by a single mode whose shape remains constant throughout the inelastic excursion. Clearly, this assumption is wrong, but in general lead to rather good predictions of the maximum responses of many MDOF systems. Nonlinear analysis under strong ground motion and consideration of coupled soil-foundation-structure effects may not be warranted for the design of individual highway overpass bridges.

Most analytical studies on RC bridge piers, including those with large cross sections, still idealise the member by its centroidal axis and define the inelastic action of the whole cross-section in a lumped sense. This does not accurately model the spread of inelasticity both along the member length and across the cross-section. Hence, a distributed plasticity model is required where the pier is discretised into a number of segments along the length, and each segment into a number of fibres across the cross-section.

3.0 PERFORMANCE BASED SEISMIC ASSESSMENT:

3.1 General

Earthquake is a highly unpredictable phenomenon and so are its consequences. With the passage of each earthquake, new experiences lead to new field of research and upgradation of the existing seismic provisions. Thus, a structure designed as per earlier codal provisions may prove to be inadequate within few years, with upgraded studies. In the context of force-based design, it has also been realized that the increase in strength may not enhance safety, nor reduce damage. The objective of most codes is to provide life safety performance during large and infrequent earthquakes. However, recent earthquakes have shown that structures may suffer irreparable or too costly to repair damages. Besides, inelastic behaviour, indicating damage, is observed even during smaller earthquakes in a few cases. Performance Based Design concept considering multi-level design objectives, seems to provide a better framework to incorporate the current codal stipulations ensuring that the structures perform appropriately for all earthquakes.

The performance of a structural system can be evaluated by non-linear static analysis. This involves the estimation of the structural strength and deformation demands and the comparison with the available capacities at desired performance levels. The basic idea of Performance Based Assessment is to evaluate structures that perform desirably during various loading scenarios. Furthermore, this notion permits the owners and designers to select personalized performance goals for the design of different structures. However, there is a need to emphasise that some minimum level or minimum acceptable criteria are required to be fulfilled by all structures.

3.2 Force Based Seismic Design

Current force-based design is considerably improved compared to the procedures used earlier. The basic sequence of operation required in Force-based seismic design is presented in **Figure 3.1.**

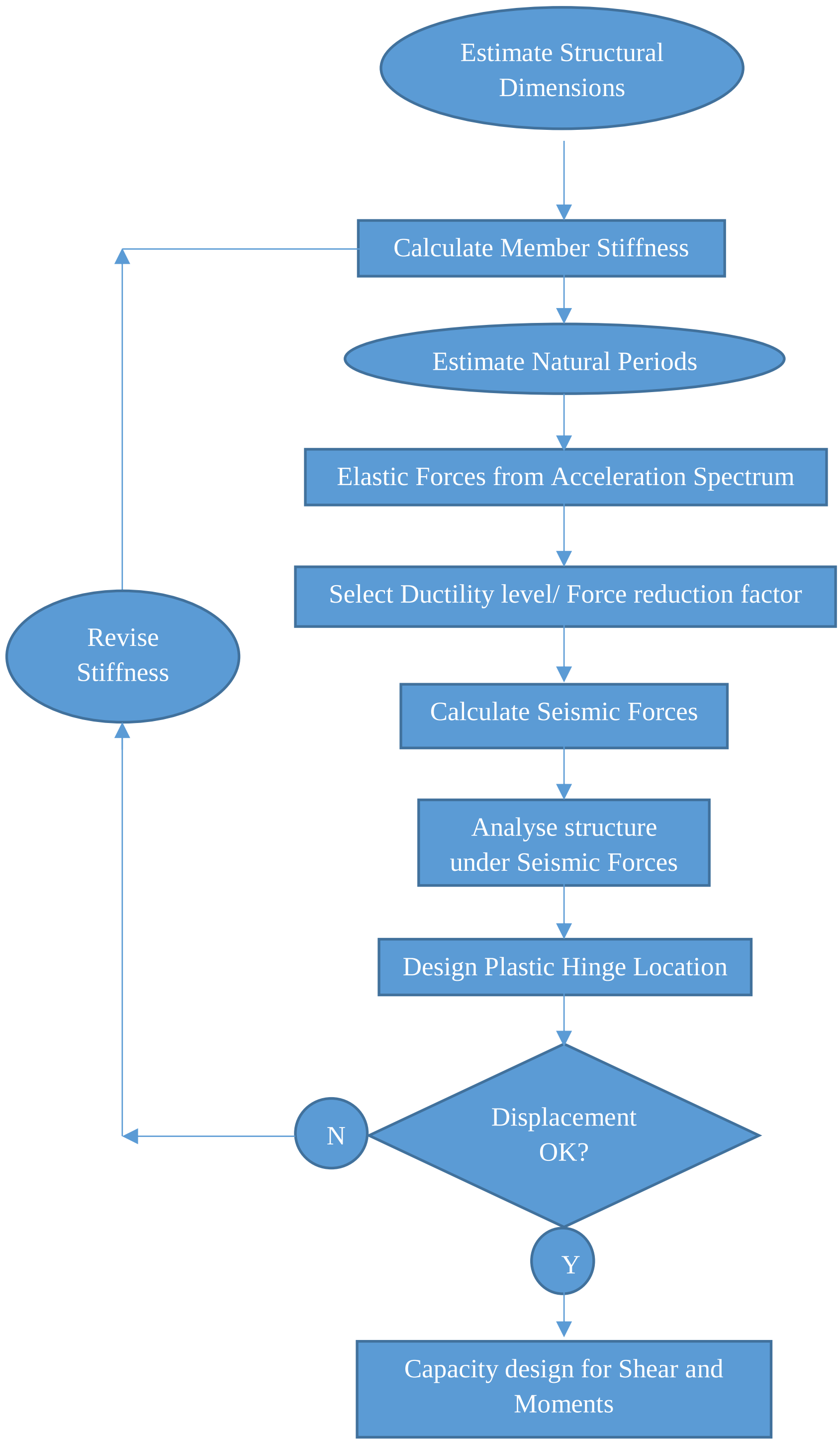


Figure 3.1: Sequence of Operations for Force-Based Design

(Courtesy: Displacement-based Seismic Design of Structures, Priestly et al.)

3.2.1 Limitations of Force Based Seismic Design

The current force based seismic design when combined with capacity design principles and detailing produces safe and satisfactory design. However, the degree of protection provided against damage under a given intensity of earthquake is non-uniform.

Force based design relies on estimates of initial stiffness to determine the period and the distribution of design forces between different structural elements. Since stiffness is partially dependent on strength of elements, this cannot be known until the design process is complete.

Distributing seismic force among the elements based on assumed initial stiffness does not seem to be very logical for many structures, because it incorrectly assumes that the different elements can be forced to yield simultaneously.

FBD is based on the assumption that unique force reduction factors are appropriate for a given structural type and material. In an MDOF system, it may possible that different elements may reach various levels of ductility under the design seismic intensity. Thus, the assumption is very general and requires to be updated.

3.3 Performance Based Seismic Engineering

3.3.1 Evolution of Performance Based Seismic Design (PBSD)

In the existing seismic design codes and practices, defined objective of providing structural safety were not postulated in general. For a very long time the need for designing and constructing civil engineering facilities for predictable performance under all the different types of excitations to which they are likely to be exposed during their lifetime has been discussed.

In the 1960s, it was recognized that structural failure generally occurs in successively more severe stages at successively less probable excitation. Thus, proposals were made that design should be done ideally following a comprehensive procedure by which the resistances of a structure to the various failure stages (performance levels) are correlated to the probabilities of the corresponding loads (excitations or hazard levels). The 1967 commentary of the Structural Engineers Association of California (SEAOC) Blue

Book[51] introduced what can be considered the general philosophy of the earthquake resistant design of buildings sheltering other than essential and hazardous facilities.

Series of earthquakes that caused huge economy loss worldwide, triggered the engineers and public policy makers alike determined that it is unacceptable to experience this magnitude of loss in the relatively frequent and moderate events. The M7.1 Loma Prieta EQ in October 1989 caused more than $8 billion in direct damage. This economic loss was judged by the structural engineering profession and public policy makers as too large for this moderate event. In January 1994, the M6.7 Northridge EQ occurred, resulting in losses estimated at more than $20 billion. Faced with this problem and with the need to repair, rehabilitate (upgrade), and reconstruct many hundreds of buildings, the California Office of Emergency Services contracted with SEAOC to develop Recommendations for Performance-Based Design and Construction Procedures which could be used immediately. In one year the SEAOC Vision 2000 Committee developed recommendations for Performance-Based Seismic Engineering (PBSE). The Building Seismic Safety Council's (BSSC) National Earthquake Hazards Reduction Program (NEHRP) in their publications [FEMA 273[18] and 274[19] Reports, 1997] also have set design objectives, prescribed design criteria introduced analytical techniques for performance evaluation.

These projects aimed to provide seismic design procedures that are documented in guidelines and commentary that can form the basis of a new generation of seismic codes and enable designers and owners to expect a new level of predictability in performance for future buildings. In so doing, a new balance between design objectives and design methods can be achieved that will result in overall economies over the lifetime of our building stock while providing building owners and the public with much greater assurance when the earthquake strikes. This in turn enabled lenders and insurers to predict earthquake losses with much more accuracy, and government response and recovery agencies to plan for known earthquake consequences.

3.3.2 Seismic Design to Seismic Assessment

Traditionally, seismic design codes provide prescriptive "deemed to comply" criteria that specify minimum levels of strength, stiffness and ductility, and outline the acceptable materials, detailing and configuration in order to achieve the minimum levels of safety and performance of structures in earthquake.

The expected performance of a structure in earthquake is not generally assessed in new design but a minimum level of performance is implied by the rules that are prescribed. These rules include the use of capacity design and minimum requirements for element detailing. The minimum levels of performance required extend beyond the design levels of shaking even though they may not be explicitly accounted for.

In our country, following the experience in Bhuj Earthquake in 2000 the seismic codes were revised, incorporating many new provisions. The bridges designed and constructed prior to the introduction of modern seismic codes, are expected to be non-conforming to the new provisions. It has been identified that seismic assessment and retrofit of the older bridges is a significant challenge for the earthquake engineering fraternity.

Internationally, the Applied Technology Council (ATC) has been publishing guidelines for the seismic evaluation of existing buildings since the 1980s, from the ATC-14[7] (1987) and ATC-22 (1988) documents, eventuating in the widely used FEMA-178[17] (1992) and FEMA-310[20] (1998). These early guidelines rely significant on the observations and lessons from earthquakes due to limited analysis tools and experimental data. However, in the bridge and infrastructure arena, there have been fewer attempts at rigorous development of the data necessary for packaging the tools in a form that allows rapid PBEE-based evaluation and assessment such as PACT in ATC-58[9]. Therefore, the development of enabling technologies that allow PBSA is required.

3.3.3 Need for Performance Based Seismic Assessment (PBSA) of Bridges

Existing bridges are expected to be non-conforming to the prescriptive rules in the seismic codes and the earlier generation of seismic assessment guidelines. By applying the modern seismic standards to these structures will only yield a large number of ‘non-compliance’ bridges without a necessary understanding of their actual seismic risk or performance. Similarly, code writers struggle in providing a prescriptive set of rules that could capture all the possible variation of non-compliant existing bridges.

One key principle is that the understanding of how the structural components behave and respond under earthquake shaking becomes more important than necessarily meeting specific code clauses. Similarly, a thorough understanding of the underlying principles of various code clauses is required such that ensure that contravening a rule does not unduly jeopardize the minimum levels of resilience that are inherently provided for in a new bridge.

Performance-based seismic assessment offers a consistent framework for engineers to evaluate how a structure may behave in an earthquake, in particular when it does not meet a number of conventional prescriptive requirements and responds in a highly non-linear manner with potentially mixed-ductility response. The modern seismic guidelines provide the tools to assess the likely behaviour of the structure and the performance consequences for a range of earthquake ground shaking.

3.3.4 Performance-Based Seismic Assessment (PBSA) Guidelines

Recognizing the needs as outlined in preceding section, a number of ‘performance-based’ seismic assessment guidelines were introduced in the late 1990s and early 2000s. In FEMA-273[18] (1997) pioneered performance-based seismic assessment of existing buildings using nonlinear analysis procedures such as nonlinear static pushover and nonlinear dynamic analysis. FEMA-273[18] and its subsequent revisions (FEMA-356[21], 2000 and ASCE-41[6] (2006) compiled and introduced definition of various performance levels and associated structural and non-structural ‘performance’, ways to use non-linear analysis techniques and definitions of “acceptance criteria” i.e. plastic capacity deformation and strength limits for various structural components.

In all these procedures, a “pushover” analysis is employed to predict the inelastic force-deformation behaviour of the structure, the different methods then differing in the technique used to calculate the inelastic displacement demand for a given ground motion. These nonlinear static methodologies were initially proposed and verified for seismic assessment and retrofitting of buildings, rather than bridges. The application of these concepts to bridges has been more limited. Studies on the so-called ‘direct’ displacement-based design (DDBD) of bridge piers or even entire appeared in the mid-1990s. The knowledge gap also prompted a series of very recent research endeavours.

3.4 Performance of Bridge Pier

Engineering community paid inadequate attention on the seismic design and performance of bridges until the collapse of several highway bridges in the 1971 San Fernando, California, USA, earthquake causing significant economic losses. Extensive research investigations have been conducted on the seismic behaviour of reinforced concrete bridges afterwards. Also, past earthquakes revealed several deficiencies in the design and detailing of bridges. Significant improvements in both design practice and analytical methods have been achieved. Some of these new developments have already been incorporated in the design codes.

For reinforced concrete bridges supported by columns, key aspects of structural performance include crushing of core concrete, spalling of cover concrete, buckling of rebar and low cycle fatigue of longitudinal rebar. Development of performance-based seismic assessment provisions for reinforced concrete bridges requires engineering approaches that consider these aspects to define the state of structural performance.

Figure 3.2: Performance Level, Damage State and Engineering Limit State

Performance-based seismic design/assessment of bridges requires that the engineer complete the following tasks: select performance objective(s), define performance level using engineering limit state, define site hazard level at site, perform structural design and evaluation using engineering approaches and quality assurance.

3.4.1 Performance Objective

A performance objective is the pairing of a performance level and a seismic hazard level. Discrete performance objectives are defined for each seismic hazard level. The structural capacity for each performance level is related to a specific state of damage or required repair and is quantified using one or more engineering limit states.

3.4.2 Performance Level

Performance Level is defined as the expected behaviour of the structure in the design earthquake in terms of limiting levels of damage to the structural and non-structural components. In major seismic design codes expected performances of bridges in future earthquake events have been specified.

The performance-based seismic design framework recommended in ATC-32 adopts three performance levels. The three performance levels outlined in **Table 3.1** are designated as Fully Operational Performance Level, Delayed Operational Performance Level, and Stability Performance Level. Each performance level is defined by the expected bridge serviceability, required repair effort, and future performance. This framework can be easily condensed to a two-level framework.

Table 3.1: Performance Levels (Courtesy: ATC-32)

Performance Level	Required Repair Effort Serviceability	Future Performance
Fully Operational	Minimal Damage Fully Serviceable	Original Level
Delayed Operational	Repairable Damage Delayed Serviceable	Slightly Reduced Relative to Original
Stability	Unrepairable Damage Unserviceable	Minimal Level (Aftershock)

To date most codes and documents advocating performance-based design of bridges have adopted a two-level design framework. The Canadian Highway Bridge Design Code[13] (CHBDC) incorporates the following performance levels for design of a new bridge structure as shown in **Table 3.2.**

Table 3.2 Performance Levels as per CHBDC (2014)

Service	Post- earthquake serviceability	Damage
		No cracks
Immediate	Bridges shall be fully serviceable for normal traffic and repair work does not cause any service disruption.	Concrete compressive strain – 0.004 Reinforcing steel tensile strain – below yield strain
Limited	Bridge shall be usable for emergency traffic and be repairable without requiring bridge closure. At least 50% of the lanes, but not less than one lane, shall remain operational. If damaged, normal services shall be restored within a month.	Reinforcing steel strain – 0.015. No residual settlement or rotation. Members shall be capable of supporting the dead plus full live loads. The structure shall retain 90% of seismic capacity for aftershocks and shall have full capacity restored by the repairs.
Service disruption	The bridge shall be usable for restricted emergency traffic after inspection. The bridge shall be repairable. Repairs to restore the full service require bridge closure.	Extensive concrete spalling, but the confined core concrete shall not crush. Reinforcing steel – 0.05 Members shall be capable of supporting the dead plus 50% live loads, excluding impact, including p-delta effects without collapse. Permanent residual settlement or rotation. The structure shall retain 80% of seismic capacity for aftershocks and shall have full capacity restored by the repairs.
Life Safety	The bridge shall not collapse, and it shall be possible to evacuate the bridge safely.	Members shall be capable of supporting the dead plus 30% live loads, excluding impact, including p-delta effects without collapse. Permanent offset shall be limited such that the bridge can be evacuated safely.

In the year 2006, FHWA[23] published the Seismic Retrofitting Manual for Highway Structures which presents a performance-based approach to the seismic retrofitting of highway bridges which are in line with recent development of performance-based seismic assessment. **Table 3.3** provides the qualitative and quantitative performance levels as described in FHWA[23] manual (2006).

Table 3.3 Performance Levels Description (FHWA[23], 2004)

Performance Level	Operational Performance Level	Post-Earthquake Serviceability	Qualitative Damage Description	Repairability
PL0		No minimum level of performance is recommended.		Closed
PL1	Life Safety	Significant damage is sustained during an earthquake and service is significantly disrupted, but life safety is assured.	Permanent offsets and cracking, yielded reinforcement, and major spalling of concrete, which may require closure to repair. Partial or complete replacement of columns may be required. Beams may be unseated from bearings but no span should collapse. Similarly, foundations are not damaged except in the event of large lateral flows due to liquefaction, in which case inelastic deformation in piles may be evident.	The bridge may need to be replaced after a large earthquake.
PL2	Operational	Damage sustained is minimal and full service for emergency vehicles should be available after inspection and clearance of debris.	Minor inelastic response and narrow flexural cracking in concrete. Permanent deformations are not apparent, and repairs can be made under non-emergency conditions with the possible exception of superstructure expansion joints which may need removal and temporary replacement.	Bridge should be reparable with or without restrictions on traffic flow.
PL3	Fully Operational	Damage sustained is negligible and full service is available for all vehicles after inspection and clearance of debris.	Evidence of movement, and/or minor damage to non-structural components, but no evidence of inelastic response in structural members or permanent deformations of any kind.	Any damage is repairable without interruption to traffic.

For a bridge meeting the Fully Operational Performance Level, repair is not required and the bridge is expected to be fully serviceable immediately following the earthquake. The future seismic performance will be essentially unaffected which requires negligible damage accumulation.

A bridge meeting the Operational Performance Level requirements is expected to have sustained some damage during the earthquake. The bridge should provide limited service to emergency vehicles. Closure of the bridge should be limited to several days provided sufficient resources are available. In future, more significant events, the bridge performance is expected to be close to the original performance.

A bridge meeting the Life Safety performance level is expected to have sustained significant damage. As a result, partial or full replacement of bridge elements (including columns and restrainers) may be required and the bridge may remain out of service for several weeks or months. The future performance of the structure is limited; however, in its damaged state, the bridge is expected to survive an aftershock of lesser intensity.

3.4.3 Seismic Hazards

A seismic hazard is the probability that an earthquake will occur in a given geographic area, within a given window of time, and with ground motion intensity exceeding a given threshold. The seismic hazard at a given site is represented as a set of earthquake ground motions and associated hazards with specified probabilities of occurrence. Ground motions are usually represented as time histories, acceleration response spectra, displacement response spectra, drift demand spectra. Four levels of probabilistic events are proposed in Blue Book[51] of SEAOC Vision 2000 report and are presented in **Table 3.4** below.

Table 3.4 Seismic Hazard Levels (Courtesy: SEAOC Vision 2000, 1996)

Event	Recurrence Interval	Probability of exceedance
Frequent	43 years	50% in 30 years
Occasional	72 years	50% in 50 years
Rare	475 years	10% in 50 years
Very rare	970 years	10% in 100 years

The Indian Standard codes mention two seismic hazard levels: DBE and MCE. The bridges are designed for Design Basic Earthquake (DBE) with partial load and material safety factors. For converting the spectral acceleration value obtained from the design spectrum (which is nothing but response spectra for 1g PGA) given in IRC:6-2014[32], which is for Maximum Considered Earthquake (MCE), we use the factor Z/2 in the equation. Where Z is known as the zone factor which is based on the classification of areas according to the seismic intensity felt by the structures. **Table 3.5** shows the Seismic Hazard Levels as per Indian Standards.

Table 3.5 Seismic Hazard Levels as per Indian Standards

Event	Probability of exceedance
Design Basis Earthquake (DBE)	10% in 50 years
Maximum Considered Earthquake (MCE)	2% in 50 years

3.4.4 Failures of Bridge Piers

In order to better understand the relationship between structural response levels and performance levels, it is instructive to consider the relationship between member and structure limit states. The different modes of failures are discussed in the following paragraphs.

3.4.4.1 Cover Concrete Spalling

In columns with poorly detailed transverse reinforcement, one of the most critical consequences of flexural yielding is the spalling of cover concrete. The effectiveness of the transverse steel will be greatly reduced when the concrete cover in the vicinity of the plastic hinge spalls. Steel in the region of spalling will then be partly exposed, which will greatly reduce anchorage. To some extent, the reduction of efficiency of lap splices in transverse reinforcement depends on the degree of spalling. Such spalling is followed by a rapid degradation in the effectiveness of the transverse steel that can lead to column failure. Different studies have indicated that a concrete compression strain of ε_{cu} of 0.004 ~ 0.005 is a conservative lower limit to initiation of spalling.

3.4.4.2 Compression Failure of Core concrete

Confined core concrete of column has increased compression strength, and more importantly, increased compression strain capacity than the unconfined cover concrete. The enhancement of compression stress- strain characteristics of the core concrete is a result of the action of well-detailed transverse reinforcement in the form of hoops or spirals. In conjunction with longitudinal reinforcement, close-spaced transverse reinforcement acts to restrain the lateral expansion of the concrete that accompanies the onset of crushing, maintaining the integrity of the core concrete. The useful limit to confined concrete compression strain is usually taken to occur when fracture of the transverse reinforcement confining the core occurs.

$$\varepsilon cu = 0.005 + \frac{1.4\ \rho_s\ f_{yh}\ \varepsilon_{su}}{f_{cc}}$$

3.4.4.3 Longitudinal Bar Buckling

Buckling may occur after longitudinal reinforcement is first subjected to inelastic tension strain under one direction of seismic loading. When the loading direction is reversed, the bars initially transfer all the compression force on the section and must yield in compression before previously formed cracks close. It is during this stage of response that the bars are susceptible to buckling. Once the cracks close, the compression stiffness of the concrete can be expected to restrain the tendency for bar buckling. It will thus be seen that bar buckling is more dependent on the inelastic tensile strain developed in a previous yield excursion, than on pure compression characteristics. The hoops or spirals act to restrain the longitudinal reinforcement from buckling when in compression. To ensure inelastic buckling does not occur, the maximum spacing of transverse hoops or spirals must be related to the bar diameter. If spacing of transverse reinforcement, in potential plastic hinge zones exceeds six longitudinal bar diameters (i.e., $s > 6d_b$), then local buckling at high compressive strains in the longitudinal reinforcement is likely. If $6db < s < 30d_b$, the limiting buckling strain is taken as twice the yield strain of the longitudinal steel i.e. $\varepsilon_b \sim 2*f_y/E_s$.

3.4.4.4 Low Cycle Fatigue of the Longitudinal Rebar

Fracture of longitudinal reinforcing steel due to low-cycle fatigue is one of the prominent failure modes for bridge piers with or without low levels of axial load. The longitudinal reinforcements may be expected to undergo large tension and compression strain reversals with large cyclic-strain amplitudes up to ε_{ap} 0.05~0.06 in medium to high seismic risk zones. Under compression the bars tend to buckle. This buckling, however, can extend beyond single hoop spacing. Buckling leads to weakening (embrittlement) of the material which in turn translates into reduced fatigue life. Hence, low-cycle fatigue failure of longitudinal reinforcement is a critical failure mode that deserves more attention. This type of behaviour can cause a rapid loss in strength that will effectively limit the ductility of a steel member.

3.4.4.5 Fracture of the Longitudinal Rebar

Tensile fracture occurs when the tensile strain reaches a critical level, as given by ε_{smax} of 0.10. This failure mode is only likely under near-field impulse-type ground motions where there is essentially a monotonic (pushover) response.

3.4.4.6 Strength Degradation

Strength degradation is complex in most of the structures. The inability to carry imposed loads, such as axial forces in a column is often related to a specified strength drop of 20% (Priestly et. al.) from the maximum attained (or sometimes from the design) strength. However, some residual strength is maintained for further increase of displacement. The occurrence of negative incremental stiffness of the moment-curvature characteristic, which is associated with strength drop, is cause for concern under dynamic response, since it implies redistribution of strain energy from elastically responding portions of the structure into the member with negative stiffness. This might be due to three effects. First, there could be cyclic strength degradation associated with low-cycle fatigue damage of various components in the lateral-force-resisting system. Interspersed might be in-cycle strength losses due to component damage as deformations increase monotonically. Superimposed on this is the negative slope associated with P-Δ effects, which may or may not be significant. Unfortunately, it is not possible to distinguish between cyclic and in-cycle strength losses solely from information normally available from a nonlinear static analysis. The P-Δ effects are always present and contribute to real negative post-elastic stiffness. This has potentially explosive consequences and hence considered as a damage limit state.

3.4.4.7 Shear Failure

If the shear capacity of a member is reached before the moment capacity, there is a potential for a sudden nonductile failure of the member, leading to collapse which is not desired. Members that cannot develop the flexural capacity in shear should be checked for adequacy against calculated shear demands. For pier columns, the shear capacity is affected by the axial loads and is based on the most critical combination of axial load and shear**.** A column that can develop the shear capacity to develop the flexural strength over the clear height has some ductility to prevent sudden catastrophic failure of the vertical support system. It is tentatively suggested that the shear capacity be based on a reduced ductility demand, corresponding to 60% of the computed displacement ductility, while the full ductility demand be used to compute the dynamic amplification.

3.5 Nonlinear Static Procedures (NSPs)

With the introduction of performance-based seismic assessment guidelines (e.g., FEMA-440[22] and ATC-40[8] respectively) and design standards (e.g., Eurocode-8[26], CEN 2005), approximate nonlinear static procedures (NSPs) have become popular in engineering practice as efficient tools to estimate seismic demands under a given performance limit state. Nonlinear Static Procedures (NSPs) represent relatively simplified approaches for the evaluation of the seismic response of existing structures, complementing well the more elaborate, or at least more time consuming, nonlinear dynamic analysis procedures.

A nonlinear analysis is an analysis where a nonlinear relation holds between applied forces and displacements. In the Nonlinear Static Procedures, the seismic loading is idealized as an equivalent static force pattern applied to a linear elastic structural model. Nonlinear effects can originate from geometrical nonlinearity, material nonlinearity. These effects result in a stiffness matrix which is not constant during the load application.

The most essential component of all existing nonlinear static procedures (NSPs) is the monotonic pushover analysis procedure. Compared to the linear static analysis procedures, the primary advantage of the NSPs is their ability to account for the redistribution of internal forces as the structural components experience nonlinearity under incremental lateral forces. The employment of the non-linear static procedure involves four distinct phases as described in **Figure 3.3.**

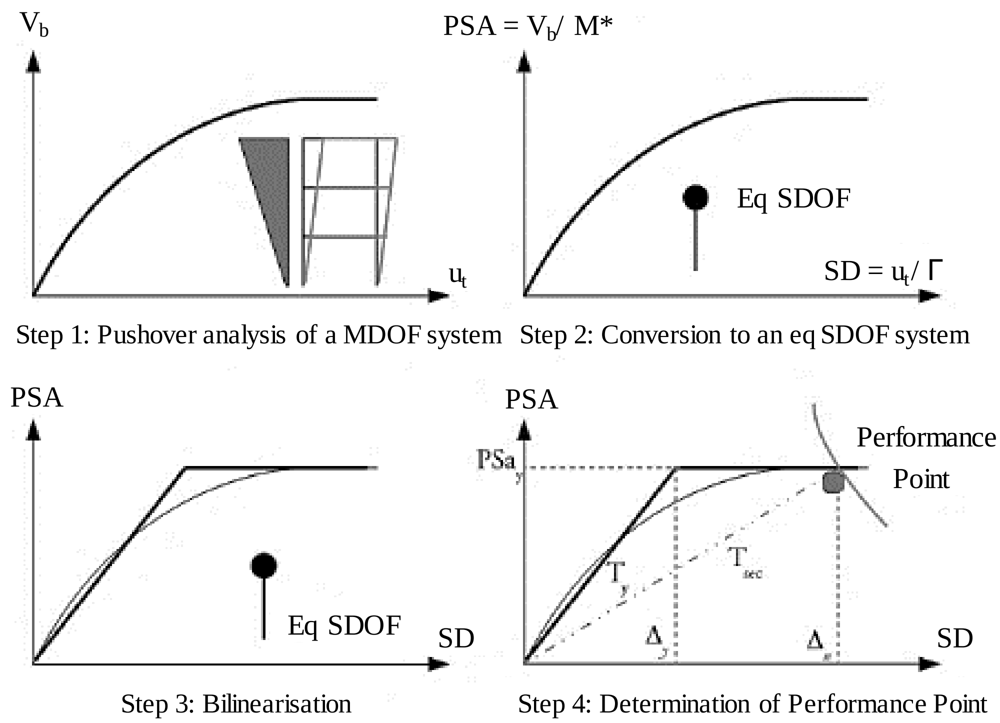


Figure 3.3: Pictorial Representation of Non-linear Static Procedures

3.5.1 Types of Nonlinearity

3.5.1.1 Geometric Nonlinearity

In analyses involving geometric nonlinearity, changes in geometry as the structure deforms are considered in formulating the constitutive and equilibrium equations. This is the type of nonlinearity where the structure is still elastic, but the effects of large deflections cause geometry of the structure to change; so that linear elastic theory is no longer valid.

3.5.1.2 Material Nonlinearity

Material nonlinearity involves the nonlinear behaviour of a material based on a current deformation, deformation history, rate of deformation, temperature, pressure, and so on.

3.6 Modified Capacity Spectrum Method

Of the various NSPs available, in our study we have assessed our bridge substructure vide widely accepted Modified Capacity Spectrum method. A brief description of the analytical evaluation of the simplified Performance Based Seismic Assessment procedure, and the terms associated with it are presented in the following paragraphs.

3.6.1 Response Spectra

Response spectra are curves plotted between maximum response of SDOF system subjected to specified earthquake ground motion and its time period (or frequency). Response spectrum can be interpreted as the locus of maximum response of a SDOF system for given damping ratio. Response spectra thus helps in obtaining the peak structural responses under linear range, which can be used for obtaining lateral forces developed in structure due to earthquake thus facilitates in earthquake-resistant design of structures.

Usually response of a SDOF system is determined by time domain or frequency domain analysis, and for a given time period of system, maximum response is picked. This process is continued for all range of possible time periods of SDOF system. Final plot with system time period on x-axis and response quantity on y-axis is the required response spectra pertaining to specified damping ratio and input ground motion. Same process is carried out with different damping ratios to obtain overall response spectra.

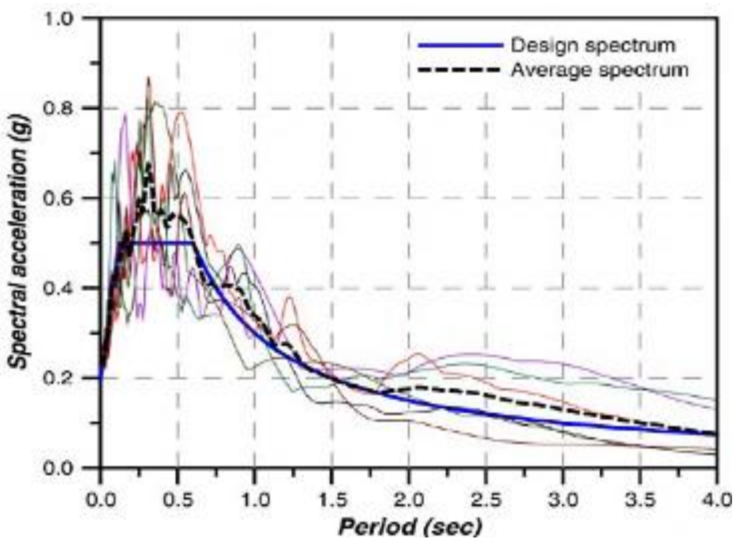


Figure 3.4: Typical Response Spectrum Plot

3.6.2 The Capacity Spectrum Method (CSM) in ATC 40

The main NSP prescribed in the ATC 40 (1996) report entitled "Seismic Evaluation and Retrofit of Concrete Buildings", is based on the equivalent linearization approach (The Capacity Spectrum Method, CSM), which was originally developed as a rapid evaluation method for a pilot seismic risk project of the Puget Sound Naval Shipyard for the U.S. Navy in the early 1970s.

In this method, combines a representation of structural capacity with a representation of seismic demand to predict an expected displacement for a structure subjected to seismic events. Structural capacity is represented by a capacity spectrum determined from a push-over analysis and seismic demand is represented by the response spectra. The seismic hazard is represented by a 5%-damped acceleration response spectrum, which is reduced (using the expressions prescribed in the ATC 40 report) based on the effective damping ratio of an equivalent linear system. The seismic demand experienced by a structure is represented by an acceleration-displacement response spectrum (ADRS). The vertical axis is pseudo-spectral acceleration (PSA) and the horizontal axis is spectral displacement (SD). Linear response spectra with varying amounts of damping represent inelastic seismic demand. Each value of damping is associated with a corresponding value of ductility from some effective damping equation. For different displacement values along the capacity spectrum, bilinear approximations are constructed which define a yield displacement for the structure. The location at which the demand and capacity ductility are equal denotes a type of dynamic equilibrium. The equilibrium point defines the expected performance of the structure, referred to as the "*Performance Point*" which is an estimate of the actual maximum displacement expected during an anticipated earthquake. The overall process is illustrated in **Figure 3.5**.

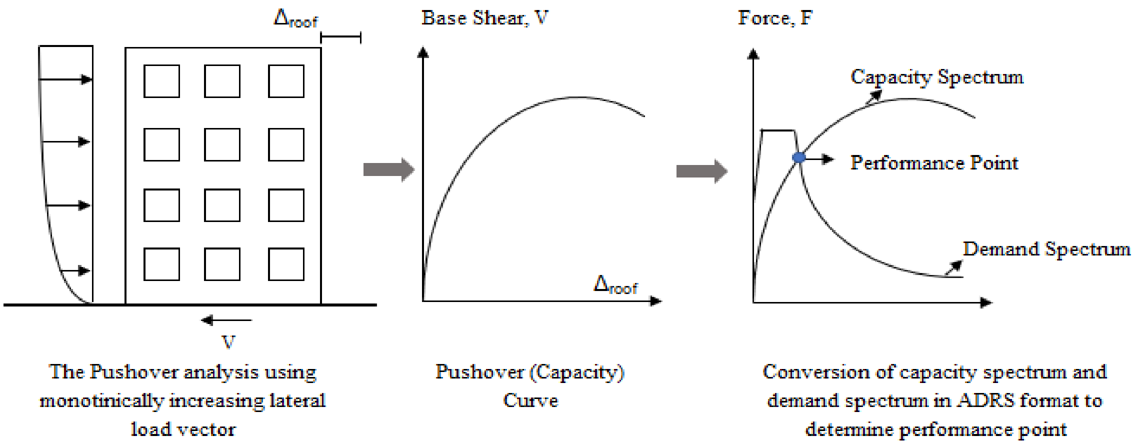


Figure 3.5: The overall process of the Capacity Spectrum Method (CSM)

3.6.3 Modified Capacity Spectrum Method (MCSM) by FEMA 440

Various studies have identified the inherent sources of uncertainties in the use of heavily damped (or reduced) demand spectra and the use of (effective) natural period corresponding to intersection of the capacity and demand spectra. It was observed that the effective period at performance point may not be a true representative of the actual condition of structure and may have little to do with the realistic nonlinear dynamic response of the system (particularly for high levels of target displacement). Few studies are done of the displacement demands as well as convergence issues for procedures mentioned in the ATC 40[8] report to determine the intersection point. As part of attempts to overcome such limitations, several other researchers continued to work on this. The FEMA 440[22] (2005) report presented a comprehensive analysis program to develop the improved versions of the capacity spectrum method (CSM).

The conventional CSM was modified for the use of the optimal effective linear period equations with the application of the modification factor to the acceleration-displacement response spectrum (ADRS) creating the modified acceleration-displacement response spectrum (MADRS). A more efficient bilinear approximation of the pushover curve was proposed. The expressions to estimate the effective time period and effective viscous damping (for reducing the elastic spectrum to an inelastic demand spectrum) were also improved. Through the implementation of the modification factor, the displacement of the effective linear system, defined by the effective period, T_{eff}, and damping, ς_{eff}, is translated down to the secant period, T_{sec}. By introducing a modification factor, the "*Performance Point*" remains at the intersection of the demand and capacity curves while the use of an effective period different from the secant period is now permitted. Typical Modified Acceleration-Displacement Response Spectrum (MADRS is presented in **Figure 3.6.**

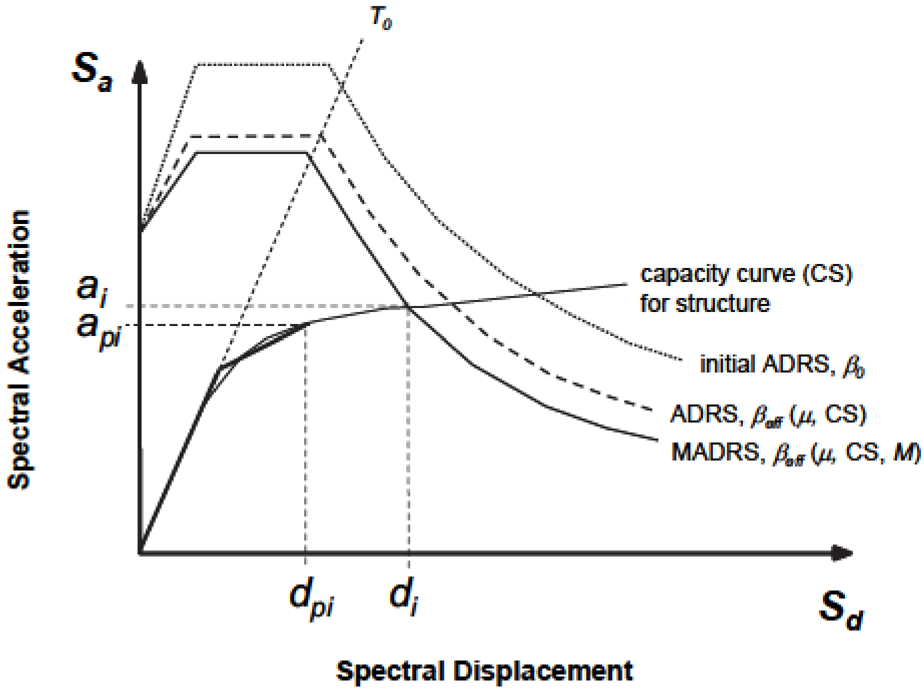


Figure 3.6: Modified Acceleration-Displacement Response Spectrum (MADRS).

3.7 Pushover Analysis

Pushover Analysis came into practice in the late 90's, which was explicitly used for analyses of building. However, its potential and applicability to the analyses of bridge structures has been recognized lately. Our present study tries to implement this procedure to assess the bridge structure.

3.7.1 Definition of Pushover Analysis

The pushover analysis of a structure is a static non-linear analysis where a structure is subjected to gravity loading and a monotonic displacement/force-controlled lateral load pattern which continuously increases through elastic and inelastic behaviour until an ultimate condition is reached. Lateral load may represent the range of base shear induced by earthquake loading, and its configuration may be proportional to the distribution of mass along the structure height, mode shapes, or another practical means.

Output generates a static-pushover curve which plots a strength-based parameter against deflection. Generally, a plot of the total base shear versus top displacement in a structure is obtained by this analysis.

The analysis is carried out till failure; thus, it enables determination of collapse load and ductility capacity. Results provide insight into the ductile capacity of the structural system, and indicate the mechanism, load level, and deflection at which failure occurs. The decision to retrofit can be taken in such studies.

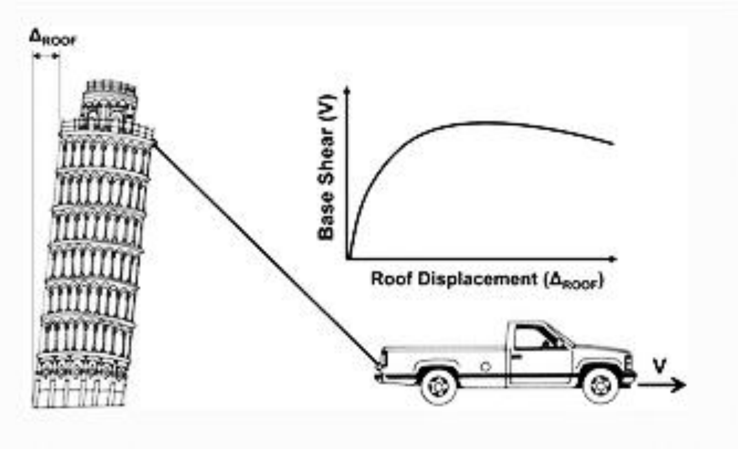


Figure 3.7: Basic Concept of Pushover analysis

3.7.2 Need for Pushover Analysis

The purpose of pushover analysis is to evaluate the expected performance of structural systems by estimating performance of a structural system by estimating its strength and deformation demands in design earthquakes by means of static inelastic analysis and comparing these demands to available capacities at the performance levels of interest. The evaluation is based on an assessment of important performance parameters, including global drift, inter-story drift, inelastic element deformations, deformations between elements, and element connection forces (for elements and connections that cannot sustain inelastic deformations). The inelastic static pushover analysis can be viewed as a method for predicting seismic force and deformation demands, which accounts in an approximate manner for the redistribution of internal forces that no longer can be resisted within the elastic range of structural behaviour. The pushover is expected to provide information on many response characteristics that cannot be obtained from an elastic static or dynamic analysis.

3.7.3 Methodology

While analysing structures, material nonlinearity is assigned to discrete hinge locations where plastic rotation occurs according to FEMA-356[21] or another set of code-based or user-defined criteria. Strength drop, displacement control, and all other nonlinear software features, including link assignment, P-Delta effect, and staged construction, are taken in considerations during static-pushover analysis.

The basic steps for a typical POA are described below:

- **Step 1:** Define the mathematical model with the non-linear force deformation relationships for the various components/elements;
- **Step 2:** Define properties and acceptance criteria for the pushover hinges and locate them on the model;
- **Step 3:** Define the pushover load cases;
- **Step 4**: Run the basic static analysis and, if desired, dynamic analysis. Then run the static nonlinear pushover analysis;
- **Step 5:** Display the pushover curve;

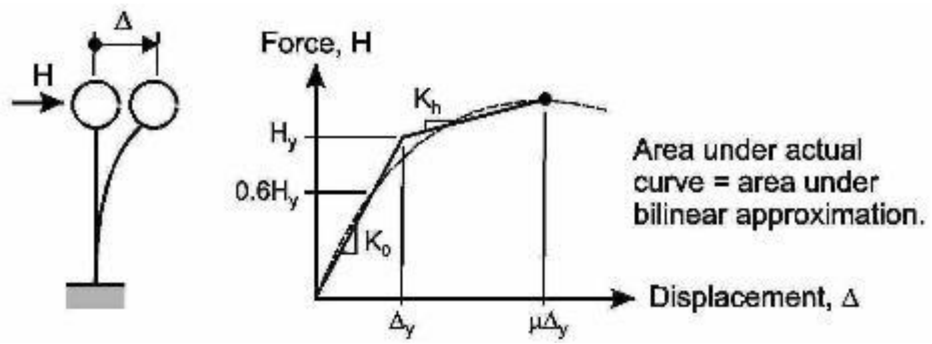


Figure 3.8: Typical SDOF model and Pushover curve

3.7.4 Modelling and Analysis considerations

The pushover analysis can be viewed as a methodology for predicting seismic force and deformation demands, which can account for, in an approximate manner, the redistribution of internal forces occurring when the structure is subjected to inertia forces that no longer can be resisted in the elastic range.

The evaluation is based on assessment of important performance parameters.

- The fundamental question in the execution of the pushover analysis is the magnitude of the target displacement at which seismic performance evaluation of the structure is to be performed. The target displacement serves as an estimate of the global displacement of the structure is expected to experience in a design earthquake. It is the roof displacement at the centre of mass of the structure. In the pushover analysis it is assumed that the target displacement for the MDOF structure can be estimated as the displacement demand for the corresponding equivalent SDOF system transformed to the SDOF domain through the use of a shape factor. Under the Non-linear Static Procedure, a model directly incorporating inelastic material response is displaced to a target displacement, and resulting internal deformations and forces are determined. The mathematical model of the structure is subjected to monotonically increasing lateral forces or displacements until either a target displacement is exceeded, or the structure collapses. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake.
- The techniques in this methodology incorporate the use of an effective nonlinear model to represent the global structure. In some cases, initial stiffness is used, as is the case, for instance, the secant stiffness is used by the CSM, to represent the initial condition of the structure. Care should be taken on the selection of this stiffness as some parameters, further in the procedures, are based on the initial stiffness of the structure;
- Selection of appropriate load distribution is crucial to predict accurately higher mode effects in the post-elastic range, mainly if they play an important role in the structural response. The modal adaptive pattern is thought to provide better results as they account for the inelastic response by suitably adjusting the load pattern based on the mode shape in the previous step. Nevertheless, other load patterns proposed can also be considered with caution;

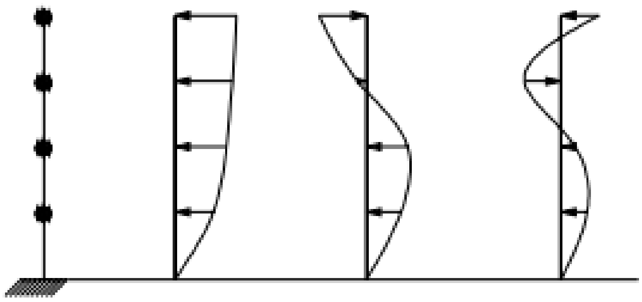


Figure 3.9: Load Patterns due to Higher Modes

- Incorporation of torsional effects, due to mass, stiffness and strength irregularities, is difficult to account for, without a 3-dimensional analysis. However, this has a great inconvenience in the definition of a lateral load pattern in both the direction. An alternative could be in performing independent analysis in both directions;
- Some researchers prefer to use site-specific spectra.

3.7.5 Limitations of Pushover Analysis

Although POA has advantages over elastic analysis procedure, some of its major drawbacks are:

- ➢ It is not able to represent accurately dynamic phenomena, without the use of more sophisticated lateral load patterns
- ➢ It is not possible to account for phenomena like stiffness and strength degradation, P-Δ effects and the duration of the seismic action
- ➢ It may not detect some important deformation modes that may occur in a structure subjected to severe earthquakes, and it may exaggerate others
- ➢ Inelastic dynamic response may differ significantly from predictions based on invariant or adaptive static load patterns, particularly if higher mode effects become important

Since the pushover analysis is approximate in nature and is based on static loading, as such it cannot represent dynamic phenomena with a large degree of accuracy. It may not detect some important deformation modes that occur in a structure subjected to severe earthquakes, and it may significantly from predictions based on invariant or adaptive static load patterns, particularly if higher mode effects become important.

Because of some of these pushover analysis's limitations, sometimes it is necessary to use the non-linear dynamic analysis (time-history analysis) as a verification tool at this developmental stage. Nevertheless, there are still some reservations to adopt this method, which are mainly related to its complexity and suitability for practical design applications.

3.8 NSP based Seismic Assessment Techniques for Bridge Piers

Two existing concrete bridge piers of different height designed in two eras (pre2000's and post2000's) are analysed in this study. Collection of data such as geographic location, material properties, geometric properties, initial design parameters, reinforcing detailing, also detailed drawings providing data regarding the pier response is collected for checking the performance of the designed piers. Then the structure is modelled using relevant analytical software [SAP2000 (CSI 2009)].
It is important that the model be able to predict the global response as well as the local response of the structure. Local performance is associated with damage level in the structure which is related to structural reparability. "Pushover analysis" is used to develop a displacement-based pushover scheme that would provide sufficient insight into the full response, i.e., till failure.

Following are the steps to perform Capacity Spectrum Method based seismic assessment for bridges as recommended by FEMA 440[22] guideline. The original procedure is further modified by an improved definition of equivalent SDOF system especially useful for bridges.

- **Step 1**: An idealized model of the given bridge pier is developed in SAP2000, with explicit consideration of material inelasticity and geometric nonlinearity.
- **Step 2**: The structure is analysed for sustained gravity loads using a load control strategy.
- **Step 3**: Pushover analysis is then performed in a direction longitudinal to the bridge axis. Pushover analysis starts only after completion of the gravity load analysis. Conventional force-based pushover algorithm is employed here. In this

pushover algorithm, the analytical model is pushed by an invariant nominal load vector using a displacement control strategy until the control node reaches its predefined target displacement or analysis fails to converge.

The purpose of nominal load vector is to define the structural nodes where loads are applied. It also characterizes the load distribution shape which remains constant throughout the analysis.

Among various load patterns as mentioned in the literature, only uniform load pattern is considered here. This is justified for such a short span integral bridge. The control node is taken as the pushed pier top for longitudinal pushover analysis, whilst node at C.G. of superstructure is taken as the control node for transverse pushover analysis.

Pushover curve, which is control node displacement vs base shear, is then plotted for each direction. Critical events, such as failure of dowel bars or formation of plastic hinge, are then identified from the pushover database for each direction. These events are marked on the pushover curve.

- **Step 4**: Each pushover curve is converted to the capacity curve of an equivalent single degree of freedom (SDOF) using the Substitute Structure method. The conversion relations are as follows.

$$d_{SDOF,j} = \frac{\sum m_i u_{i,j}^2}{\sum m_i u_{i,j}} \qquad a_{SDOF,j} = \frac{V_{b,j}}{\sum m_i u_{i,j}} d_{SDOF,j} \qquad M_{SDOF,j} = \frac{\sum m_i u_{i,j}}{d_{SDOF,j}}$$

where $d_{SDOF,j}$ is displacement of the equivalent SDOF system at the i^{th} pushover step

$a_{SDOF,j}$ is acceleration of the equivalent SDOF system at the i^{th} pushover step

m_i is the lumped weight (in kN) at i^{th} node

$u_{i,j}$ is the i^{th} node displacement at the j^{th} pushover step

$V_{b,j}$ is the base shear at the j^{th} pushover step

- **Step 5**: The capacity curves are then bi-linearized based on equal energy principle for each critical event to facilitate the estimation of dynamic properties of the equivalent SDOF system. The dynamic properties, namely, initial elastic time period T_0, post-elastic stiffness α and ductility ratio μ up to the critical event, are computed as follows.

$$T_0 = 2\pi\sqrt{\frac{d_y}{a_y}} \qquad \alpha = \frac{\frac{a_{pi}-a_y}{d_{pi}-d_y}}{\frac{a_y}{d_y}} \qquad \mu = \frac{d_{pi}}{d_y}$$

- **Step 6**: The standard seismic hazard represented by the design response spectrum for 100 years of return period at the bridge site, as prescribed by IRC: SP 114-2018 following elastic response spectrum, is also plotted in the acceleration-displacement format so that graphical comparison of capacity and demand can be made possible.

- **Step 7**: For a particular event, the effective time period T_{eff} and equivalent viscous damping ratio β_{eff} are computed by the equations as given in Section 6.2 of FEMA 440[22] guidelines. The equations are as follows:

$$T_{eff} = \begin{cases} T_0 & \mu \le 1.0 \\ [0.20(\mu-1)^2 - 0.038(\mu-1)^3 + 1.0]T_0 & 1.0 < \mu < 4.0 \\ [0.28 + 0.13(\mu-1) + 1.0]T_0 & 4.0 \le \mu \le 6.5 \\ \left[0.89\left[\sqrt{\frac{\mu-1}{1+0.05(\mu-2)}} - 1.0\right] + 1.0\right]T_0 & \mu > 6.5 \end{cases}$$

$$\beta_{eff} = \begin{cases} \beta_0 & \mu \le 1.0 \\ 4.9(\mu-1)^2 - 1.1(\mu-1)^3 + \beta_0 & 1.0 < \mu < 4.0 \\ 14.0 + 0.32(\mu-1) + \beta_0 & 4.0 \le \mu \le 6.5 \\ 19\left[\frac{0.64(\mu-1)-1}{\{0.64(\mu-1)\}^2}\right]\left(\frac{T_{eff}}{T_0}\right)^2 + \beta_0 & \mu > 6.5 \end{cases}$$

- **Step 8**: The spectral modification factor B (β_{eff}) is computed using the following relation. This factor is used to scale down the design response spectrum, as shown by ADRS (β_0), to an overdamped spectrum, as denoted by ADRS (β_{eff}).

$$B(\beta_{eff}) = \frac{4.0}{5.6 - \ln \beta_{eff}} \qquad S_a(\beta_{eff}) = \frac{S_a(\beta_0)}{B(\beta_{eff})}$$

- **Step 9**: Since this effective linearization procedure is based on effective time period, unlike the conventional capacity spectrum method, which is based on secant time period, further modification factor M is multiplied to acceleration component only. This results Modified Acceleration Displacement Response Spectrum MADRS (b_{eff}, M) as shown in the **Figure 3.10.**

$$M = \left(\frac{T_{eff}}{T_{sec}}\right)^2 = \frac{1+\alpha(\mu-1)}{\mu}\left(\frac{T_{eff}}{T_0}\right)^2$$

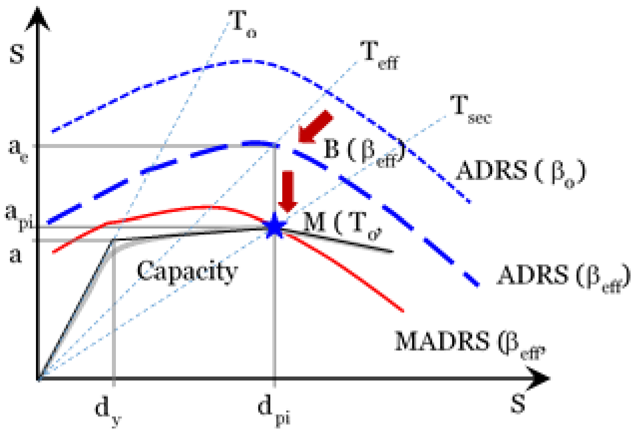


Figure 3.10: Target Displacement Demand (FEMA 440)

- **Step 10**: The point at which the capacity curve intersects the reduced response curve represents the performance point, at which capacity and demand are equal. **Figure 3.11** depicts a pictorial representation of this procedure.

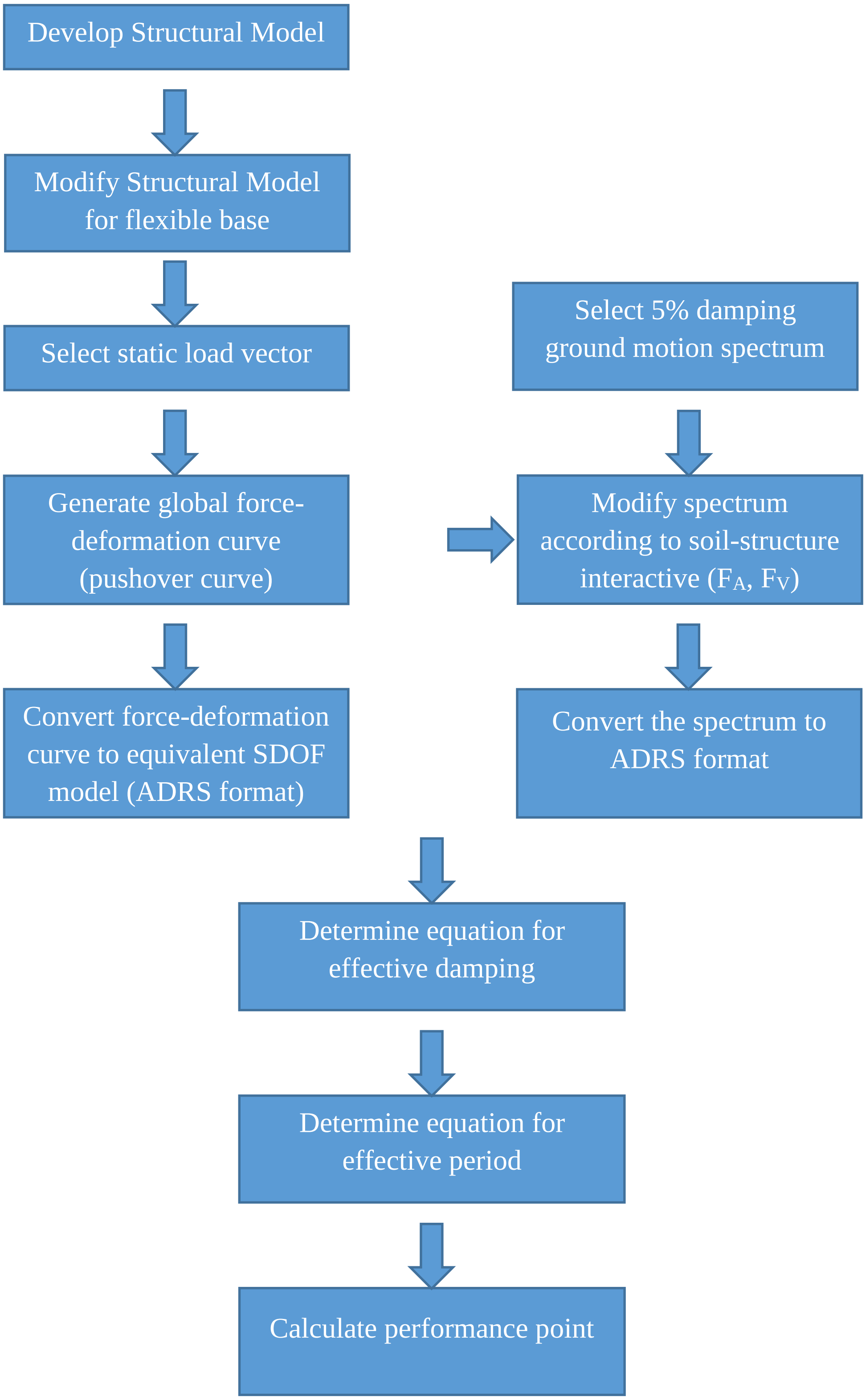


Figure 3.11: Pictorial Representation of the NSP based Seismic Assessment Technique

4.0 NUMERICAL STUDY:

4.1 General

The present study is intended to assess the seismic performance of two existing bridge piers: one short and one long pier, designed with existing Force based design approach as stipulated in IRC:06-2014[32], that is redesigned with the provisions in IRC:06-2000[30] and to compare the structural efficiency of these four piers according to most widely accepted methods of Performance Based Seismic Assessment.

Later, a nonlinear static procedure (NSP) based analysis is carried out according to ASCE-41[6].

4.2 Problem Definition

The seismic assessment is primarily based on the available drawings and design. The viaduct is 9.51km long, with simply supported PSC box type superstructure supported on RCC circular pier founded on piles. Of the whole viaduct two typical piers: shortest and the longest, are chosen for our study.

Basic steps followed in this study may be summarised are as under:

1. At first both the RCC substructures are designed for DL+SIDL+EQ using limit state design methodology and the seismic provisions are considered as per IRC:06-2014[32], considering all the load combinations.
2. Later both the same structures are redesigned using Working Stress method and the seismic provisions as per IRC:06-2000[30]. For simplicity of the problem, live load is ignored.
3. The available substructure sections as obtained from force-based design (pre/post 2000) are modelled in SAP2000 and pushover analyses are performed.
4. In the final stage Modified Capacity Spectrum Method is performed to assess their performance.

Typical pier details are shown in **Figure 4.1** to **Figure 4.3.**

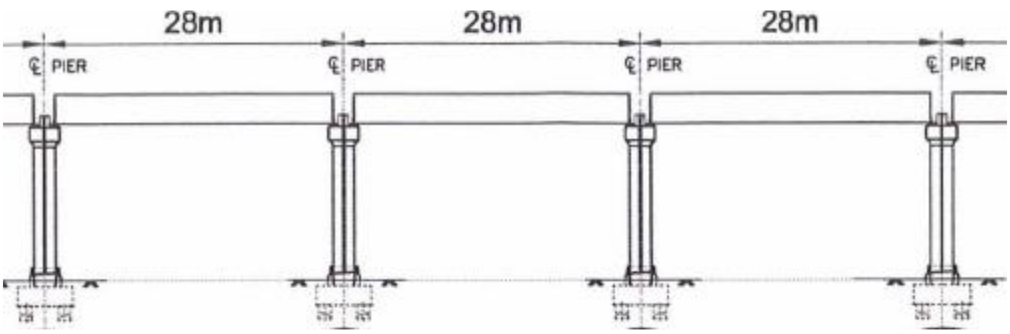


Figure 4.1: General Pier Arrangement

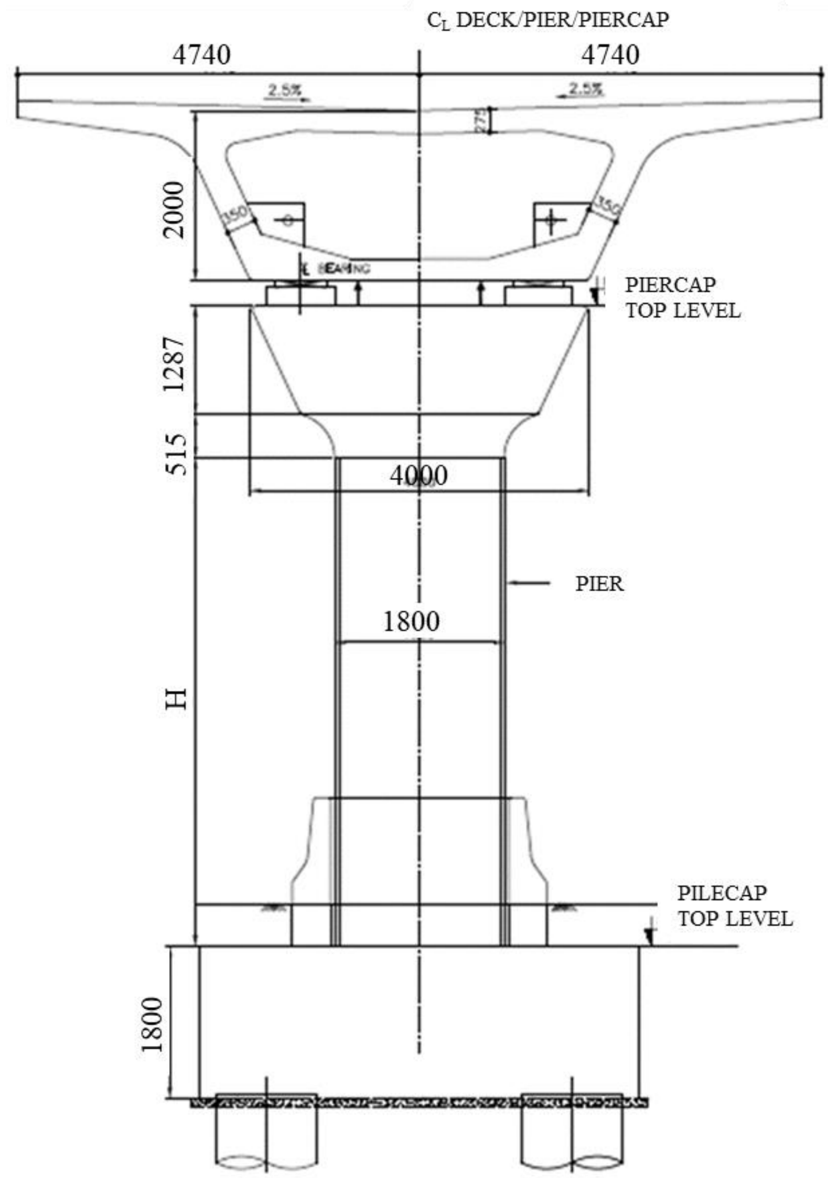


Figure 4.2: Elevation of Pier

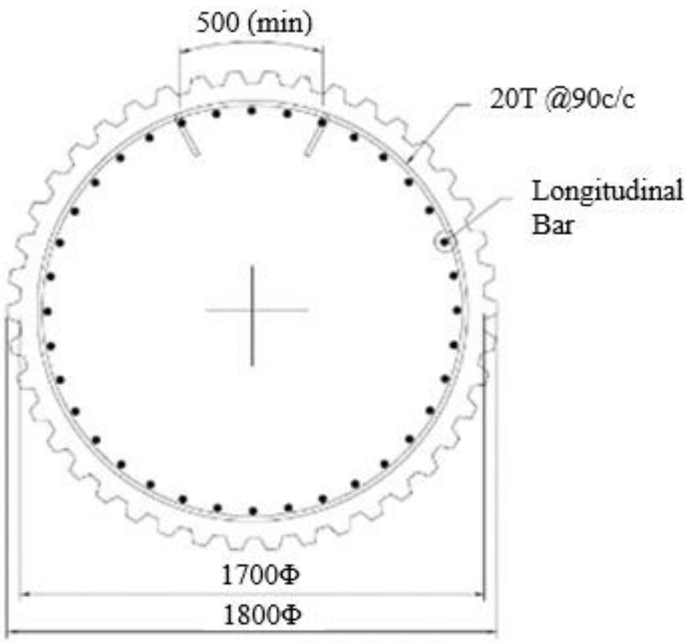


Figure 4.3: Typical Pier Section Details

The bridge data is summarized as below:

Inventory	Description
Location	Joka- Mominpur Metro viaduct.
Year Designed	2012.
Design Life	100 years.
Lengths/ Span	Total viaduct length 9.51 km.
Superstructure Width	10.50 m wide superstructure.
Superstructure Span	28 m superstructure span on either side.
Superstructure Type	Precast PSC Box superstructure simply supported on pier.
Lane/ Traffic	Double track of Broad-Gauge rail.
Pier	RCC circular pier.
Foundation	Pile.
Bearing	Elastomeric type.

The piers are designed using in-house excel spreadsheets following provisions laid down in the IRC standards and with the available design data.

4.3 Design data

The substructures studied use the following basic geometric and material properties:

4.3.1 Geometric Properties:

DESCRIPTION	SHORT PIER	LONG PIER
Pier Column Diameter	1.800 m	1.800 m
Height of pier	8.128 m	15.128 m
Overall left/right span	28.000 m	28.000 m
Effective left/right span	26.300 m	26.300 m
Width of the deck	10.500 m	10.500 m
Radius of curvature	0.000 m	0.000 m
Pier cap size (average)	2.800 m x 4.000 m x 1.325 m	2.800 m x 4.000 m x 1.325 m
Pile cap size	5.200 m x 5.200 m x 1.800 m	
Pile arrangement	4 nos. of 1.200 m dia. piles	5 nos. of 1.200 m dia. piles
Cover to pier / pier cap	50 mm	50 mm
Cover to pile / pile cap	75 mm	75 mm

4.3.2 Material Properties:

Grade of substructure concrete	M 40
Grade of foundation concrete	M 35
Unit weight of concrete	25 kN/m^3
Grade of reinforcement steel	Fe 500
Unit weight	78.5 kN/m^3

4.3.3 Loading Data:

Superstructure Dead Loads	= 2 x 150 x 28.000 / 2= 4200 kN
Superstructure SIDL	= 2 x 75 x 28.000 / 2= 2100 kN

Weight of bearing	= 4 x 0.575 x 0.600 x 0.100 x 78.5= 10.8 kN
Weight of pedestal	= 4 x 0.800 x 0.800 x 0.200 x 25.0= 12.8 kN
Weight of pier cap	= 1 x 2.800 x 4.000 x 1.325 x 25.0= 371 kN
Weight of seismic restrainers	= 1 x 1.200 x 1.100 x 1.100 x 25.0= 36.3 kN
Weight of short pier column	= 1 x 3.140 x 0.850^2 x 6.800 x 25.0= 386 kN
Weight of long pier column	= 1 x 3.140 x 1.050^2 x 13.800 x 25.0= 1195.2 kN
Vehicular live load	= 0.00 kN (For simplicity unloaded condition has been considered in our study.)

4.3.4 Seismic Data:

Seismic zone	= IV
Zone factor (100 years)	= 0.24
Response reduction factor	= 4.00
Importance factor	= 1.50
Type of soil	= III

4.3.5 Seismic Design Provisions:

In IRC:6-2000[30], the horizontal design earthquake load on bridges is calculated based on a seismic coefficient. The equivalent static horizontal seismic load on the bridge is specified as:

$$F_{eq} = \alpha\beta\lambda W,$$

where α is horizontal seismic coefficient, β is soil-foundation system factor, λ is importance factor (1.5 for important bridges, and 1.0 for regular bridges), and W is the seismic weight of the bridge. The seismic weight, acting at the vertical centre of mass of the structure, includes the dead load plus fraction of the superimposed load depending on the imposed load intensity;

In IRC:6-2014[32], the design horizontal seismic force F_{eq} of a bridge is dependent on its flexibility, and is given as

$$F_{eq} = A_h W,$$

where the design horizontal seismic coefficient A_h is given by:

$$A_h = \frac{\left(\frac{Z}{2}\right)\left(\frac{S_a}{g}\right)}{\left(\frac{R}{I}\right)}.$$

where ‘Z’ is the zone factor, ‘I’ is the importance factor, ‘R’ is response reduction factor, and Sa g is the average response acceleration coefficient for 5% damping depending upon the fundamental natural period ‘T’ of the bridge. The ‘Sa/g' value depends on the type of soil and the natural period ‘T’ of the structure.

4.4 Force-based Design Results

After performing the Force Based Seismic Design procedures as per IRC:06-2014[32], the comparative design results of the aforesaid piers are shown in **Table 4.1** as below:

Table 4.1: Comparative Design Results

Parameters	**Short Pier**		**Long Pier**	
Code: IRC: 06	Pre 2000	Post 2000	Pre 2000	Post 2000
Nomenclature	1A_pre2000	1A_post2000	9A_pre2000	9A_post2000
Concrete Grade	M40	M40	M40	M40
Pier Height (m)	8.128	8.128	15.128	15.128
Seismic Design Coefficient	0.09	0.11	0.09	0.05
Design base shear (kN)	619	1160	632	498
Design moment at base shear (kNm)	5029	9429	9562	7527
Design axial load (kN)	7163	9670	7608	10271
Design method	Working stress	Limit State	Working stress	Limit State
Ductile detailing	No	Yes	No	Yes
Long Rebar	16nos. 32ϕ	28nos. 32ϕ	78nos. 32ϕ	30nos. 32ϕ
Percentage Steel	0.51%	0.88%	2.47%	0.95%
Confining Rebar	8ϕ @300mm c/c	20ϕ @90mm c/c	8ϕ @300mm c/c	20ϕ @90mm c/c

The pier sections were kept constant and reinforcement was varied as per strength requirement. It should be noted that, in the case of reinforced concrete sections, the effective stiffness $(EI)_{eff}$ depends not only on the cross-sectional dimensions, but also on the reinforcement provided.

From the results above it is evident that for the same location, carrying same gravity loads, due to changes in provisions laid down in design codes, structures are designed differently. Area of reinforcement, detailing, materials used, even section dimensions may vary for the same structure designed in various era.

4.5 SAP Model Inputs

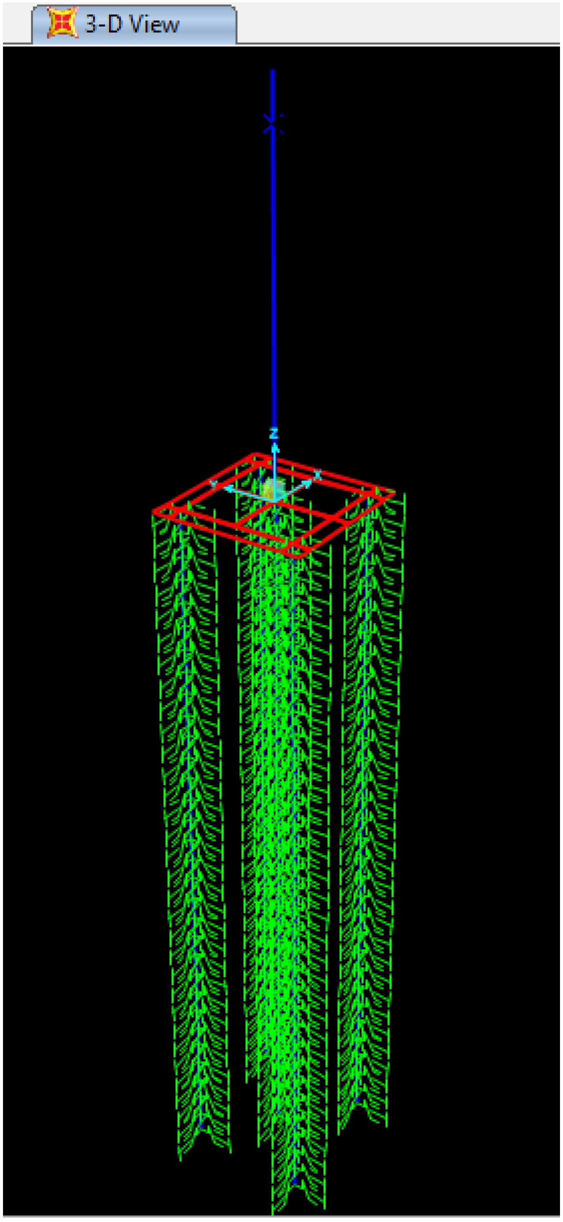
Analytical model of the four designed viaduct piers: 1A_pre2000, 1A_post2000, 9A_pre2000 and 9A_post2000 are developed in SAP2000 (Version 20.1.0). Since the viaduct comprises of simply supported superstructures on elastomeric bearings, a single pier system is modelled which represents the viaduct substructure as whole.

The tributary mass and vertical reactions from adjacent spans are lumped on the top of piers to simulate the effect of adjacent spans. Seismic mass is contributed by the dead weight. The contribution of live load to the seismic mass in is ignored.

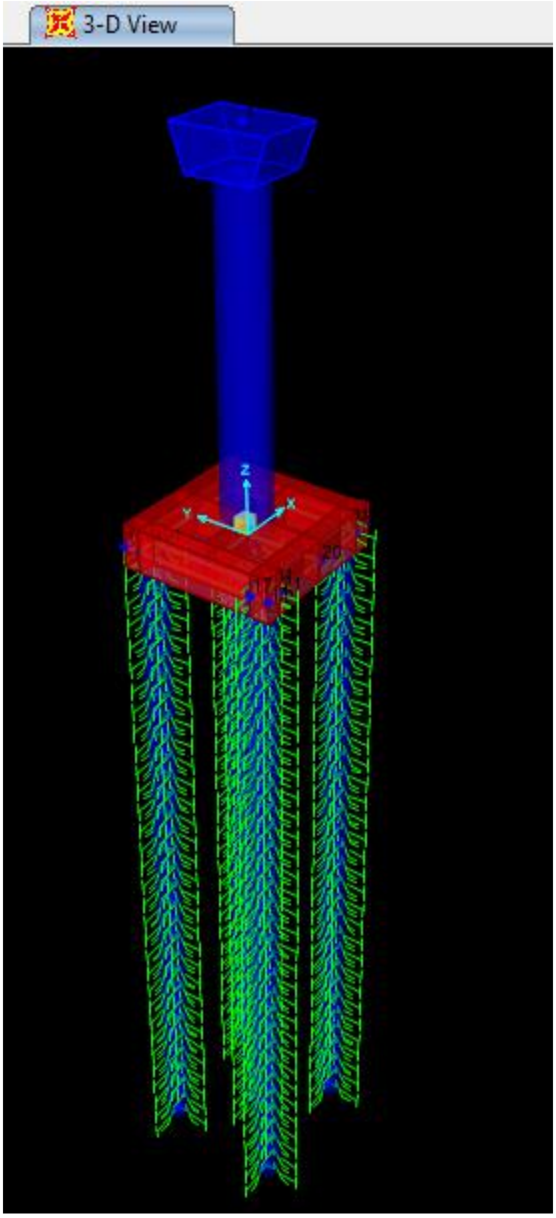
The analysis models include material inelasticity in the form of lumped plasticity at the location of potential plastic hinge locations. Non-linear behaviour of the piers was simulated using axial force–biaxial moment (P-M-M) interaction hinges. The pile cap was modelled with thick shell element. Soil structure Interaction during seismic shaking is incorporated explicitly into the analytical model using various non-linear springs so that both strength and stiffness degradation, and hysteretic soil damping with increase in deformation can be addressed approximately.

The top node of the pier was selected as 'control node' for monitoring the displacement. The pier is subjected to a monotonically increasing displacement (in increments) at its control node in longitudinal direction until target displacement is reached. Gravity loads are initially applied in a force-controlled manner until the total load reaches the target value, which is same as the design gravity loads for linear analysis.

Figure 4.4 to **Figure 4.6** presents rendered view of the analytical model considered.



(a) Line Model



(b) Extruded view

Figure 4.4: **3D-Modelling of Bridge Pier in SAP2000**

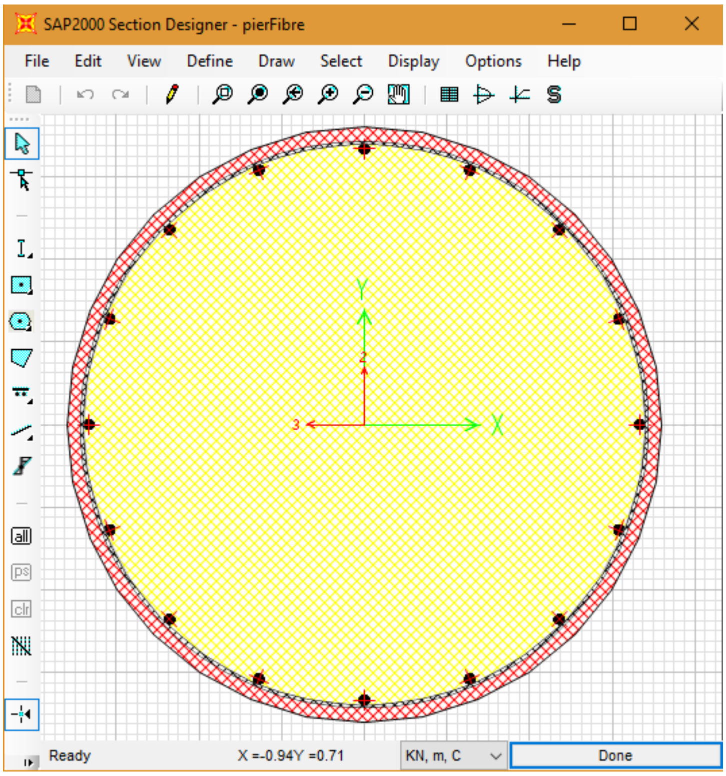


Figure 4.5 (a): Short Column Section Designed as per IRC:6-2000

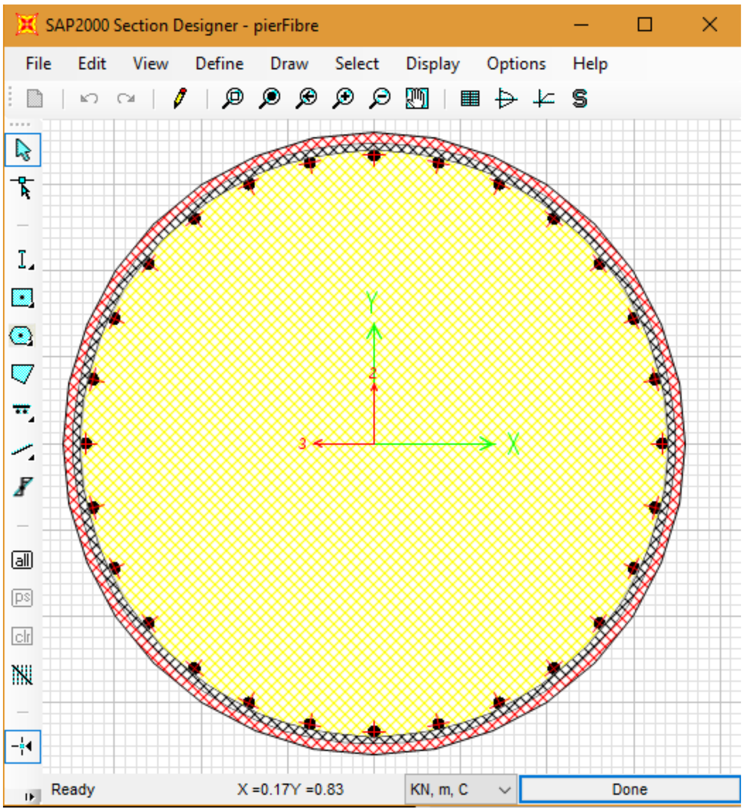


Figure 4.5 (b): Short Column Section Designed as per IRC:6-2014

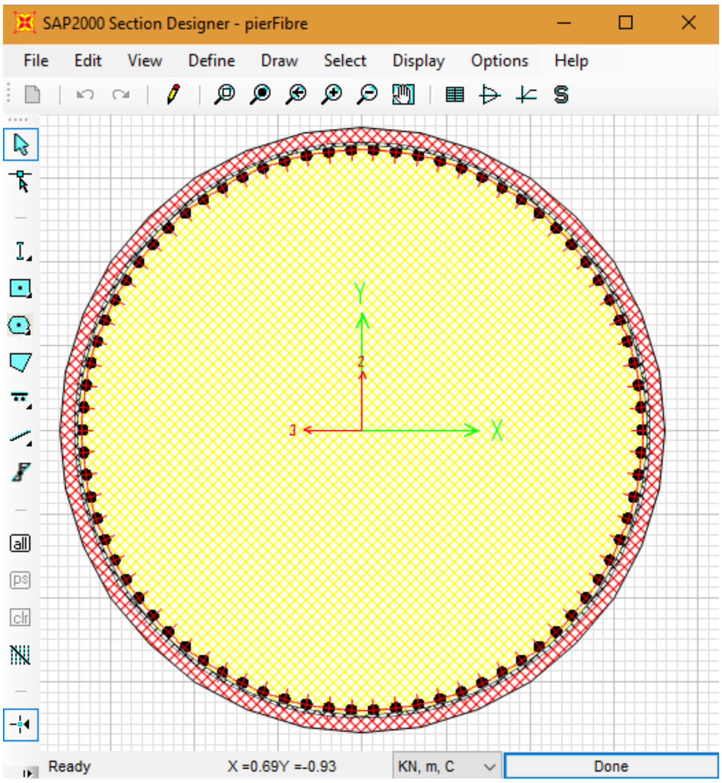


Figure 4.6 (a): Long Column Section Designed as per IRC:6-2000

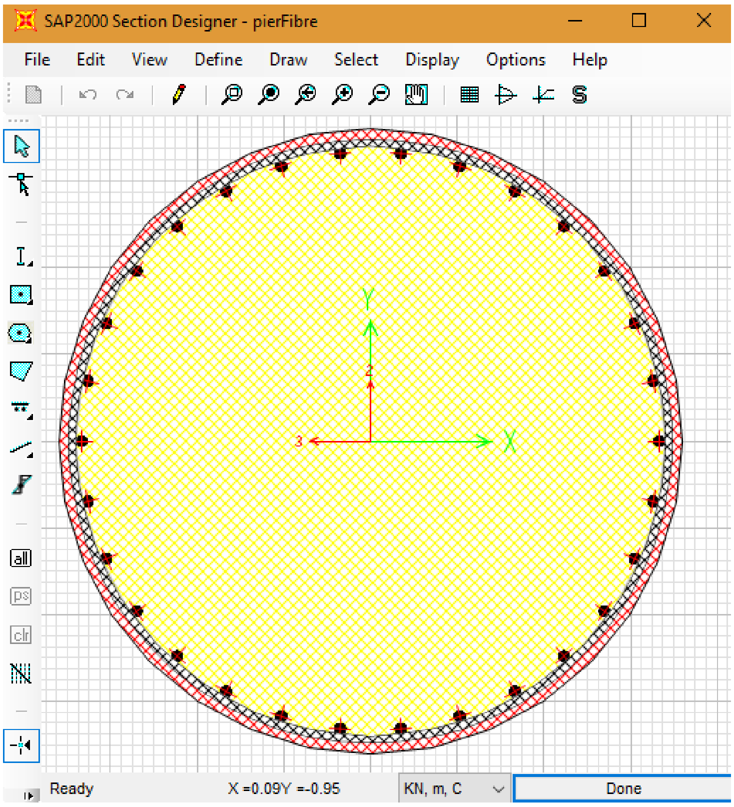


Figure 4.6 (b): Long Column Section Designed as per IRC:6-2014

4.6 Steps followed for SAP2000 modelling

- **Step: 1 Material Properties Definition**

  For design purpose characteristic strengths of the constituent materials are used. However, for assessment the probable strength of materials is considered. Probable strength of concrete M 35 is 36.6 MPa, M 40 is 39.9 MPa and of reinforcing bars of grade Fe500 is 550 MPa. These strengths are used in the SAP2000 model. This is because, we are assessing actual performance of the structure and not the design performance of the structure. The inputs provided for the material definition in SAP2000 model are shown in **Figure 4.7**.

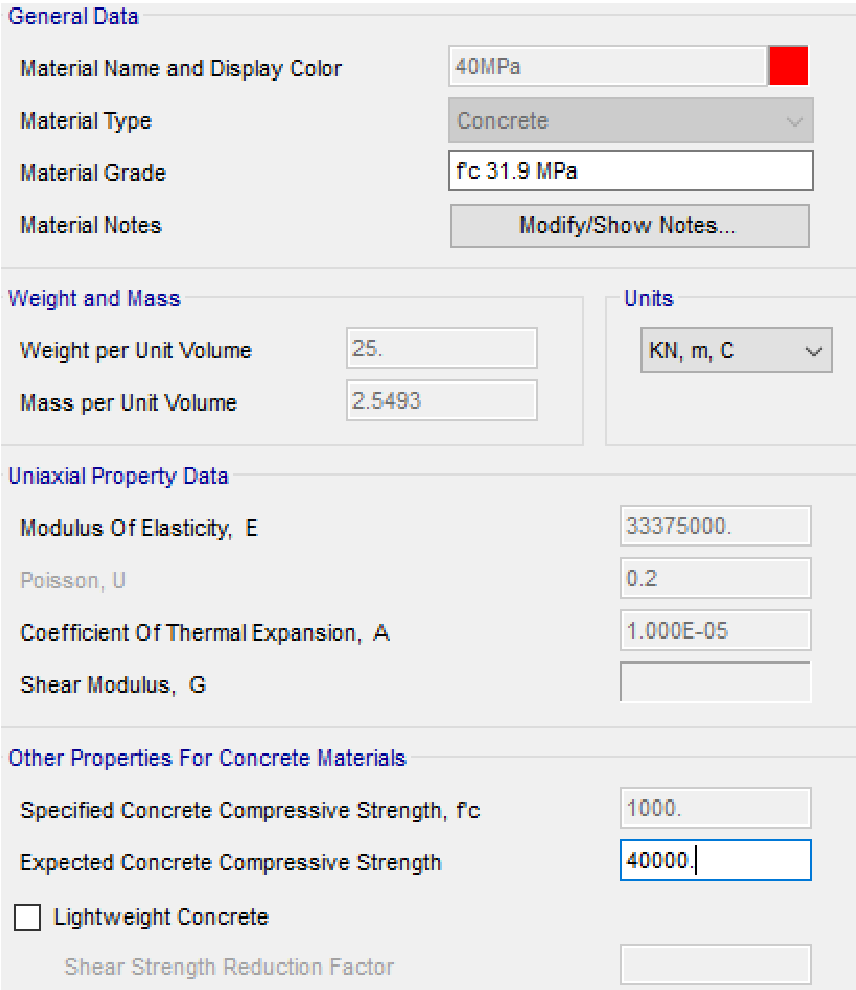


Figure 4.7 (a): Concrete Material Property Data in SAP2000

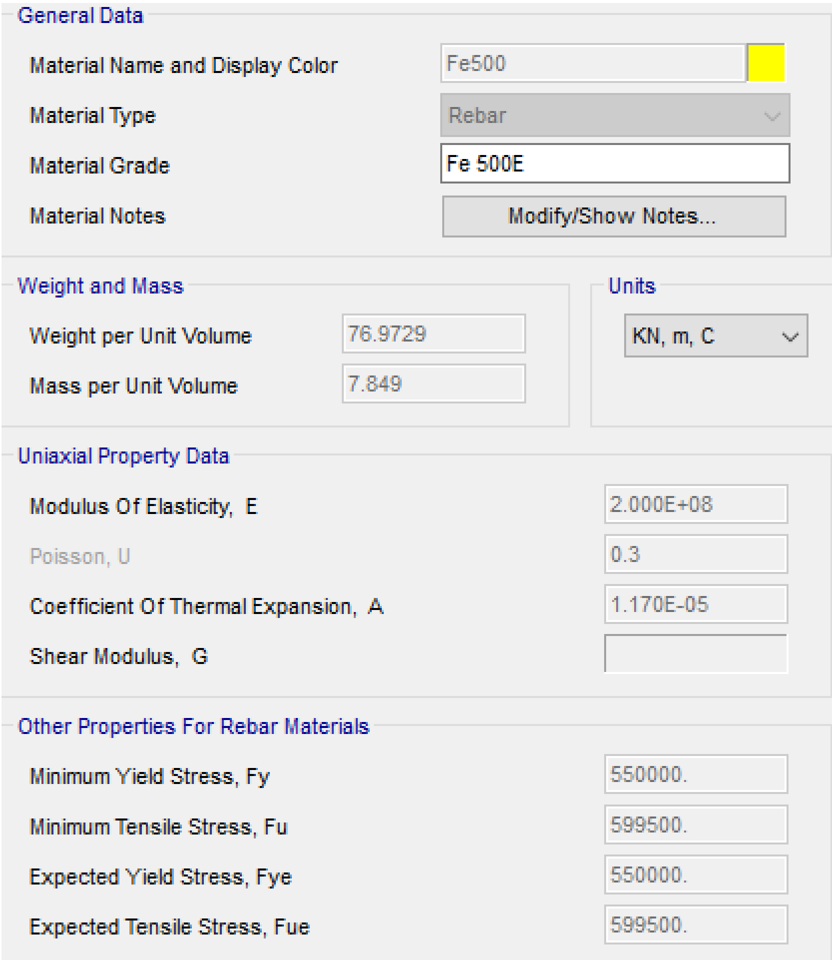


Figure 4.7 (b): Rebar Material Property Data in SAP2000

- **Step: 2 Frame Sections Definition**

  Using the section designer, the column section is defined. Actual longitudinal reinforcement is given as input. Using the transverse reinforcement details, Mander confined concrete model is defined.

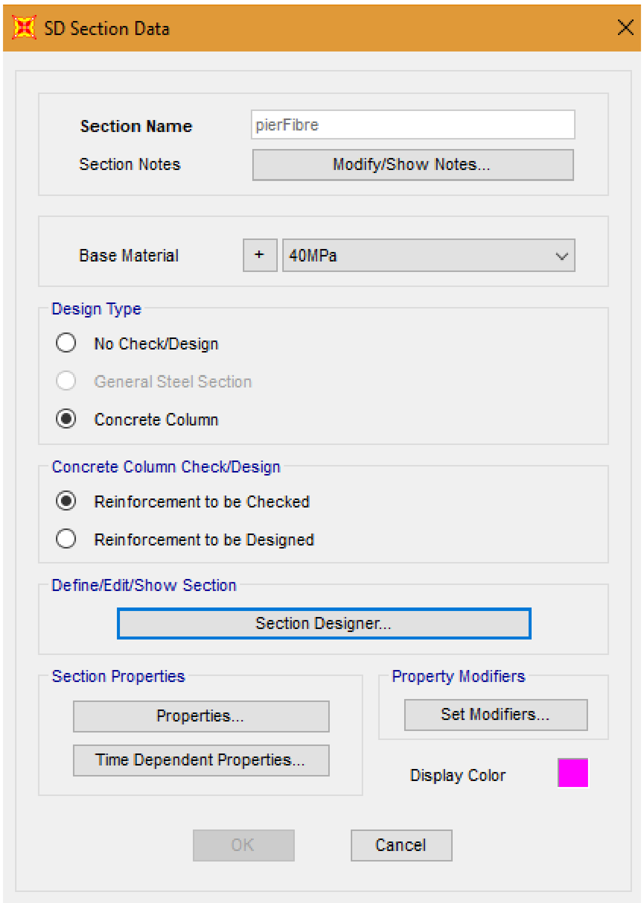


Figure 4.8: Section Details in SAP2000

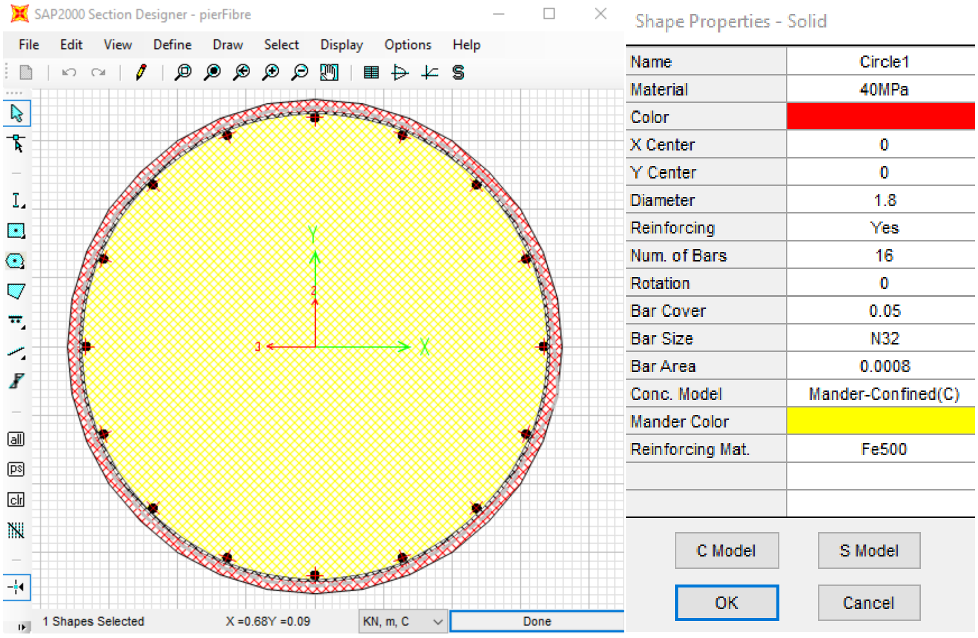


Figure 4.9: Section Designer in SAP2000

Typical stress strain curve with the limiting values marked on it are shown in the **Figure 4.10**. Where ε_{cu} and f_{cu} are ultimate strain and corresponding moment ε_{cc} and f_{cc} maximum strain and compressive stress in confined concrete respectively.

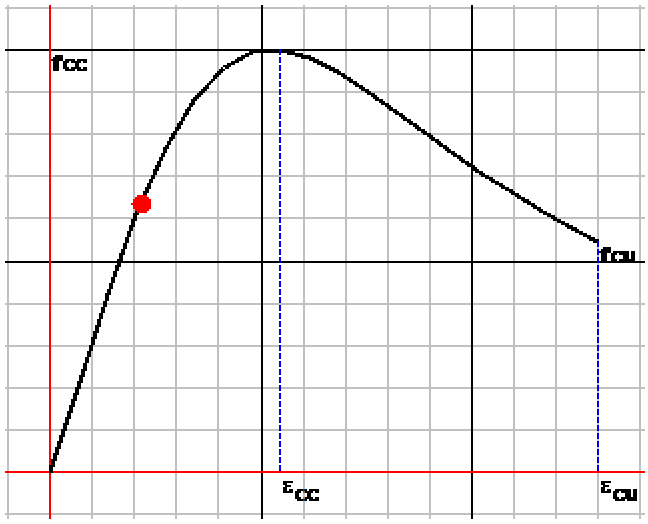


Figure 4.10: Typical Stress-strain curve

The stress strain graphs are plotted for each of the defined pier sections with provided longitudinal and transverse sections in the **Figure 4.11**.

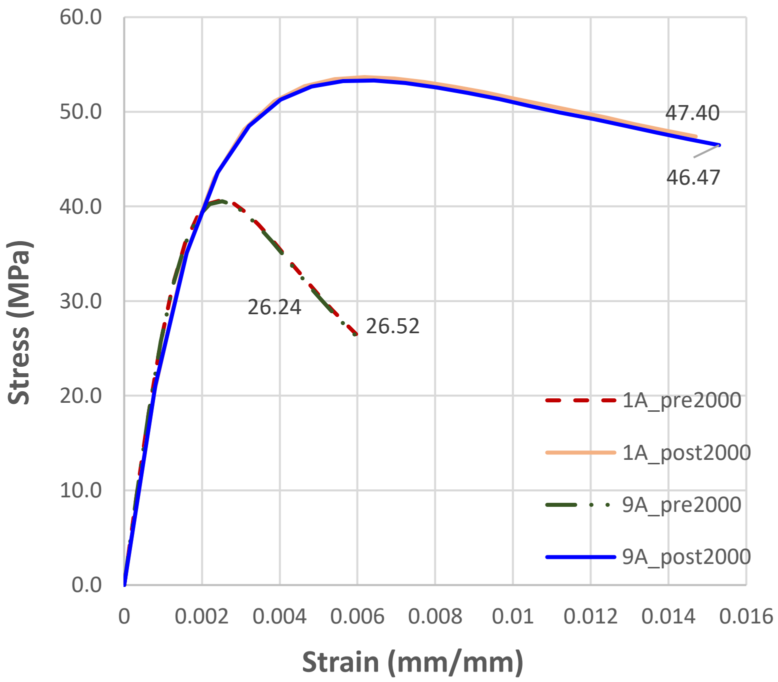


Figure 4.11: Stress-strain curve for Pier sections

The comparative values of the above-mentioned stress and strains for various pier sections are shown in **Table 4.2**.

Table 4.2: Limiting stress-strain for the Piers

Pier Marked	ε_{cc}	ε_{cu}	f_{cu}	f_{cc}
1A-pre2000	0.0025	0.0060	40.69	26.52
1A-post2000	0.0062	0.0147	53.67	47.40
9A-pre2000	0.0025	0.0060	40.55	26.24
9A-post2000	0.0064	0.0153	53.51	46.47

- **Step: 3 Structure Definition**

  The analytical frame structure has been modelled in SAP with the actual dimensions of the structure. The clear height of the column is to be taken at the base of the piercap. The top of the column is defined at the bearing top location. The location of the nodes and the developed model is shown in the **Figure 4.12**.

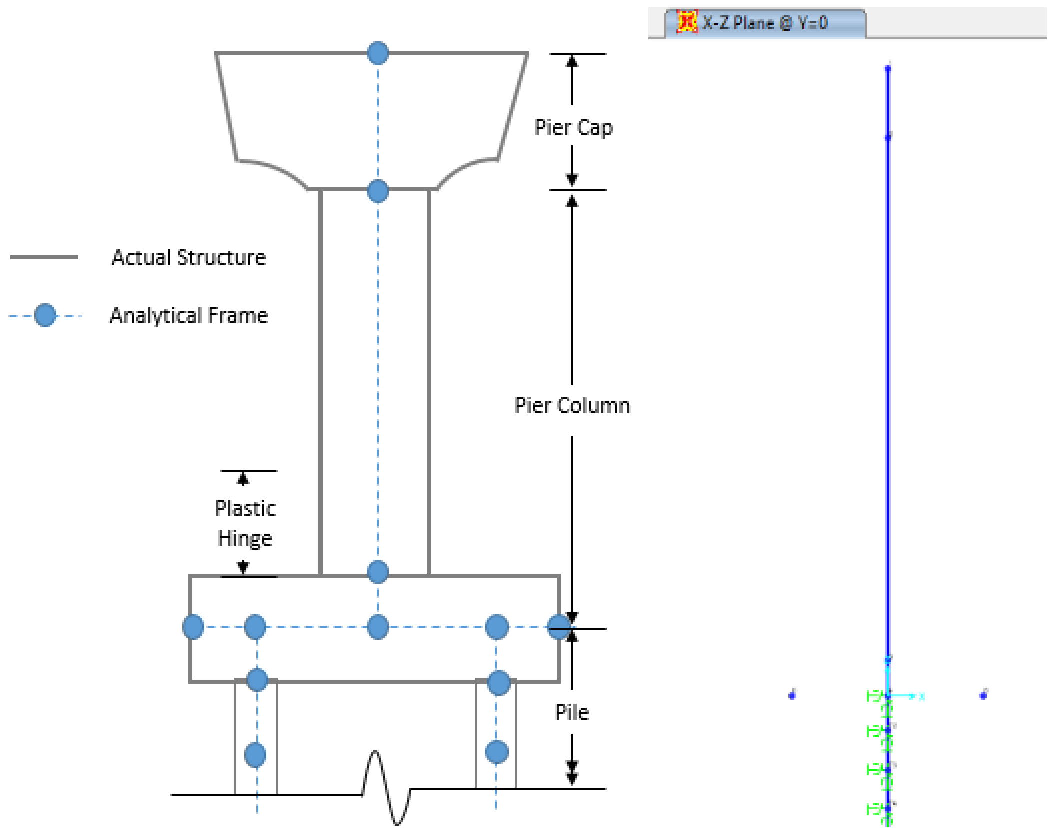


Figure 4.12: Structural Model in SAP2000

- **Step: 4 Definition and Location of Fibre Hinges**

  The idealized cantilever models assume the formation of plastic hinges at the end of each segment near the point of fixity of the column. The curvature of the column increases linearly with height from the point of inflection (zero moment) to the point of fixity (maximum moment). In the plastic hinge zone, the plastic moment and curvature are assumed constant as shown in **Figure 4.13**.

  Fibre P-M2-M3 hinge is defined for the pier. The approximate length of plastic hinge has been calculated and introduced in the model from the empirical formula by Priestly et. al.

$L_P = kL_C + L_{SP} \geq 2L_{SP}$, where L_p is Plastic Hinge Length;

$k = 0.2\left(\frac{f_u}{f_y} - 1\right) \leq 0.08$

$L_{SP} = 0.022 f_{ye} d_{bl}$ (f_{ye} in MPa);

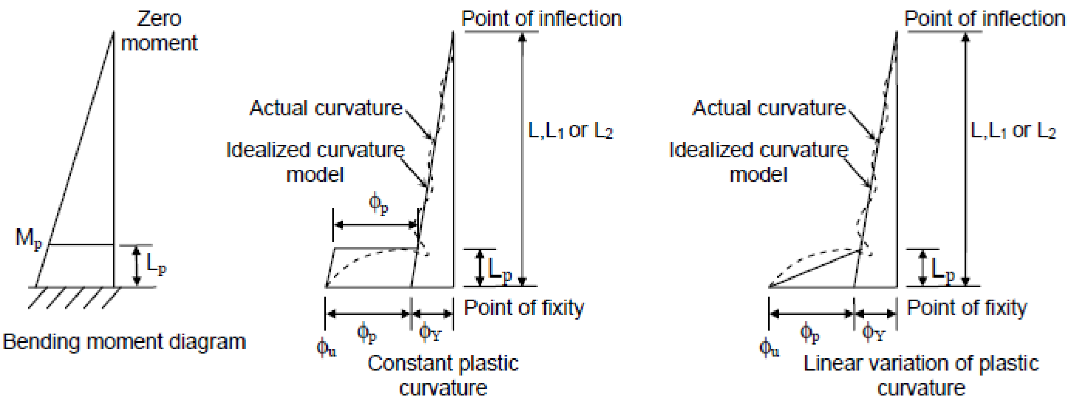


Figure 4.13: Local Deformation Capacity in Pier Column

- **Step: 5 Assign Fibre Hinges to the Frame**

  The plastic hinge zone is assigned in the inelastic model at the column base with its centroid at the centroid of the plastic hinge length, while the rest of the element outside the plastic hinge is assigned an elastic frame element with a solid cross section, according to its geometry, using effective section properties.

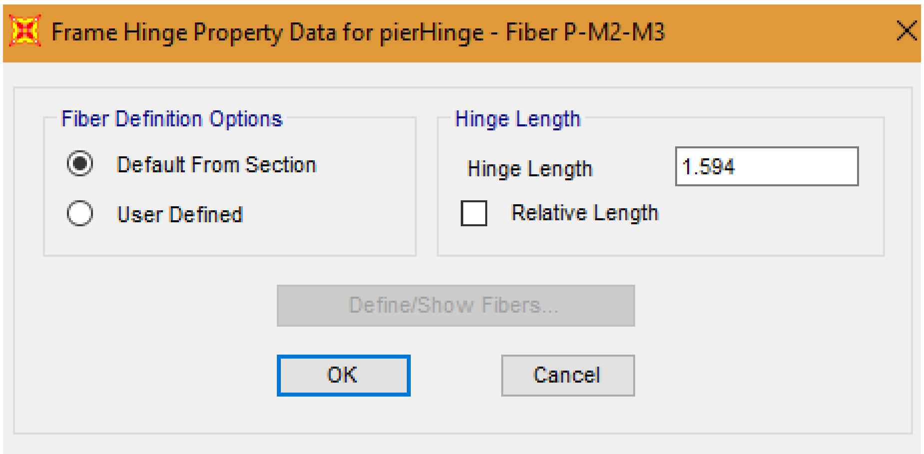


Figure 4.14: Plastic Hinge Definition

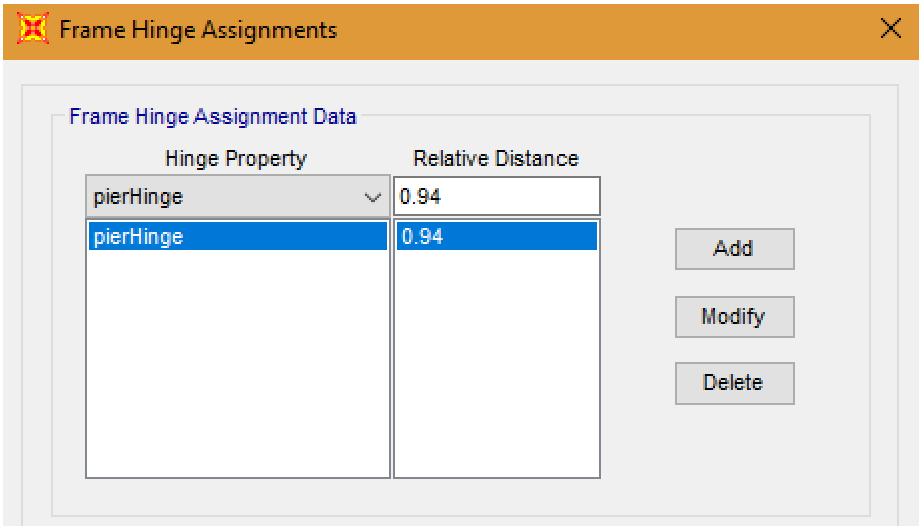


Figure 4.15: Plastic Hinge Assignment

- **Step: 6 Soil Structure Interaction**

  The dynamic interaction between the soil and the pile shaft of bridge foundations has a significant effect on the seismic response of bridges. Although it is impractical to include all the effects of the soil and foundation on the earthquake response of a bridge, yet the soil-structure interaction introduces flexibility and energy dissipation into the system compared with an assumption of a rigid or pinned support. The stiffness and damping properties of a foundation depend on the characteristics of the soil, piles, and the connections between the piles and the pilecap. The group effects of the large number of piles in bridge foundations can significantly affect the dynamic properties (Section 4.2.2 of ATC 32).

Soil springs have been calculated at 1m interval using the available bore log data as shown in **Figure 4.16** at the bridge locations and the same has been assigned at non-linear soil links in SAP.

Geotechnical Interpretation Report: Joka-BBD Bag corridor

Bore Hole No.	**Terminating Depth (m)**	**Standing Water Table (in RL)**	**Chainage**	**Northing**	**Easting**	**Reduced Level**
P2/4	40.100	2.635	+6229.473	777397.326	533044.555	4.835

Layer Details								
Stratum No.	Description	Depth below EGL (in R.L.)		Average Field N-value	Bulk Density (t/m^3)	Liquid Limit (%)	Plasticity Index (%)	Shear Strength Parameters
		From	To					
I	Soft brownish grey silty clay with traces of rubbish	4.835	3.835	-	-	-	-	
III	Soft to firm greyish clayey silt with traces of Mica	3.835	-4.965	3	1.71	46	20	C =2.9 t/m^2, Φ = 0 deg
IIIA	Medium dense whitish grey micaceous silty fine sand	-4.965	-12.665	19	1.89*	Non-Plastic		C =0.0 t/m^2, Φ = 28 deg
V	Firm to stiff greyish clayey silt, Sand percentage observed at the bottom	-12.665	-16.965	7	1.86	42	18	C =4.5 t/m^2, Φ = 0 deg
VI	Medium dense to very dense whitish grey micaceous silty fine to medium sand	-16.965	-35.265	19	1.92*	Non-Plastic		C =0.0 t/m^2, Φ = 32 deg

Figure 4.16: Available Bore Log Data

The soil-pile interaction is modelled using the Beam on Nonlinear Winkler Foundation (BNWF) method in which the pile is modelled using beam elements and nonlinear spring elements are used to represent the vertical and lateral response of the surrounding soil. The material inelasticity and geometric nonlinearity of the pile elements are included in the analysis model, wherever appropriate.

There are three types of soil springs. The lateral soil resistance is modelled using the p-y springs. The t-z springs represent the skin friction along the pile shaft and the q-z spring simulates the end bearing response. All springs are assumed uncoupled, i.e., response of one spring depends only on the respective soil deformation at the spring location and is not influenced by the response of other springs.

To simulate the uncoupled vertical and lateral responses of the soil springs, three nodes of same coordinates are required to define a zero-length spring element with uniaxial translational spring triplet as shown in the **Figure 4.17**. The pile nodes are part of the pile elements. The soil spring connects the slave and the fixed nodes. The slave nodes are connected to the pile nodes via kinematic constraints of relevant degrees of freedom (DOFs).

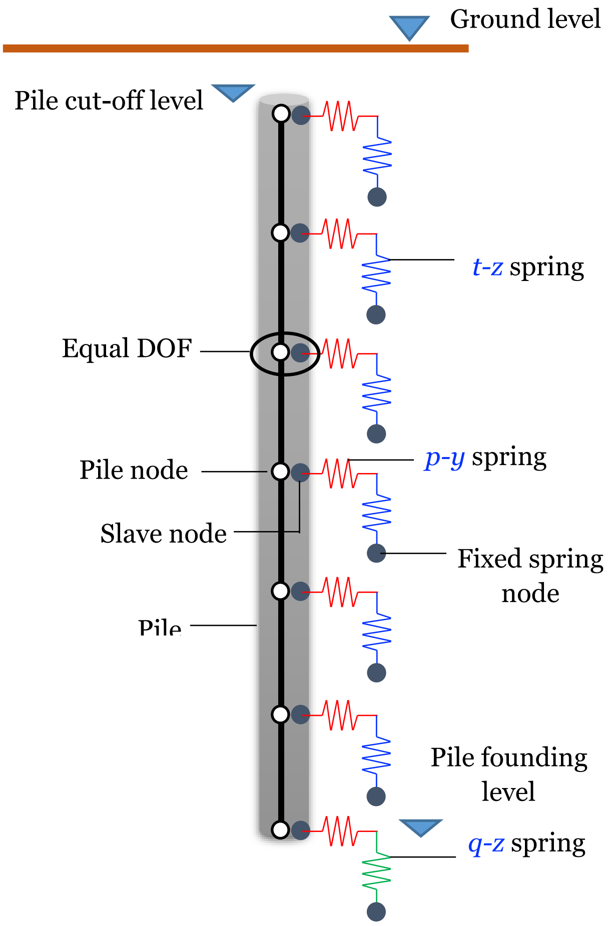


Figure 4.17: A Typical Soil-Pile Interaction using Winkler Springs

Table 4.3: Non-linear Spring Assigned

Spring RL	Spring ID	Soil Type	Non-liquefiable soil					
			p_{ult} (kN)	y_{50} (mm)	k_{50} (kN/m)	t_{ult} (kN)	z_{50} (mm)	k_{50} (kN/m)
-1.00	1	1	104	60.00	869	69	3.72	9222
-2.00	2	1	126	60.00	1046	69	3.72	9222
-3.00	3	1	147	60.00	1223	69	3.72	9222
-4.00	4	1	168	60.00	1400	69	3.72	9222
-5.00	5	1	189	60.00	1578	69	3.72	9222
-6.00	6	1	211	60.00	1755	69	3.72	9222
-7.00	7	1	252	60.00	2102	77	3.72	10286
-8.00	8	1	274	60.00	2285	77	3.72	10286
-9.00	9	1	296	60.00	2468	77	3.72	10286
-10.00	10	1	313	60.00	2610	77	3.72	10286
-11.00	11	1	313	60.00	2610	77	3.72	10286
-12.00	12	2	1489	15.43	48262	225	1.30	86527
-13.00	13	2	1790	17.06	52458	252	1.30	96760
-14.00	14	2	2118	18.69	56655	278	1.30	106994
-15.00	15	2	2472	20.31	60852	305	1.30	117227
-16.00	16	2	2785	21.41	65048	331	1.30	127461
-17.00	17	2	3009	21.72	69245	358	1.30	137694
-18.00	18	2	3232	22.00	73442	385	1.30	147928
-19.00	19	2	3456	22.26	77638	411	1.30	158161
-20.00	20	2	3679	22.48	81835	438	1.30	168395
-21.00	21	2	6380	12.53	254538	539	1.30	207184
-22.00	22	2	6761	12.66	266954	571	1.30	219569
-23.00	23	2	7143	12.78	279371	603	1.30	231955
-24.00	24	2	7524	12.89	291787	635	1.30	244341
-25.00	25	2	7715	12.68	304204	651	1.30	250534
-26.00	26	2	7715	12.18	316620	651	1.30	250534
-27.00	27	2	7715	11.72	329037	651	1.30	250534
-28.00	28	2	7715	11.30	341453	651	1.30	250534
-29.00	29	2	7715	10.90	353870	651	1.30	250534

➢ **Step:7 Loads Definition**

Dead Load from the superstructure, the piercap weight and 1/3 of pier column load has been applied as gravity loading at the top of piercap. Remaining 2/3 of pier load is applied at the base. Pilecap load has been applied at top of the piles.

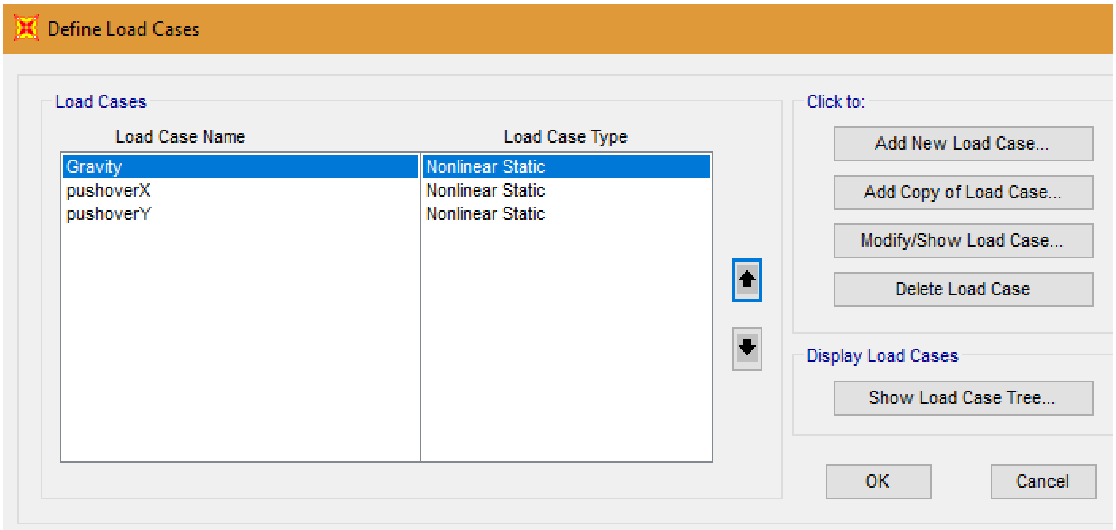


Figure 4.18: Load Definition in SAP2000

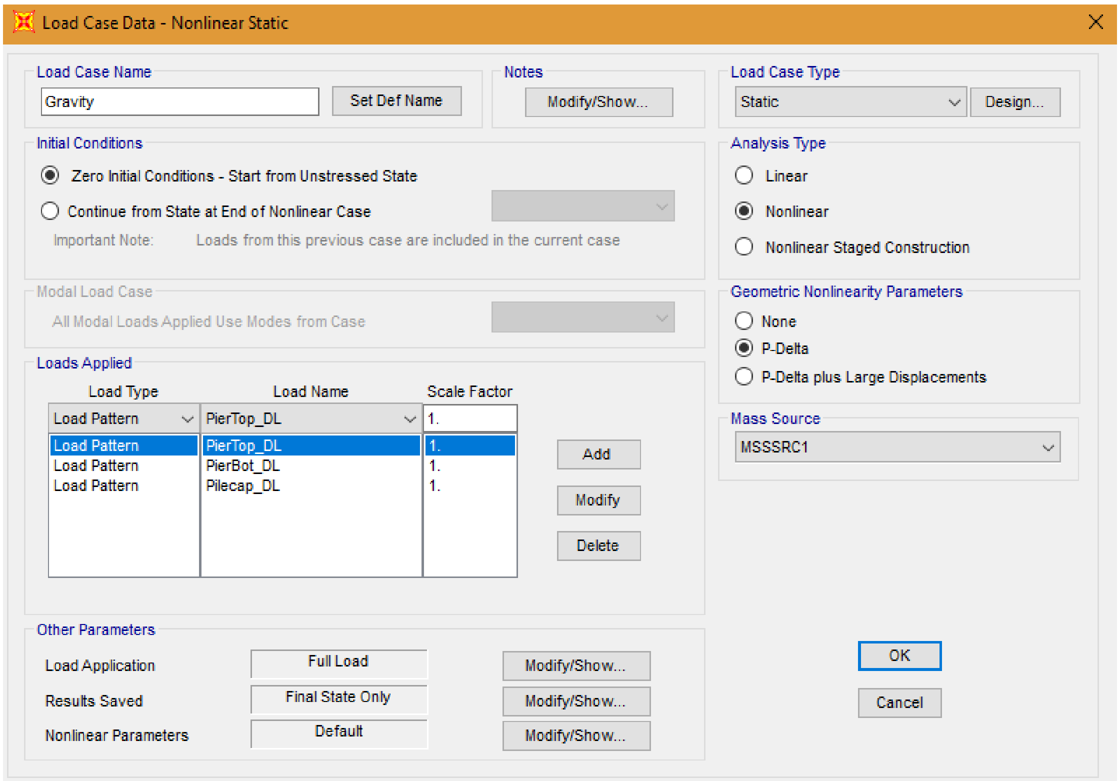


Figure 4.19: Gravity Load Case

Joint Loads - Force

File View Edit Format-Filter-Sort Select Options

Units: As Noted

Filter:

Joint Text	LoadPat Text	CoordSys Text	F1 KN	F2 KN	F3 KN	M1 KN-m
1	pushoverX	GLOBAL	100	0	0	0
1	pushoverY	GLOBAL	0	100	0	0
1	PierTop_DL	GLOBAL	0	0	-6874	0
3	PierBot_DL	GLOBAL	0	0	-289	0
101	Pilecap_DL	GLOBAL	0	0	-295	0
201	Pilecap_DL	GLOBAL	0	0	-295	0
301	Pilecap_DL	GLOBAL	0	0	-295	0
401	Pilecap_DL	GLOBAL	0	0	-295	0

Figure 4.20 (a): Load Value of Short Column

Joint Loads - Force

File View Edit Format-Filter-Sort Select Options

Units: As Noted

Filter:

Joint Text	LoadPat Text	CoordSys Text	F1 KN	F2 KN	F3 KN	M1 KN-m
1	pushoverX	GLOBAL	100	0	0	0
1	pushoverY	GLOBAL	0	100	0	0
1	PierTop_DL	GLOBAL	0	0	-7023	0
3	PierBot_DL	GLOBAL	0	0	-585	0
4	Pilecap_DL	GLOBAL	0	0	-404	0
101	Pilecap_DL	GLOBAL	0	0	-404	0
201	Pilecap_DL	GLOBAL	0	0	-404	0
301	Pilecap_DL	GLOBAL	0	0	-404	0
401	Pilecap_DL	GLOBAL	0	0	-404	0

Figure 4.20 (b): Load Value of Long Column

➢ **Step: 8 Lateral Load Definition**

Lateral displacement is applied as pushover load at piercap top both in traffic direction (Pushover-X), and transverse to it (Pushover-Y). Since the SDOF model of substructure is considered, results of both the direction will be similar. In our case we have focused on Pushover-X load for analysis.

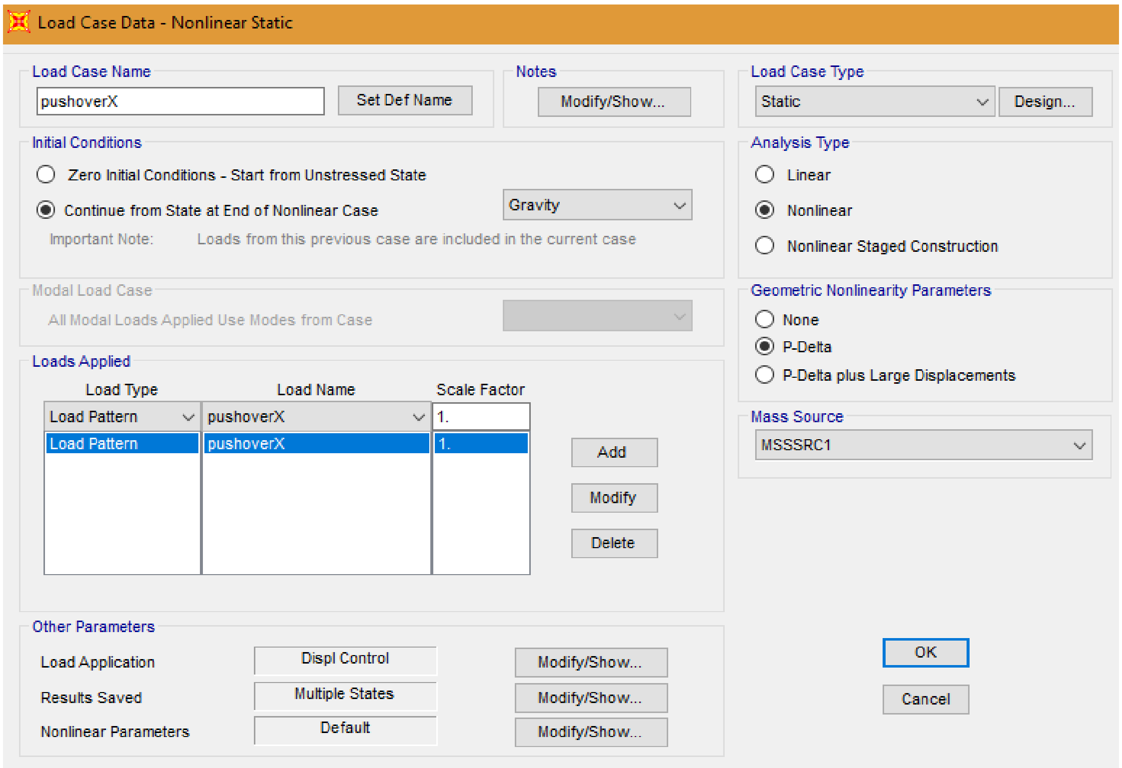


Figure 4.21: Pushover Load Definition

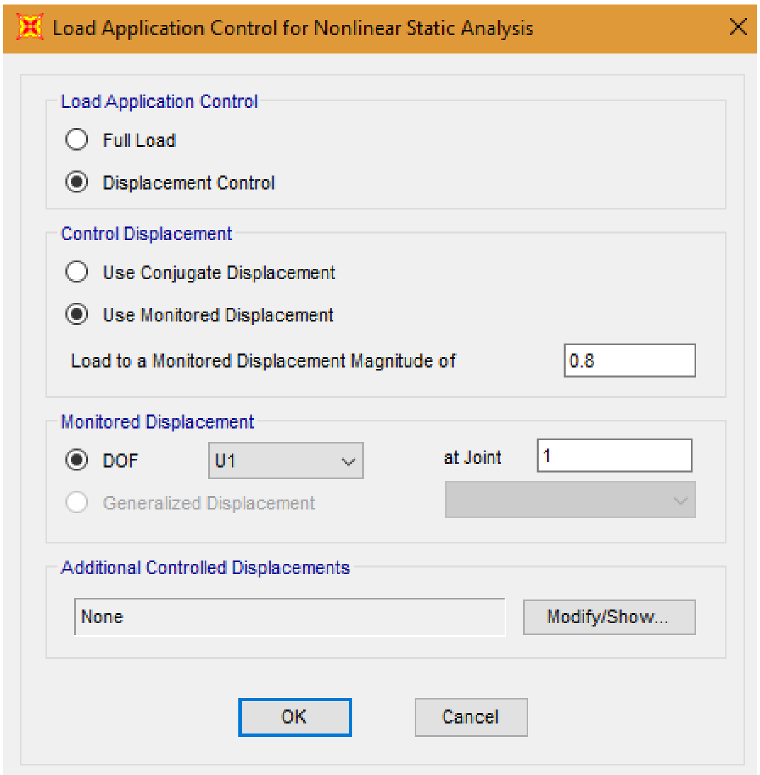


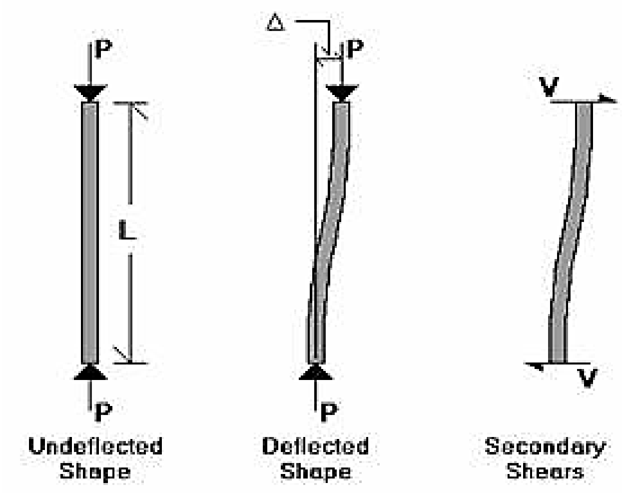
Figure 4.22: Pushover Load Application

4.7 Parametric Study

Amongst the numerous properties to be considered for analysis of the pier models, the following were primarily compared for carrying out a parametric study of the efficiency of the various models studied.

4.7.1 P-Δ Effect

To account for the secondary moments induced by deflections in the members, P-Delta analysis is used in substructure model. This is considered because the ends of the member may no longer be vertical in the deflected position. The magnitude of the secondary moment is equal to “P”, the axial force in the member, times “Delta”, the distance one end of the member is offset from the other end.

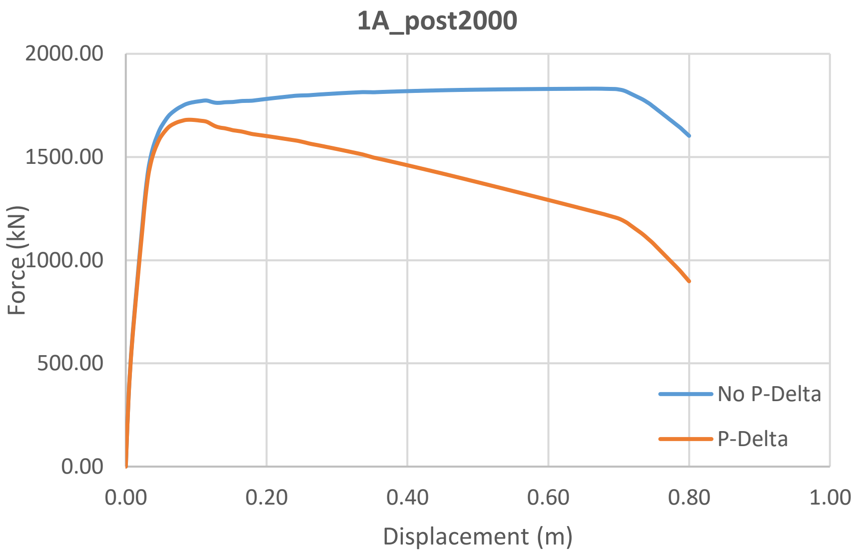


$$P * \Delta = V * L$$

Figure 4.23: P-Δ Behaviour

P-delta is a second-order non-linear effect that use the displacements (deformations) from the previous step, update the geometry of the structure, which is deformed due to incremental loads, and satisfy the equilibrium on the deformed geometry. A new set of displacements are then computed and added to the ones from the previous step.

To observe its effect, piers are modelled with two parameters: (i) With P-Δ, (ii) Without P-Δ. Pushover Analyses are performed and the curve for the same have been presented in the **Figure 4.24**.



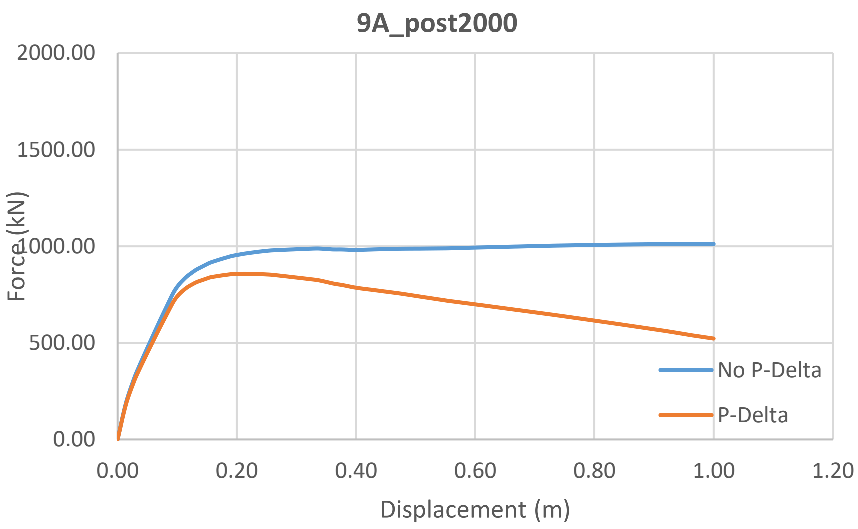


Figure 4.24: Effect of P-Δ in Pushover Analysis

4.7.2 Plastic Hinge Characteristics

There are multiple possibilities to model the plastic hinge when this concept is used in structural analysis. In this paper, two possibilities are considered: plastic hinge of P-M3 type and plastic hinge of fibre type (P-M2-M3 type).

The plastic hinge of P-M3 type is defined according to ASCE41. Its behaviour is presented in **Figure 4.25**.

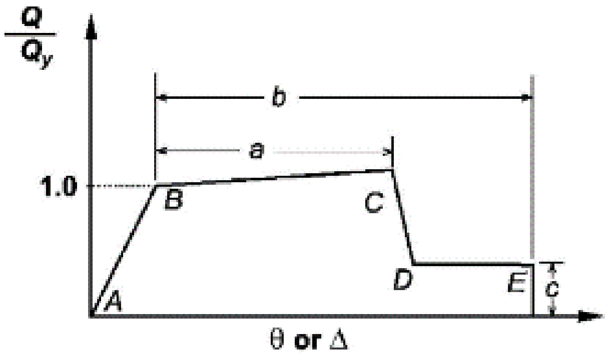


Figure 4.25: M3 Plastic Hinge Behaviour

The fibre plastic hinge (P-M2-M3 type) involves a process of dividing the section in multiple longitudinal fibres. In this method, for each fibre in the cross section, the material nonlinear stress-strain curve is used to define the axial σ - ε relationship. Summing up the behaviour of all the fibres in the cross section and multiplying by the hinge length gives the axial force-deformation and biaxial moment-rotation relationships.

To distinguish the two types of Plastic Hinges, a typical pier is analysed for the two parameters: (i) With M3 plastic hinge, (ii) With P-M2-M3 plastic hinge. The M3 plastic behaviour has been calculated from ASCE-41 17 (Table 10-9) and plotted in **Figure 4.26**.

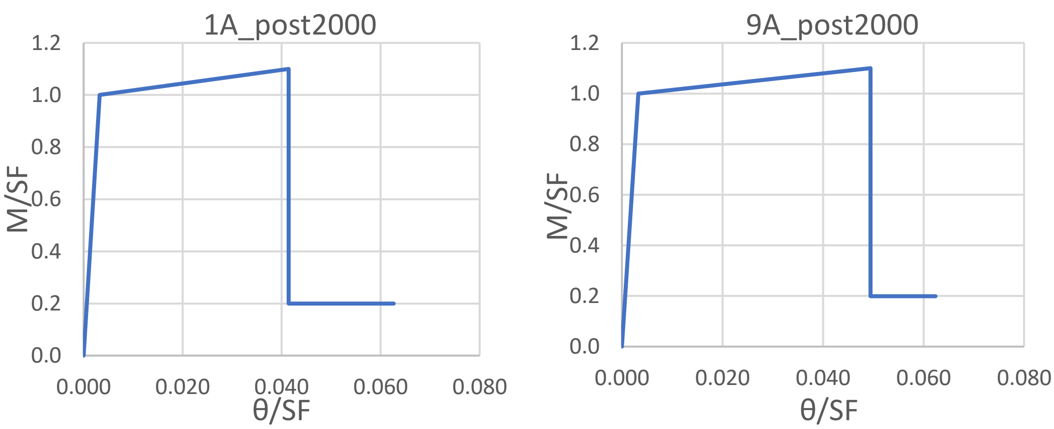


Figure 4.26: Moment Curvature plot at P-M3 Hinge Location

The backbone curves are calculated and put as an input to define P-M3 plastic hinge properties and the substructures are analysed with the same. While the fibre P-M2-M3 type plastic hinge is auto generated in SAP2000, only the plastic hinge length is calculated using equations by Priestly et al. Pushover Analyses are performed and the curve for the same have been presented in the **Figure 4.27**.

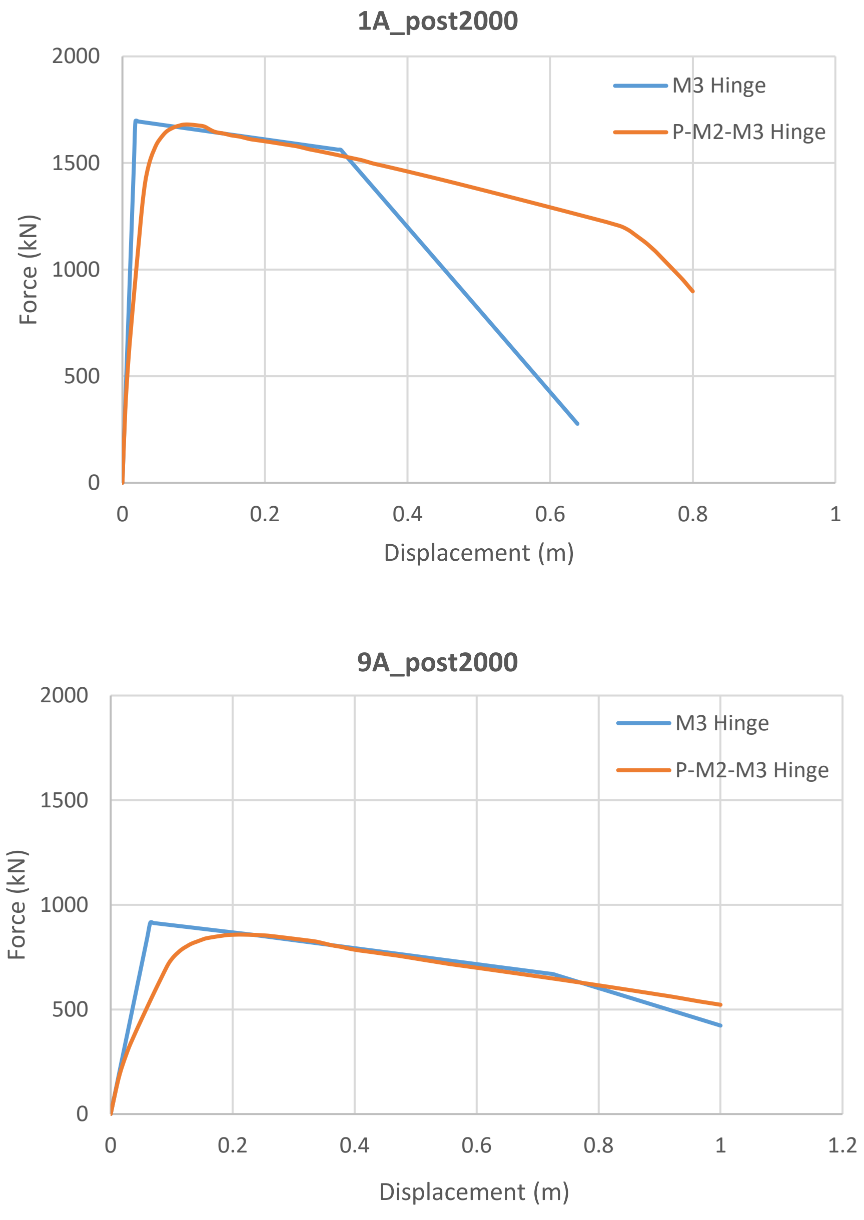


Figure 4.27: Effect of Plastic Hinge types in Pushover Analysis

4.7.3 Effect of Foundation Flexibility

For a structure supporting on flexible foundation, as Soil Structure Interaction (SSI) extends the elastic period and increases damping of the structure-foundation elastic system, the structural ductility could also be affected by frequency-dependent foundation-soil compliances. To find the answer to this and to understand the potential benefit of taking soil structure interaction into account when performing finite element seismic assessment of structures through nonlinear pushover analyses, the piers are modelled both with and without pile foundation.

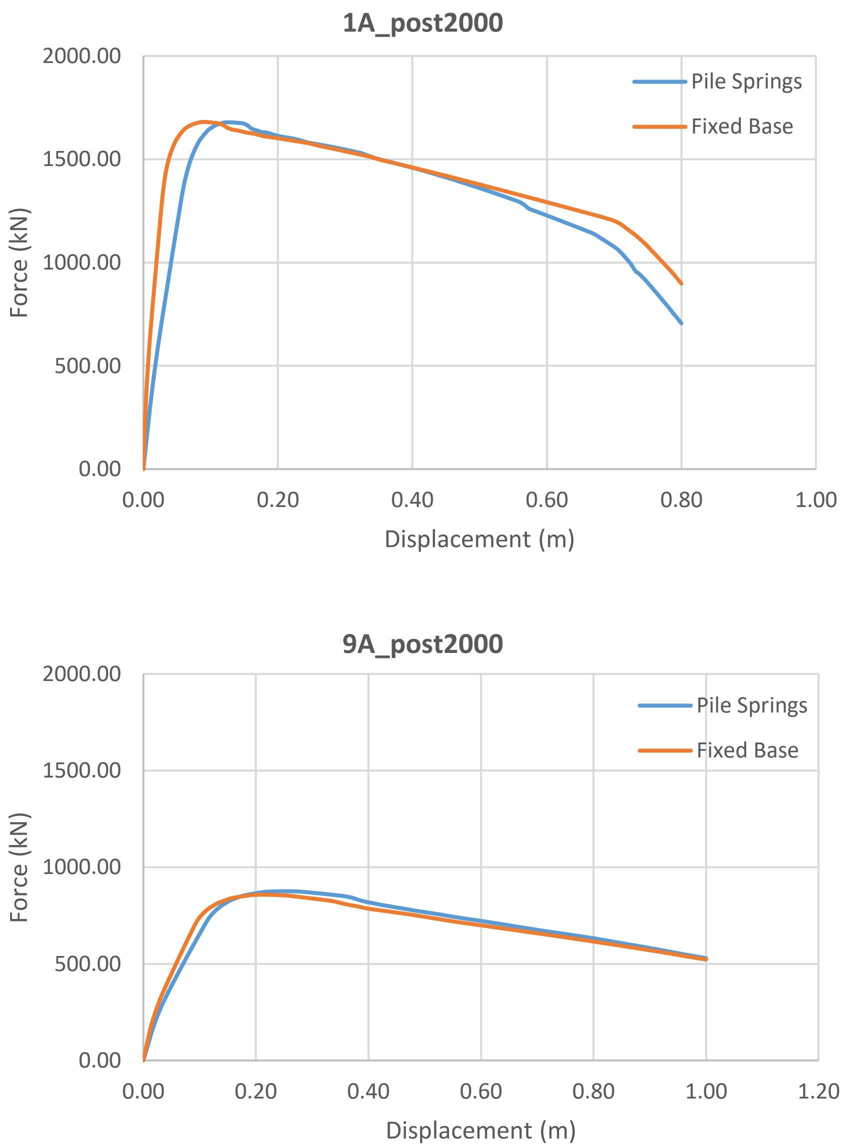


Figure 4.28: Effect of Foundation Flexibility in Pushover Analysis

Based on the results of the parametric study, the parameters selected in this study for assessment of RCC substructure are:

(i) P-Δ analysis,
(ii) P-M2-M3 plastic hinge
(iii) With SSI

4.8 Steps followed for Bridge Pier Assessment using MCSM

- **Step: 1 Conversion of Pushover Curve to Capacity Spectrum (ADRS format)**

  If the base acceleration (S_a) is plotted with respect to the top displacement (S_d), it is termed as capacity spectrum. The spectral acceleration and the spectral displacement, as calculated from the linear elastic response spectrum for a certain damping value is plotted as acceleration-displacement-response-spectrum (ADRS). Any point (V_B, Δ_{top}) on the capacity curve (Pushover curve) is converted to corresponding point (S_a, S_d) on the capacity spectrum using the following equations:

  $Sa = V/ (W \times \alpha)$

  $Sd = \Delta_{top} / PF \times \Phi_{top}$

  Where, V = the base shear

  W = Appropriate Load acting at the top of pier

  α = modal mass co-efficient for the first natural mode

  Δ = roof displacement obtained from pushover curve

  PF = modal participation factor for first natural mode

  Φ = roof level amplitude of the first mode

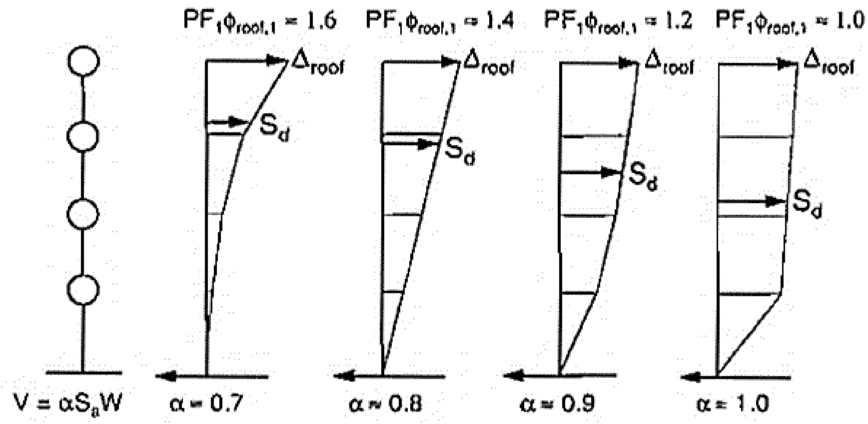


Figure 4.29: Factors Depending on Mode Shapes

- **Step: 2 Bi-linearize the Capacity Spectrum**

  A bilinear representation of the capacity spectrum as obtained above, is needed to estimate the effective damping and appropriate reduction of the of the spectral demand. Construction of bilinear representation requires the definition of the point a_{pi}, d_{pi}. This point is the trial performance point and is calculated in accordance with the procedures in ATC-40[8].

- **Step: 2 Determination of Dynamic Properties of the Equivalent SDOF Systems**

  For the bilinear representation developed, the values of dynamic properties like post-elastic stiffness α, ductility μ, are then calculated. Using these calculated values, the corresponding effective damping βeff is then calculated. Similarly, the corresponding effective period 'Teff', is then calculated. Calculation of dynamic properties like α, T_s, K_{eq} of the eSDOFs of the four pier systems with the assumed performance and yield points are presented in the **Table 4.4** below.

Table 4.4: Dynamic properties of eSDOF Systems

Pier Marked	Origin			Performance point (assumed)			Yield point (assumed)		Dynamic properties of pier SDOFs		
	Step	a_0 (g)	d_0 (mm)	Step	a_{pi} (g)	d_{pi} (mm)	a_y (g)	d_y (mm)	α	T_0 (s)	K_{ey} (g/mm)
1A_pre2000	0	0.000	0	75	0.167	150	0.185	28	-0.02	0.781	0.007
1A_post2000	0	0.000	0	83	0.221	332	0.253	78	-0.04	1.113	0.003
9A_pre2000	0	0.000	0	77	0.239	385	0.250	200	-0.05	1.795	0.001
9A_post2000	0	0.000	0	130	0.099	650	0.139	125	-0.07	1.903	0.001

- **Step: 3 Developing Response Spectra of Ground Motions**

  Site-specific approach to developing response spectra is used, in cases where the objective is to develop ground motions that are more accurate for the local seismic and site conditions. Site specific response spectra can be determined from the general procedure using national ground motion maps and site factors.

  Due to unavailability of site-specific data, in this study Response Spectrum has been considered from available Indian standards. The following equations are used for plotting normalized spectrum, and then it is multiplied with A_h to derive design spectrum both for DBE and MCE, where

A_h = Horizontal seismic coefficient = $(Z/2) \times (I) \times (S_a/g)$

For rocky or hard soil sites,
Type I soil with N > 30

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T, & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.40 \\ 1.00/T & 0.40 \le T \le 4.00 \end{cases}$$

For medium soil sites,
Type II soil with 10 < N ≤30

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T, & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.55 \\ 1.36/T & 0.55 \le T \le 4.00 \end{cases}$$

For soft soil sites,
Type III soil with N < 10

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T, & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.67 \\ 1.67/T & 0.67 \le T \le 4.00 \end{cases}$$

Figure 4.30: Response Spectrum (Courtesy: IRC: SP: 114-2014)

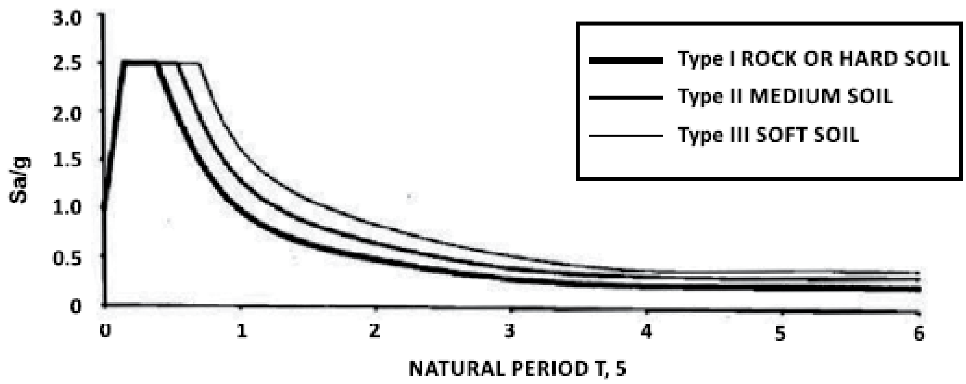


Figure 4.31: Spectra for Elastic Response Spectrum Method

- **Step: 3 Conversion of Demand Spectrum to ADRS format**

  Both the selected spectrum for Design Basis Earthquake and Maximum Considered Earthquake, modified for SSI when appropriate, are converted to an acceleration-displacement response spectrum format in accordance with the guidance in ATC-40.

These are the initial ADRS demand. The locus of the demand point in the ADRS plot is referred to as the demand spectrum which corresponds to the inelastic deformation of the structure.

- **Step: 4 Development of Modified ADRS plot**

  The MADRS demand spectrum is generated by modifying the ADRS for the various values of effective damping. Multiplying the ordinates of the ADRS demand corresponding to the effective damping ‘βeff’ as calculated in the step 2, by the modification factor results in the modified ADRS demand curve (MADRS).

- **Step:5 Determination of actual Performance Point**

  The performance point is defined as the intersection of the capacity spectrum with the modified ADRS (MADRS). A possible performance point is generated by the intersection of radial secant period T_{sec}, with the MADRS. An iterative process is followed where series of possible performance point is generated, which are compared with the assumed initial performance point, until both matches. In our study since two level of earthquake demand are considered, i.e. DBE and MCE, two corresponding performance point will be generated.

- **Step: 6 Determination of Performance of the Structure**

  The maximum displacement at the performance point (if reached) are then compared to the structural capacity, and the damage it experiences at that displacement are quantified. If they are within limits the structures are considered to withstand that particular level of earthquake. Otherwise, repair or replacement depending on the level of damage is suggested.

5.0 RESULTS AND DISCUSSIONS:

5.1 General

The behaviour of simply supported bridge superstructure supported over long pier, short pier has been designed by IRC: 6-2000[30] and IRC:6-2014[32] and analysed with the help of software SAP2000. The various structural parameters like displacement and base shear has been studied for seismic loading. The analysis was carried out by considering response spectrum and pushover analysis. The pushover analysis carried out by using standard pushover analysis. And finally, comparative assessment of the piers was carried out using Modified capacity spectrum method, and failure modes were identified for each of the piers designed both in pre2000's and post2000's. The results of each of these are discussed in the subsequent paragraphs in detail.

5.1.1 Force Based Design

Until the year 2002, the structures were designed using working stress method. The pier structures were designed for seismic force, where the effect of flexibility of slender structures were not considered. The magnitude of base shear and hence moment at the pier base was directly proportional to the height of the structure. Thus, the long column designed in the pre-2000's would require higher reinforcement percentage to cater the seismic responses.

However, in the post 2000's with the revision of seismic provisions the effect of flexible structure was taken into consideration while calculating seismic force. The variation of base shear with the height of the structure as per pre2000 and post2000 Indian seismic code is shown in **Figure 5.1**.

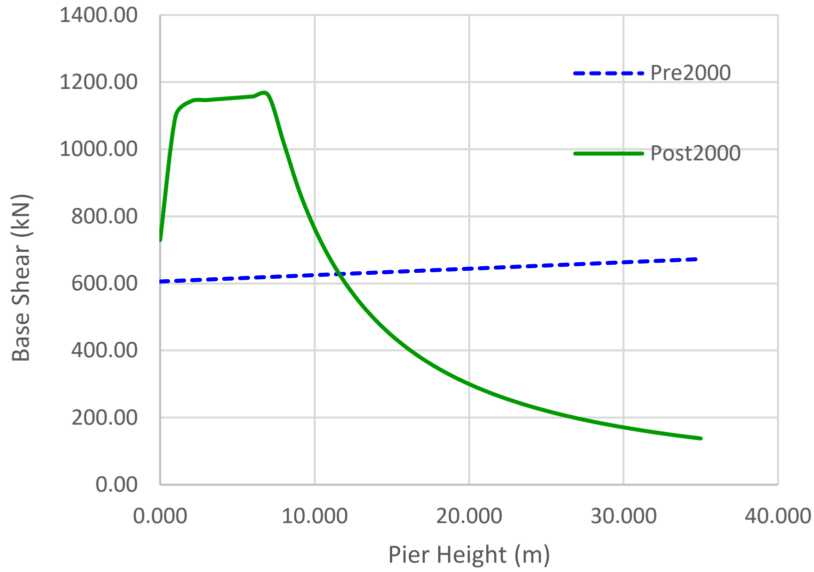


Figure 5.1: Variation of Base Shear with Pier Height

The displacement capacity of the short piers is clearly less than that of the longer piers. The displacement capacity of heavily reinforced columns is reduced as the longitudinal reinforcement ratio increases, and hence the force-based design approach will tend to reduce the displacement capacity.

5.1.2 Parametric study

The case studies/scenarios identified, and models discussed in the preceding sections were analysed for various parameters using nonlinear static analysis (pushover analysis) module available in SAP2000.

The parametric study is carried out by grouping/combining the model case studies into four (4) groups. The nomenclature used to designate bridge piers studied is described as follows.

The first character (i.e., ‘P’) indicates P-Δ analysis. The number character after (i.e., ‘1’ or ‘2’) indicates with or without respectively. The third character (i.e., ‘H’) indicates hinge properties. The number character after (i.e., ‘1’ or ‘2’) indicates P-M2-M3 or without M3 types respectively. The fourth character (i.e., ‘F’) indicates type of foundations for the piers. The number character after (i.e., ‘1’ or ‘2’) indicates fixed base or pile springs respectively. The summary is shown in **Table 5.1.**

Table 5.1: Grouping of the Models

Pier mkd.	P-Δ analysis		Hinge properties		Foundation	
	With	Without	P-M2-M3	M2	Fixed	Springs
P2-H1-F1		√	√		√	
P1-H2-F1	√			√	√	
P1-H1-F1	√		√		√	
P1-H1-F2	√		√			√

The graphical comparison of the above parameters of Pushover Analyses for different scenarios are presented in Error! Reference source not found. to **Figure 5.2.**

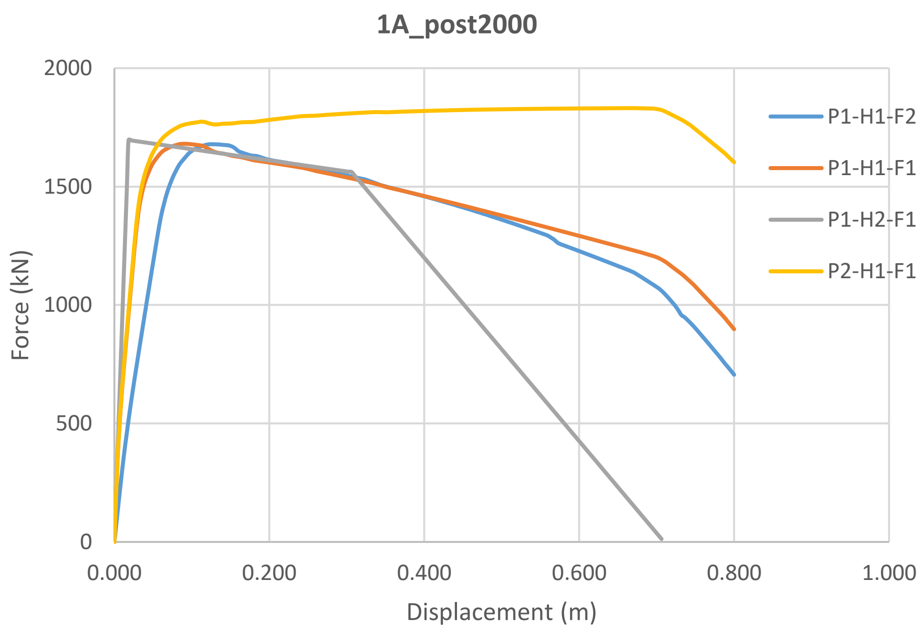


Figure 5.2: **Parametric Study of POA for 1A_post2000 Bridge Pier**

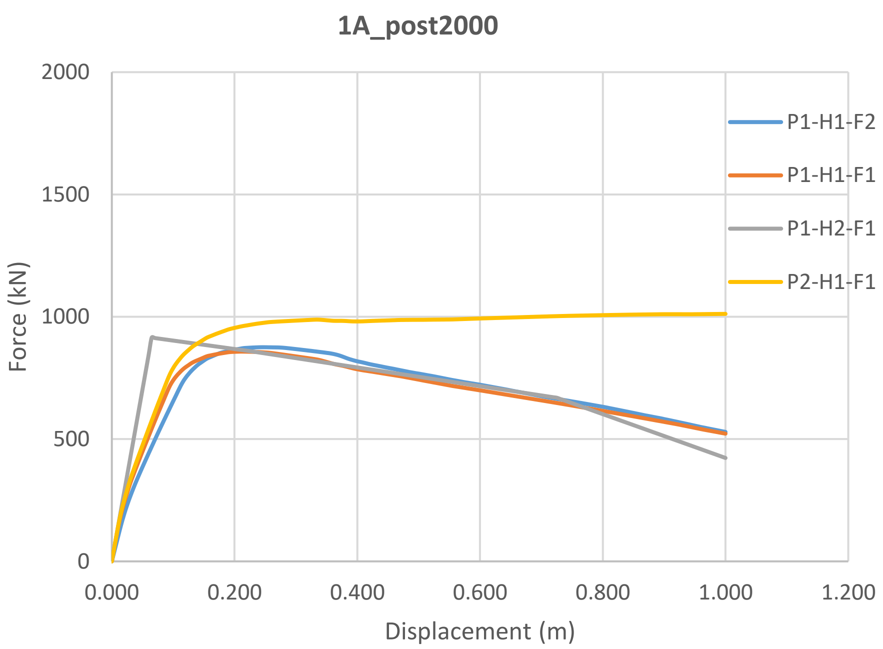


Figure 5.3: **Parametric Study of POA for 9A_post2000 Bridge Pier**

From the above graphs the following may be established:

(a) According to the AASHTO[2] (2011) code, P–Δ effects can be neglected in the analysis, if moment due to P–Δ effects is less than 0.25 times the plastic moment capacity of the pier. Similar provision exists in the CALTRANS[12] (2010) code, where P–Δ effect can be neglected if the moment due to P–Δ effects is less than 0.20 times the plastic moment capacity of the pier. In the present study, the influence of P–Δ effects on the estimated seismic performance of the considered bridge models was examined. For this purpose, the results of pushover analysis were compared, with and without considering P–Δ effects. From the pushover curves obtained it is observed that the P–Δ moment is 0.09 times the plastic moment capacity for short pier and 0.14 times the plastic moment capacity for long piers. However, the graphs of pier mkd. P2-H1-F1 and P1-H1-F1 show that the P–Δ effect results in significant post-yield softening and ignoring the P–Δ effect results in over-estimation of capacity.

(b) It may be well observed that the yield displacement of the distributed plasticity model (P-M-M hinge) and concentrated plasticity (P-M hinge) are similar. If properly defined, the concentrated plasticity model is capable of accounting for the effects of axial load-shear-bending moment interaction through some simplified rules based on plasticity theory. But this model require calibration under idealized assumptions about either force or deformation distribution within the member. ASCE-41[6] (2017) provides some empirical values to calculate moment, rotation at the failure points both for members under pure compression and flexure. However, the applicability of the same in the Indian scenario is quite questionable as the constituent material and reinforcement detailing vary considerably. The location of inelastic deformation needs to be specified a *priori* as well. Furthermore, plasticity theory-based interaction rule, in general, overestimates axial growth of a reinforced concrete member under cyclic loading resulting due to cracking and therefore increases artificially member's strength.

Distributed inelasticity models, on the other hand, do not suffer from the limitations of calibration and a priori specification of the location of inelastic deformation, but are computationally more demanding. In order to estimate the longitudinal responses like axial load and flexural moments about two orthogonal directions, each cross-section is discretized into a reasonable number of fibres. Each fibre is assigned to a uniaxial stress-strain response of the material associated with it. By integrating the material responses over all the fibres using midpoint rule, longitudinal responses are computed. This modelling strategy is known as *fibre section modelling*. Although the high computational cost of integrating a complex material response usually limits the selection to a few relatively simple hysteretic models, with the development of improved element formulation, efficient nonlinear solution algorithms and computing power, it is now easy to employ distributed inelasticity model even for a relatively complex spatial structure and therefore used for the present study.

From the graphs obtained above it is not possible to incorporate the effects of post-yield buckling, low cycle fatigue etc. in the cyclic response of reinforcement, in the distributed plasticity model, where engineering judgement is required to predict the failure points.

(c) Parametric study shows that the bridge becomes increasingly flexible with reduction of the modulus of subgrade reaction of the soil. Increase in the yield displacement of the bridge with SSI condition, as compared to the fixed base design condition, is around 35% for short pier and 17% for long pier. Under dynamic loading, the SSI becomes predominant. Thus, ignoring the effects of soil spring results in over-estimation of the capacity.

5.1.3 Pushover Analysis

The output is Pushover curve, which is the plot of pier Base Shear versus Displacement at Piercap top. Pushover curves for the four pier sections are shown in **Figure 5.4**.

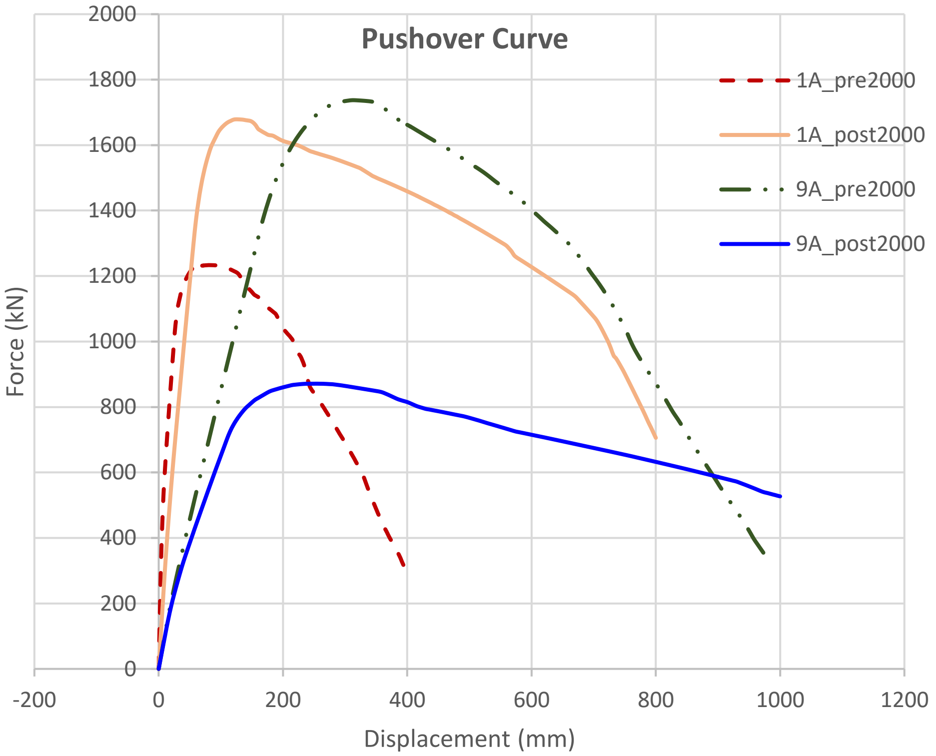


Figure 5.4: Pushover Curve for Bridge Piers

From the pushover analysis we get an estimate of the lateral strength of the structure. With the increase in magnitude of lateral loading, the progressive non-linear behaviour of various structural elements is captured, and weak links and failure modes of the structure are identified. It is evident that the short column in the pre2000 has minimum post yielding displacement capacity whereas the same pier designed as per latest design standard shows improved lateral capacity and ductility. On the other hand, the long column designed with the pre2000 code with the highest reinforcement percentage has the highest lateral strength, but the same column designed as per post-2000 code has the minimum lateral strength with high ductility post yielding.

5.1.4 Failure Modes of Column

The different failure modes as assessed with the strain limits mentioned in FHWA and ASCE guidelines for existing structures are presented in **Table 5.2**.

Table 5.2: Failure Modes for the Pier Columns

Pier	Failure Mode	Pushover step	Displacement (mm)	Force (kN)
1A-pre2000	Cover concrete spalling	68	136	1181
	Longitudinal bar buckling	75	150	1151
1A-post2000	Low fatigue cycle of longitudinal bar	83	332	1520
9A-pre2000	Cover concrete spalling	74	370	1699
	Longitudinal bar buckling	77	385	1678
9A-post2000	Strength reduction	130	650	695

The onset of spalling of the cover concrete at a concrete compressive strain limit ε_{cu} of 0.005 is a significant limit state, for the pre-2000 columns with minimal confinement. Exceedance of this limit state will result in extensive spalling that leads to a local condition that can be expected to require remedial action. Spalling is typically associated with onset of negative incremental stiffness and possibly sudden strength loss.

The unconfined concrete core will deteriorate considerably as soon as the longitudinal bars buckle. Under cyclic load, which is the actual condition during an earthquake, when the buckling strain (ε_b) reaches 0.005, the structures will no longer be able to carry further loads. This demarcates onset of permanent damage for the pre-2000 columns where confining rebars are not sufficient to restrain this buckling. The service will get disrupted and structure will require extensive repairs or may need to be replaced for the bridge to perform in the major aftershocks.

Due to the modern provisions for the design of post2000 bridge columns require high degree of confinement, the inelastic action in these regions can lead to low-cycle fatigue failure of the reinforcement. For the use of high-strength bars in post2000 pier columns, under cyclic-strain reversals, the peak cycle stress drops quickly due to softening, whereas for reinforcing bars, cycling causes hardening over the first few cycles, after which the peak cycle stress decreases very gradually almost constant over a large number of cycles until incipient failure occurs at the onset of a fatigue crack. Cycling can still continue, but the crack propagates quickly with the peak stress dropping rapidly until fracture occurs. Bar rupture due to low-cycle fatigue and the associated strength degradation is one of the common features of reinforcing steel observed during cyclic loading. Repeated loading and unloading reduces the strength of reinforcing steel in each cycle and eventually leads the material to reach its failure limit even if it never reaches the ultimate strength.

The shear capacities of pre-2000 columns, with nominal transverse reinforcement as recommended by IRC:21-2000 are much lesser than those of post2000. Solid circular piers with single hoops as transverse reinforcement showed the least shear capacity. Shear capacities of the pier columns are represented in **Figure 5.5.**

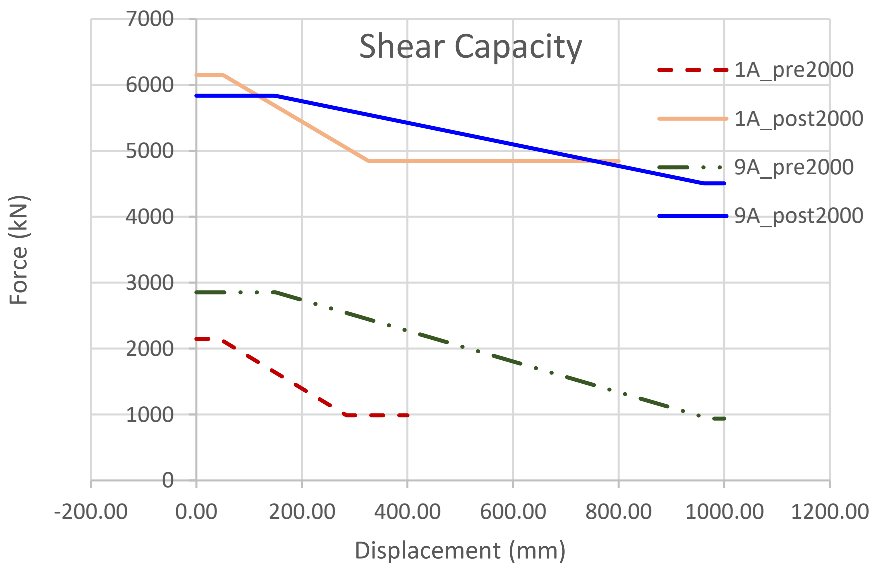


Figure 5.5: Shear Capacities of Bridge Piers

However, the first-mode response and the shear capacity of all the piers do not intersect, hence it is concluded that shear does not influence the displacement capacities of the piers considered for this study. **Figure 5.6** shows shear capacity v/s lateral strength of the piers analysed.

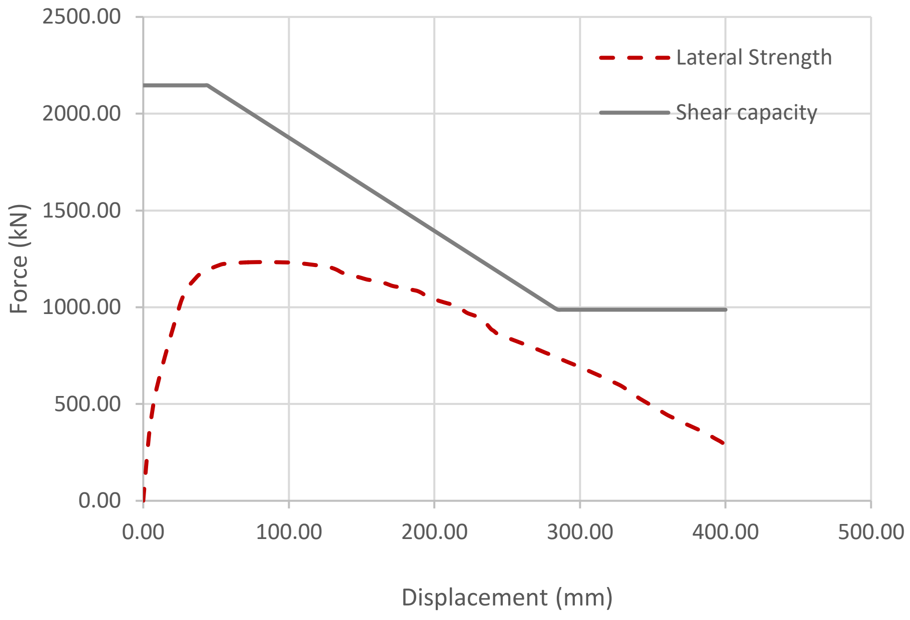


Figure 5.6 (a) Shear Capacity v/s Lateral Strength for 1A_pre2000 Bridge Pier

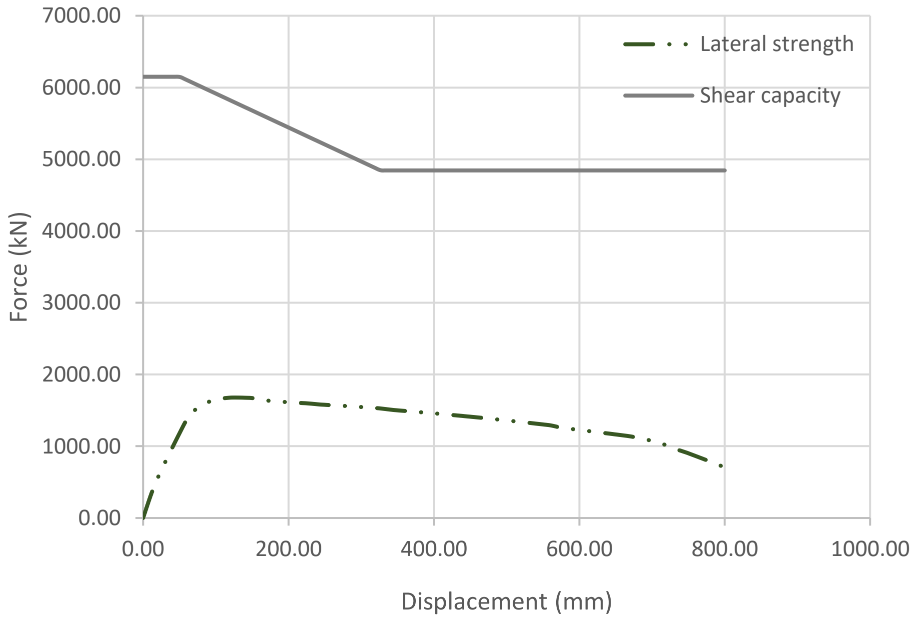


Figure 5.6 (b) Shear Capacity v/s Lateral Strength for 1A_post2000 Bridge Pier

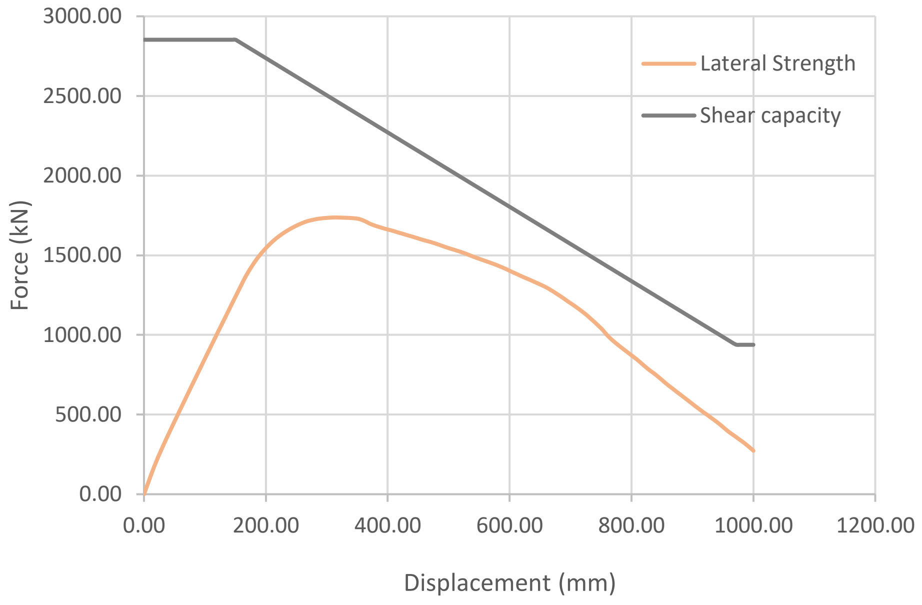


Figure 5.6 (c) Shear Capacity v/s Lateral Strength for 9A_pre2000 Bridge Pier

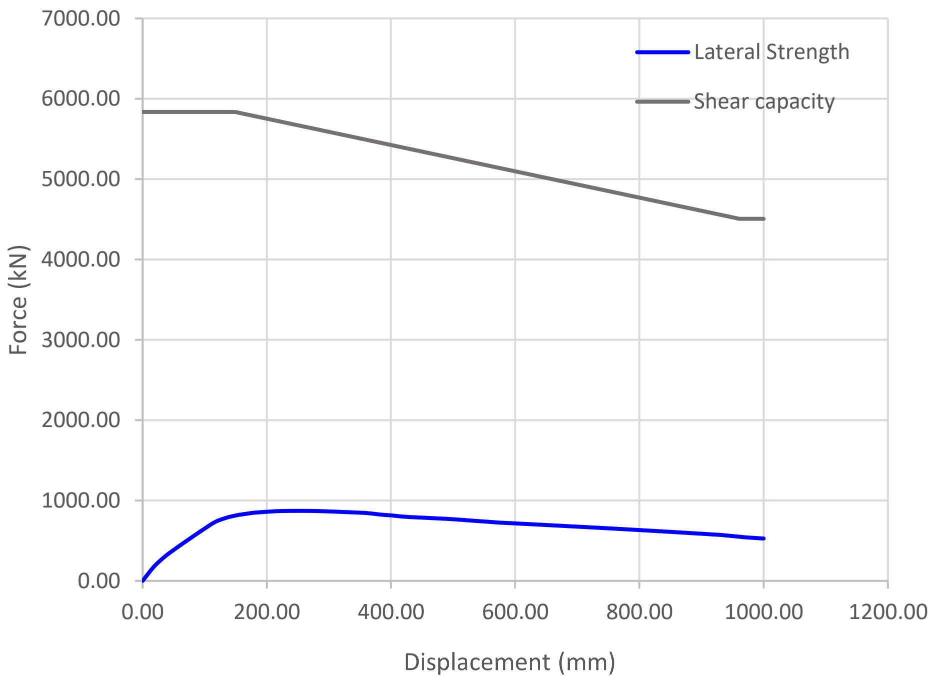


Figure 5.6 (d) Shear Capacity v/s Lateral Strength for 9A_post2000 Bridge Pier

5.1.5 Pier Assessment

ADRS plot has been derived for all the four types of pier assessed with MCSM. Both DBE and MCE level of seismic demand has been plotted and the capacity spectrum obtained from the pushover analyses of each of the pier types are plotted. It is worth mentioning that POA is an analytical tool, which is not capable of recording actual point beyond which the structure will no longer be able to withstand further load and undergo severe damage. It is from the limiting strain values as mentioned in various international standards (e.g. FHWA manual, CHBDC, ASCE-41) using engineering judgement, the failure points of each type of pier have been identified. Thus, the SAP output for POA has been considered to be valid up to the failure point beyond which the pushover curve obtained from POA have been neglected. Each of the results of assessment has been discussed in detail in the following paragraphs.

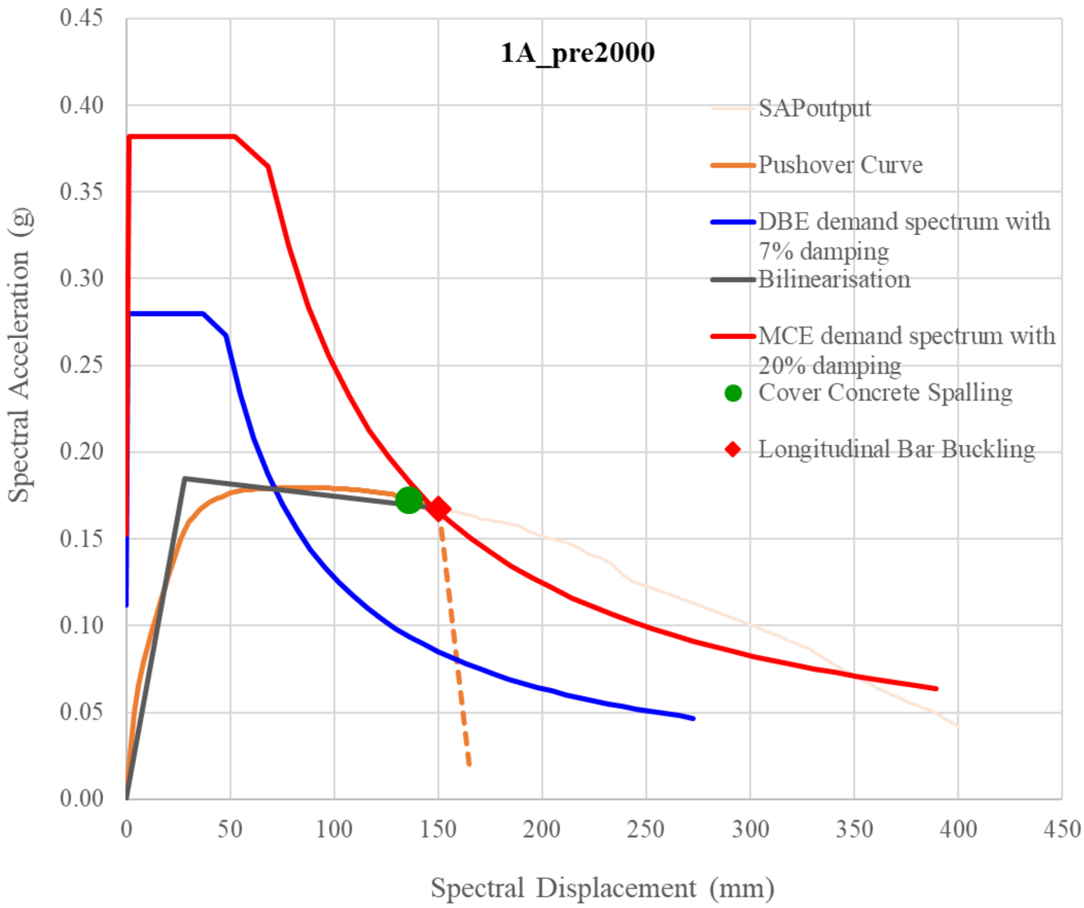


Figure 5.7: Performance Assessment for Bridge Pier 1A_pre2000

From the **Figure 5.7** it is clear that excessive damage occurs at Design Basis Earthquake in the pre-2000 short pier at about 150 mm deflection at pier top. Initial spalling of the concrete cover followed by buckling of the longitudinal reinforcement is expected to occur since the concrete core is unconfined. Once the cover concrete had completely spalled off, under cyclic load, the spiral and longitudinal reinforcement will be exposed, and strength degradation is predominant as bar buckles. The structure will no longer be able to resist earthquakes of large magnitude after first shock. Needless to say, it is unsustainable at Maximum credible earthquake.

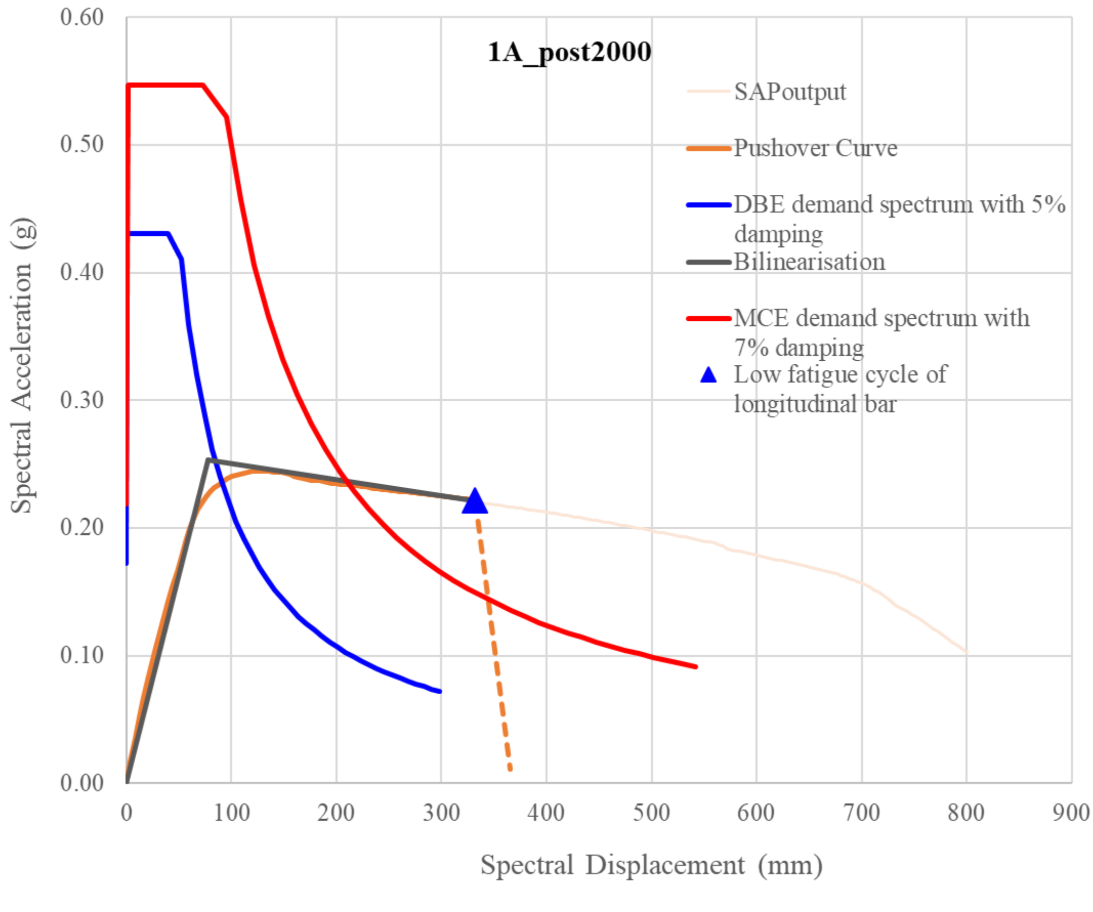


Figure 5.8: Performance Assessment for Bridge Pier 1A_post2000

Short pier column designed as per post 2000 codal provisions has high lateral strength, but less displacement capacity. Low fatigue failure of the longitudinal bars is predominant as shown in **Figure 5.8**. It may withstand up to MCE level of earthquake, however severe damage is expected to occur due to low fatigue cycle of the longitudinal bar that results in progressive strength reduction of the structure. The structure will undergo a maximum deflection of about 320mm at top, after which it will no longer be able to sustain gravity load.

In this regard, it is worth mentioning that seismic demand is highly unpredictable and cannot be quantified. Magnitude of the earthquake if exceeds the predicted seismic hazard, the structure may even fail before reaching MCE level of EQ. Influence of live load horizontal force like braking when added to this, will have severe effect.

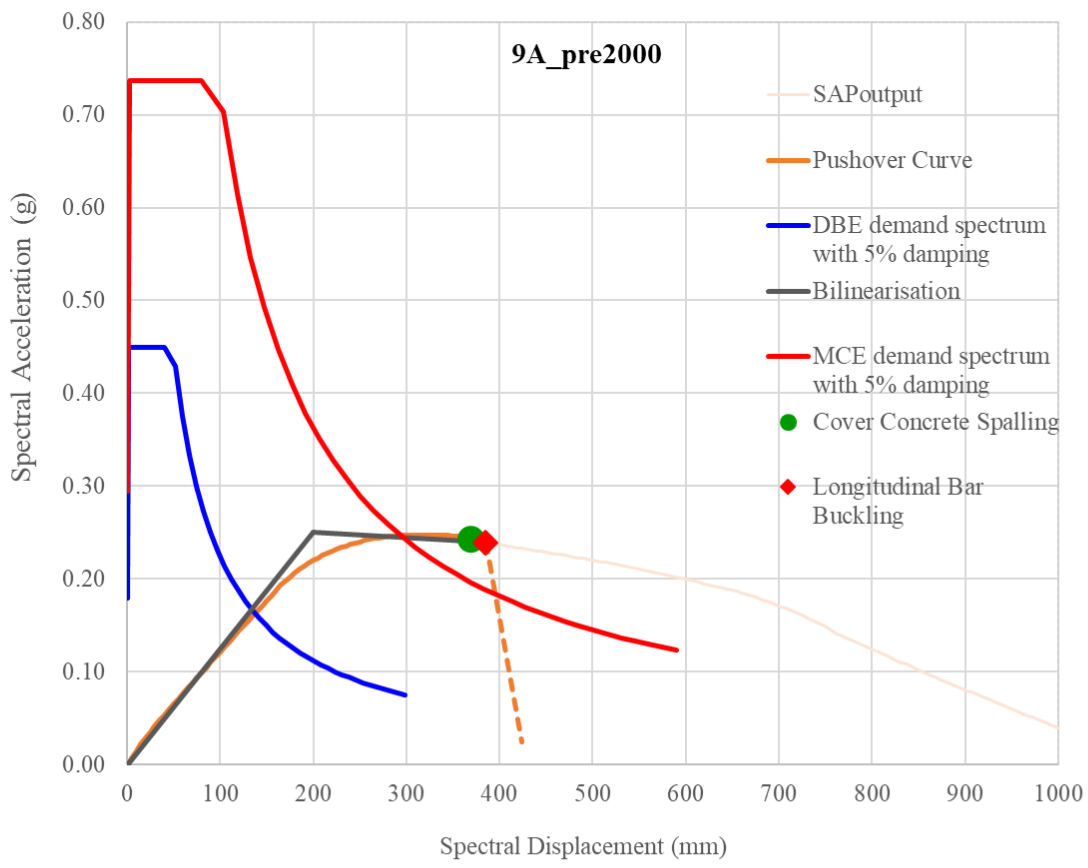


Figure 5.9: Performance Assessment for Bridge Pier 9A_pre2000

Due to presence of high reinforcement percentage in the long column designed as per the codal provisions of the pre-2000, the column has huge lateral strength. The pier behaves almost elastically up to the Design Basis earthquake after which it enters into its plastic zone. The strength degradation occurs with extensive spalling followed by longitudinal bar buckling at a top displacement of about 380mm as shown in **Figure**

5.9. Since P-Δ effect is predominant in the long columns, the degradation is rapid just after the pier experience MCE level of earthquake.

However, the provided reinforcement is on the higher side due to conservative seismic design as in pre-2000 codal provisions. The lateral load carrying capacity of the structure is high enough to resist up to Maximum Considered Earthquake after which severe damage is likely to occur.

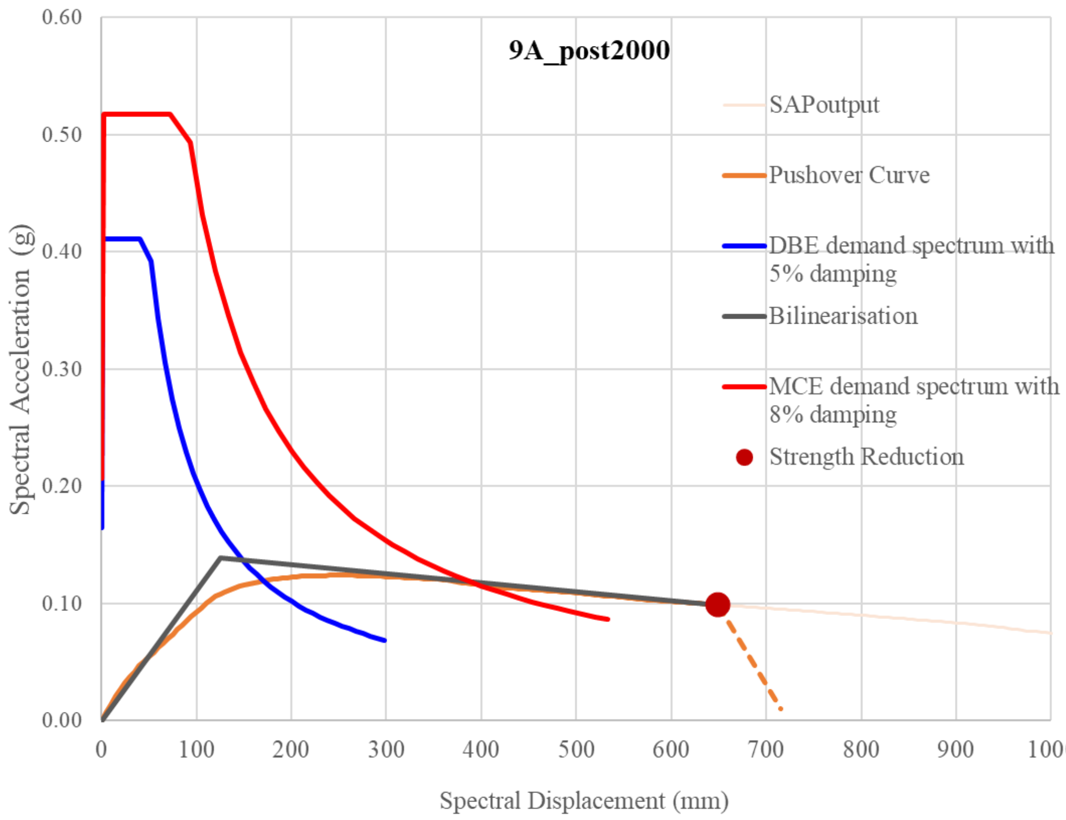


Figure 5.10: Performance Assessment for Bridge Pier 9A_post2000

The long column designed as per post2000 IRC code is found to be most economic. In spite of the least lateral strength, it is capable of undergoing highest displacement. Increasing the amount of transverse reinforcement, is found to enhance the displacement ductility of the pier and produce improved post-yield response.

As shown in **Figure 5.10** the bridge pier is capable of withstanding both DBE and MCE level of earthquake without any kind of major structural failure. However, at a displacement of around 650mm the strength of the pier reduces more than 20% of its maximum capacity, beyond which the structure will no longer be able to resist gravity load and the P-Delta effect will be predominant.

Thus, it is seen that inadequate transverse reinforcement causes crushing of concrete in the core of the cross-section on reaching the unconfined concrete strain and buckling of longitudinal steel, resulting in rapid strength degradation. Increasing the amount of transverse reinforcement, is found to enhance the displacement ductility and produce improved post-yield response as in post2000 piers. This signifies the importance of transverse reinforcement on the overall response of piers.

6.0 CONCLUSION:

6.1 Concluding Remarks

Presented herein are the outcomes of a seismic assessment study of typical reinforced concrete bridge piers in India. This study is limited to concrete bridge piers only. Based on the comparative study of the seismic design provisions laid down in the pre and post 2000 Indian Standards the following inferences may be drawn:

- The seismic design until late 2000's was largely based on the elastic behaviour of the structure and therefore, no special detailing was stipulated. The seismic load was estimated based on a seismic coefficient depending on the specified seismic zone only.
- There were two major limitations in this pre-2000 design procedure. Firstly, dynamic properties of the structure in the seismic load calculation were not considered. Secondly, only the elastic responses of the structure were considered. Past earthquakes showed that it is practically impossible to design a structure elastically under a severe seismic shaking.
- After the 2001 Bhuj earthquake, realising these limitations, the Indian aseismic design for bridges have adopted the current European practice. In the updated IRC 6, seismic loading is computed based on the dynamic properties of the structure, e.g. time period and energy dissipation characteristics during a shaking. It is also dependant on the design response spectrum for the specified seismic hazard in terms of peak ground acceleration (PGA).
- For the design part, IRC 21 was withdrawn, and a completely new IRC 112 was introduced in 2011. IRC 112 prescribes relevant seismic detailing where energy dissipation is likely to occur. This allows a structure to deform inelastically without any significant degradation in strength.
- Since most of the bridges were built following the previous seismic design recommendations, it is equally important to assess the seismic resilience of existing bridges to avoid any catastrophic closure of a route important from economic, cultural and societal perspective.

Attempt has been made to assess the seismic performance of existing bridge piers (both short and long) designed as per codal stipulation of pre and post 2000. Based on the numerical study on seismic assessment conducted and results discussed the following conclusion may be drawn:

- From the Pushover analysis it may be observed that the short column in the pre2000 has minimum post yielding displacement capacity whereas the same pier designed as per latest design standard shows improved lateral capacity and ductility.
- On the other hand, the long column designed with the pre2000 code is assumed to have the highest lateral strength, but the same column designed as per post-2000 code has the minimum lateral strength with high ductility post yielding.
- The likely failure mode for columns designed to pre-2000 standards seems to be excessive spalling of concrete followed by buckling of longitudinal bars.
- These columns also show a significant P-delta effect in the capacity curve and may lead to instability after a design level earthquake.
- The short pier designed in pre 2000's is unlikely to sustain earthquakes of large magnitude after first shock. Whereas, the long pier designed in pre 2000's with high percentage of reinforcement is expected to behave almost elastically at design level shaking after which it enters into its plastic zone. However, due to lack of confinement, excessive damage of concrete for higher seismic hazard level is expected to occur.
- Given the excessive damage in the pier columns in pre 2000's following a design level shaking, repair might not be economically feasible.
- Based on this study, it is recommended to focus more on the assessment of pre-2000 relatively stiffer short piers compared to the long piers, which are more vulnerable.
- The pier columns designed in accordance with the current standard (post 2000) are likely to perform better and is expected to withstand a shaking significantly larger than a design level shaking.
- Increase in the amount of transverse reinforcement in the piers designed as per post 2000 standards, is found to enhance the displacement ductility and produce improved post-yield response of piers.
- The likely failure mode for the post 2000 designed piers seems to be low cycle fatigue fracture of longitudinal bars for short pier, whereas strength degradation

of about 20% of the peak strength is expected to be the cause of long pier failure.

- Though the shear capacities of the pier designed with the earlier codal provisions are much lesser than those designed as per latest standards, the first-mode response and the shear capacity of all the piers do not intersect. Thus, shear seems not to influence the displacement capacities of the piers considered for this study.
- It is expected that the post 2000 designed piers would suffer nominal damage under a design level shaking and can be reinstated relatively quickly.

Thus, there is a need for seismic assessment of bridges in a more holistic approach to ensure a better seismic resiliency in the major highway routes in India.

6.2 Future Scope of work

This thesis work has explored performance based seismic assessment of RCC bridge piers, however the fields covered are so extensive that some issues are left for future developments in both theories and applications.

- The performance level of a structure for different seismic hazard levels, quantification of damage, development of more reliable analytical procedures including non-linear time history analysis are areas which need further research in the present scenario.
- The empirical formulae used for the assessment purpose like the damping parameters, the modifiers used for the modified capacity spectrum method are solely dependent on the experiments performed with the available material and prevailing environmental conditions in foreign country. The derivation and applicability of the same to the Indian scenario needs to be identified and verified. in our country.
- Dynamic analyses like linear time history analysis or a response spectrum analysis may be carried out due to its ability to consider the effect of higher modes of vibration. The application of elastic dynamic analysis is adored to provide the true answer for response prediction under very damaging ground motions with the present study of NSP is incapable of.

- This report focused only reinforced concrete pier column of a very long viaduct, where isolation of a single pier served the purpose of seismic assessment of the bridge system as a whole. Study may be extended for other bridge forms whose seismic behaviour is completely like small to medium length bridge, bridges with widely varying pier heights, and curved spans, skew bridges with bent columns.
- Extensive works to predict the appropriate characterisation of future site-specific probable ground motion at a specific site need to be done as the success of earthquake assessment of the structure hinges depends on this. Furthermore, the response spectra provided in the Indian standards are for a maximum period of 4secs. However, there are cases where the period of the structure is more. Extensive study on this could extend the response spectrum up to a time period of 10secs or more.
- Experimental work to verify whether the predictions from the model correspond well with the experimental data and to validate the considered analytical model involving modification if any.

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APPENDIX A:

Interaction Curves for the Pier Column designed in post 2000's

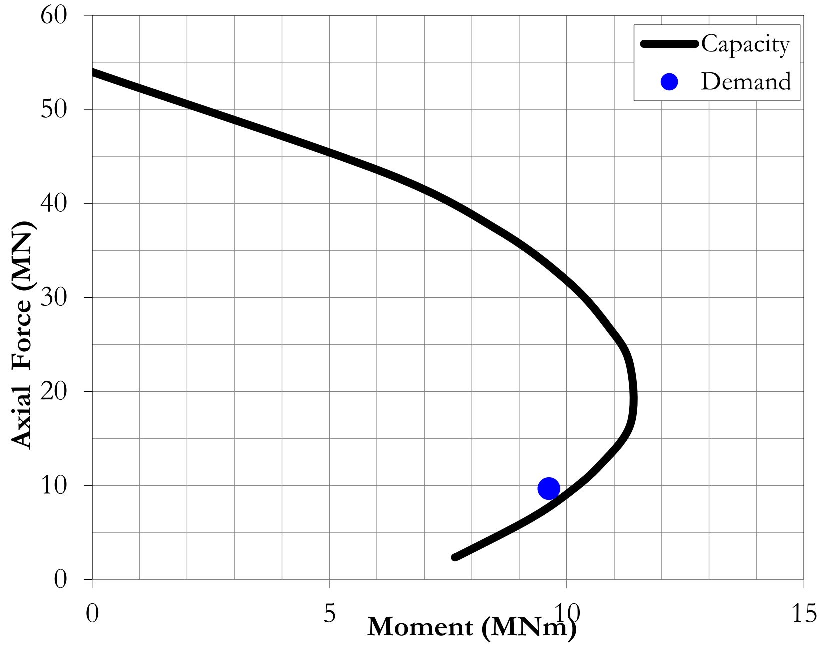


Figure A.1 Interaction Curve for 1A_post2000 Bridge Pier

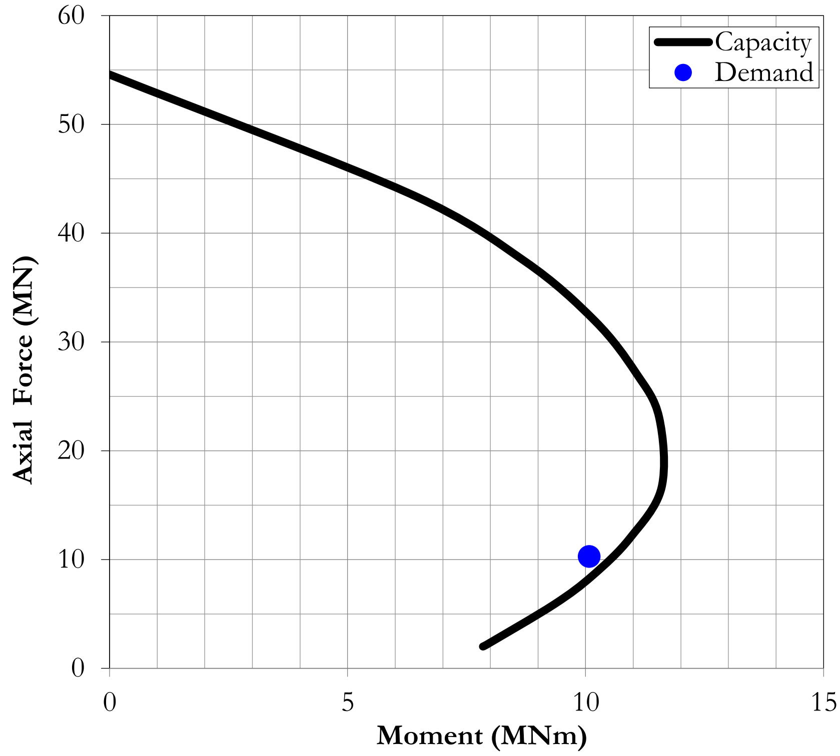


Figure A.2 Interaction Curve for 9A_post2000 Bridge Pier

APPENDIX B:

Mander-Confined Concrete model for the Pier Columns

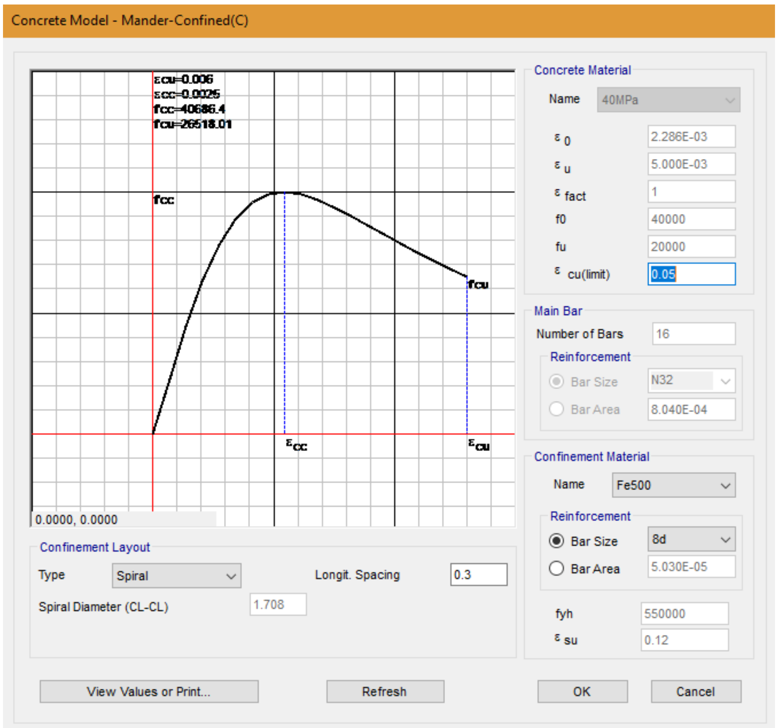


Figure B.1 Mander-Confined concrete model for 1A_pre2000 Bridge Pier

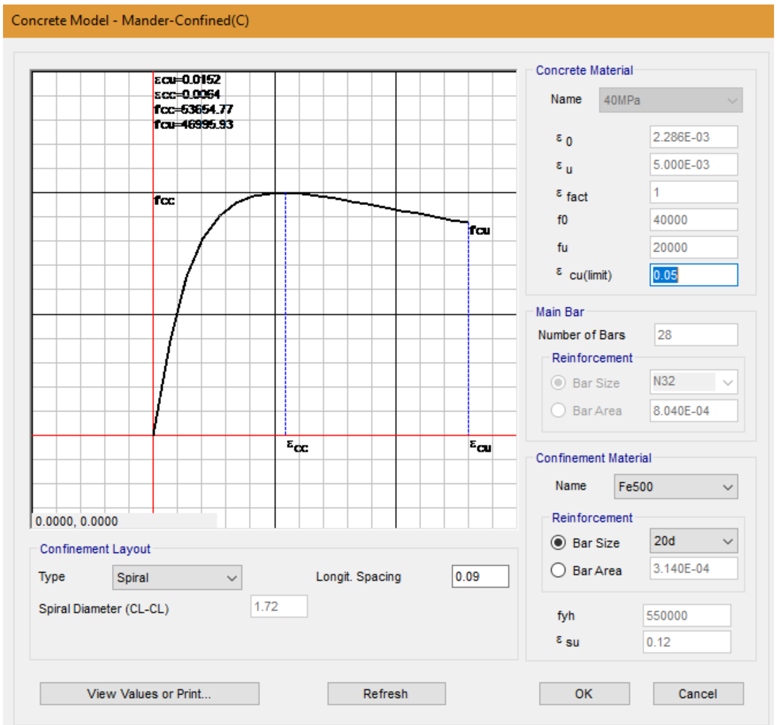


Figure B.2 Mander-Confined concrete model for 1A_post2000 Bridge Pier

APPENDIX B:

Mander-Confined Concrete model for the Pier Columns

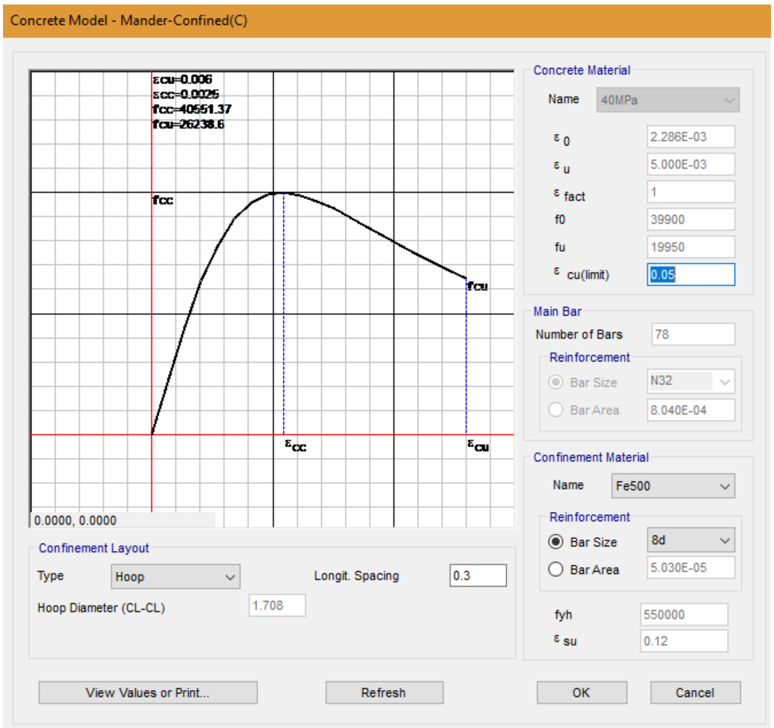


Figure B.3 Mander-Confined concrete model for 9A_pre2000 Bridge Pier

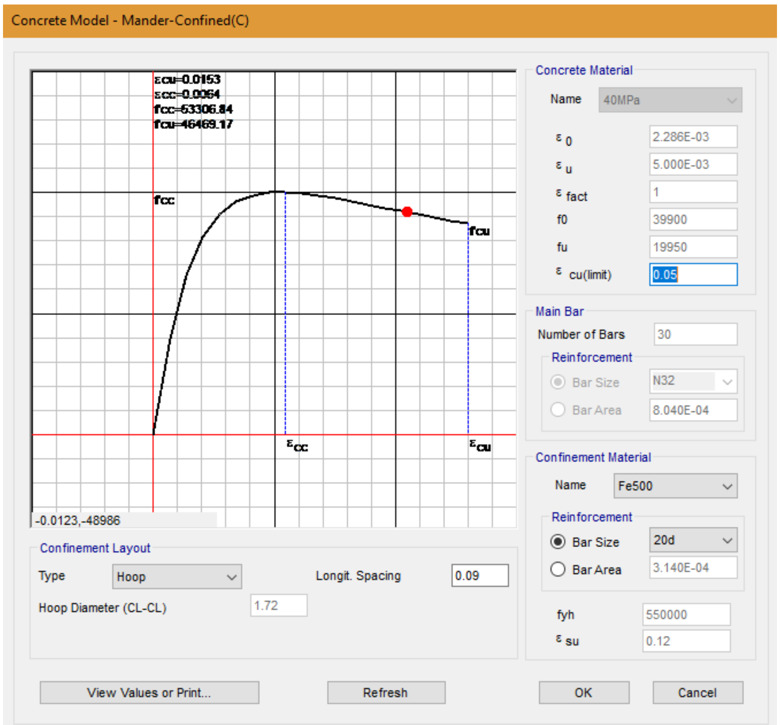


Figure B.4 Mander-Confined concrete model for 9A_post2000 Bridge Pier

APPENDIX C:

Moment-curvature graph for the Pier Columns

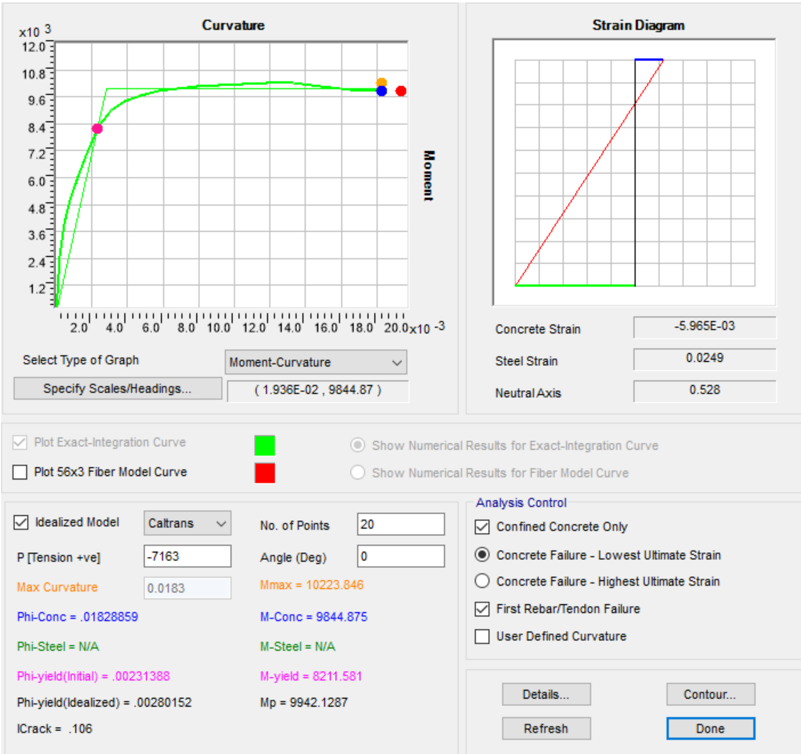


Figure C.1 Moment-curvature for 1A_pre2000 Bridge Pier

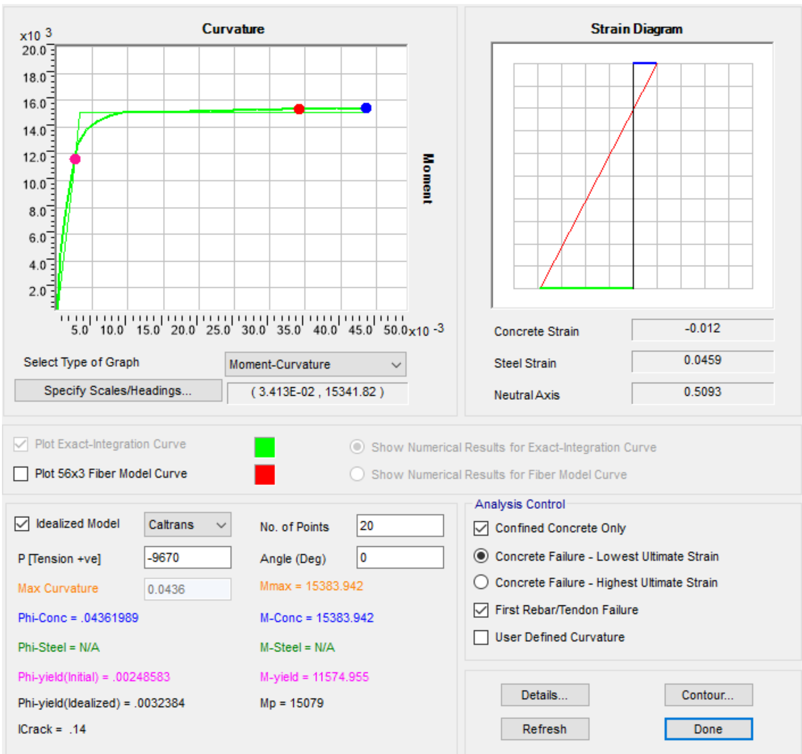


Figure C.2 Moment-curvature for 1A_post2000 Bridge Pier

APPENDIX C:

Moment-curvature graph for the Pier Columns

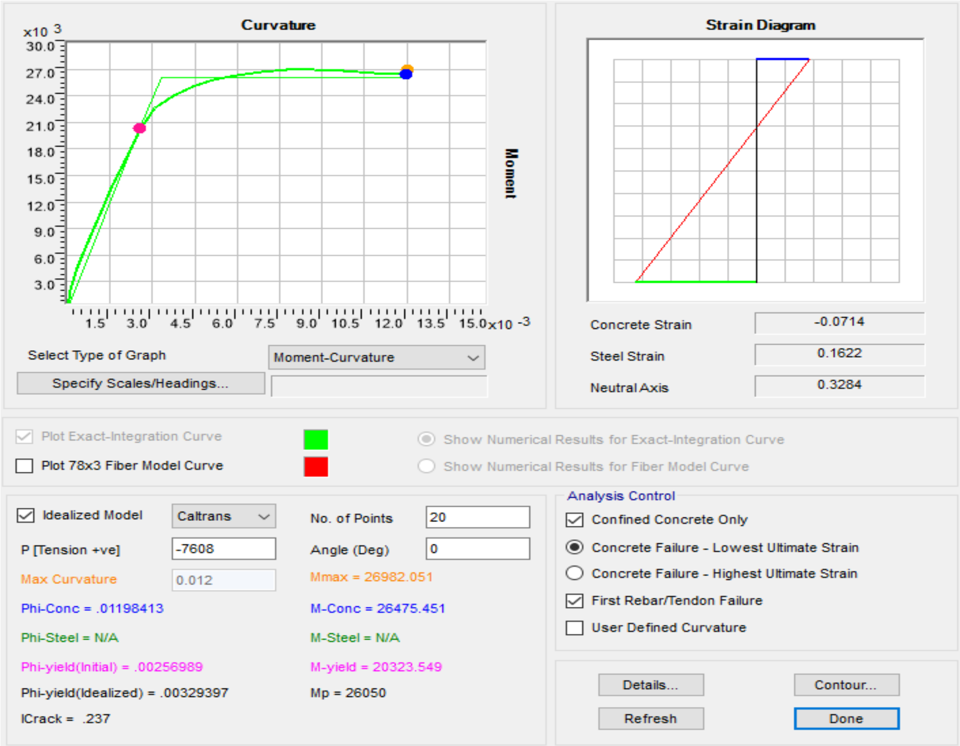


Figure C.3 Moment-curvature for 9A_pre2000 Bridge Pier

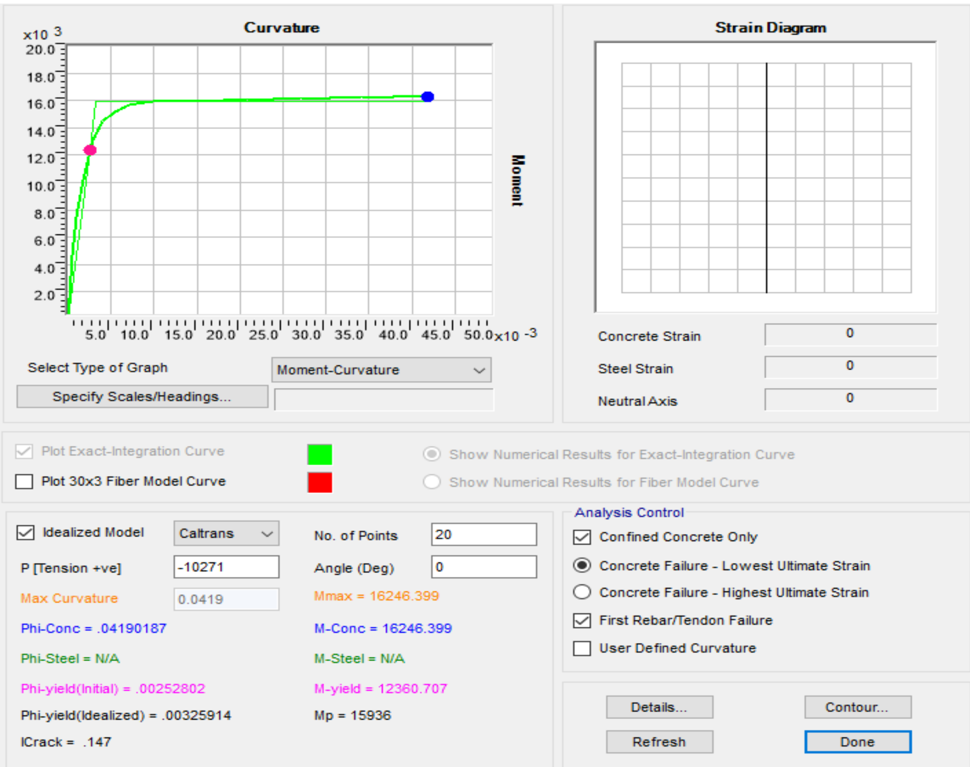
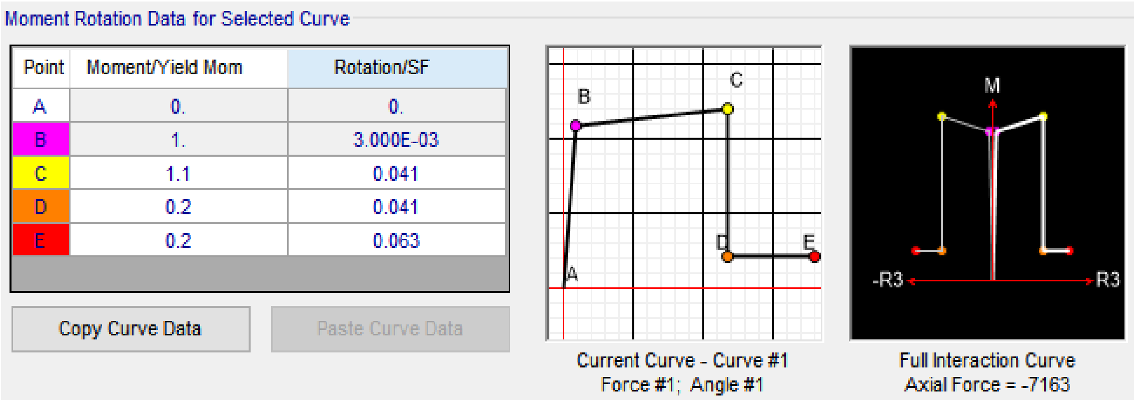


Figure C.4 Moment-curvature for 9A_post2000 Bridge Pier

APPENDIX D:

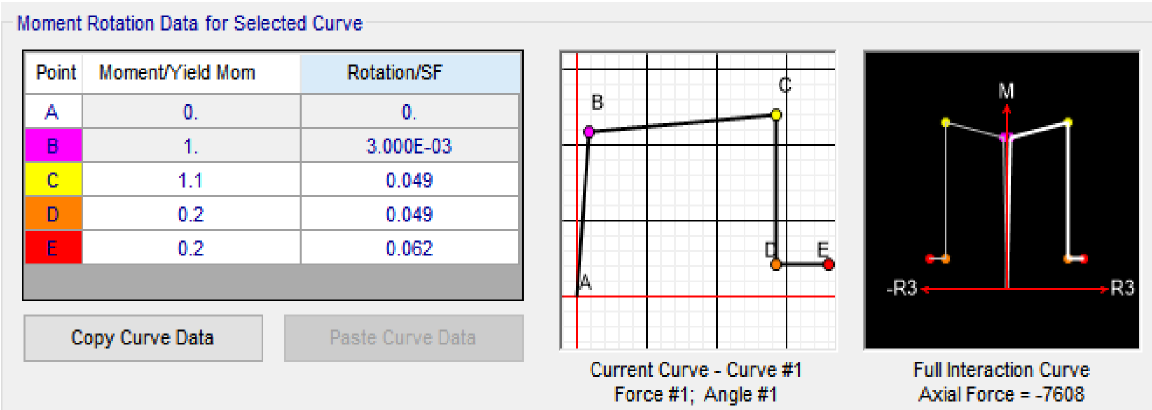
M3 Plastic Hinge Definition for Pushover Analysis of the Pier Columns



Moment Rotation Data for Selected Curve

Point	Moment/Yield Mom	Rotation/SF
A	0.	0.
B	1.	3.000E-03
C	1.1	0.041
D	0.2	0.041
E	0.2	0.063

Figure D.1 P-M3 Hinge properties for 1A_post2000 Bridge Pier



Moment Rotation Data for Selected Curve

Point	Moment/Yield Mom	Rotation/SF
A	0.	0.
B	1.	3.000E-03
C	1.1	0.049
D	0.2	0.049
E	0.2	0.062

Figure D.2 P-M3 Hinge properties for 9A_post2000 Bridge Pier

APPENDIX E:

Parametric Study for the Pier Columns

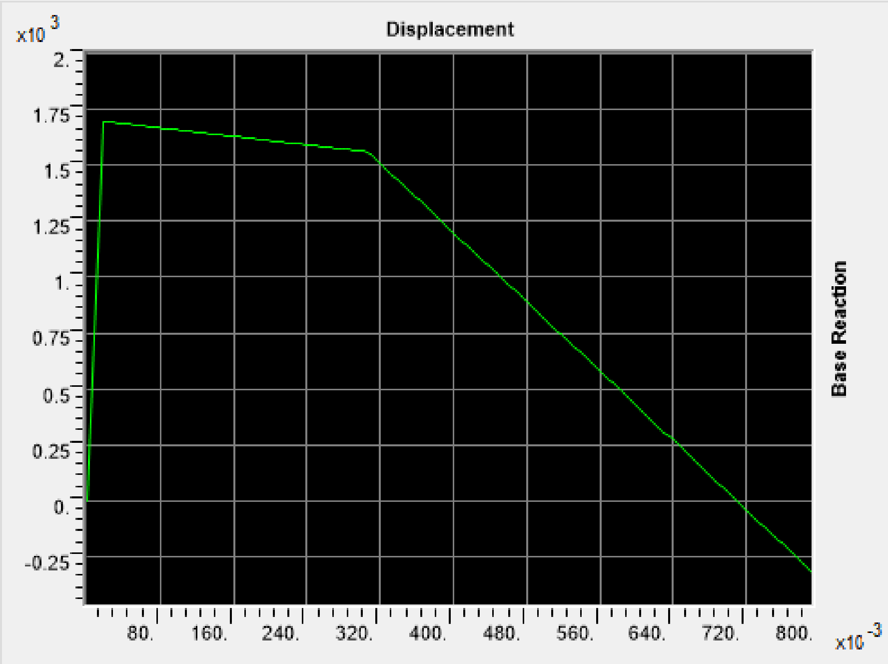


Figure E.1 Pushover Curve for (P1-H2-F1) 1A_post2000 Bridge Pier

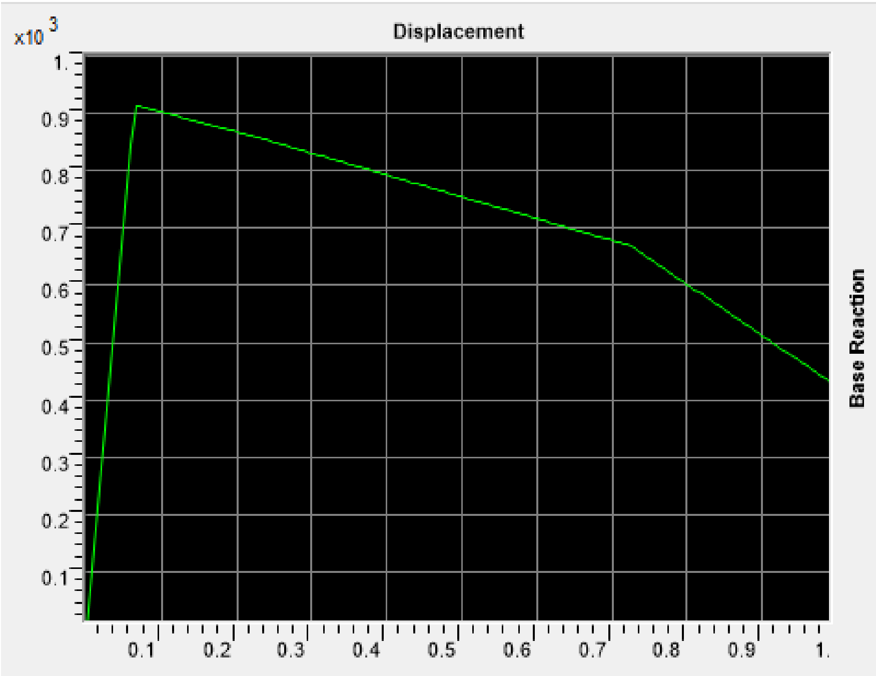


Figure E.2 Pushover Curve for (P1-H2-F1) 9A_post2000 Bridge Pier

APPENDIX E:

Parametric Study for the Pier Columns

Figure E.3 Pushover Curve for (P1-H1-F1) 1A_post2000 Bridge Pier

Figure E.4 Pushover Curve for (P1-H1-F1) 9A_post2000 Bridge Pier

APPENDIX E:

Parametric Study for the Pier Columns

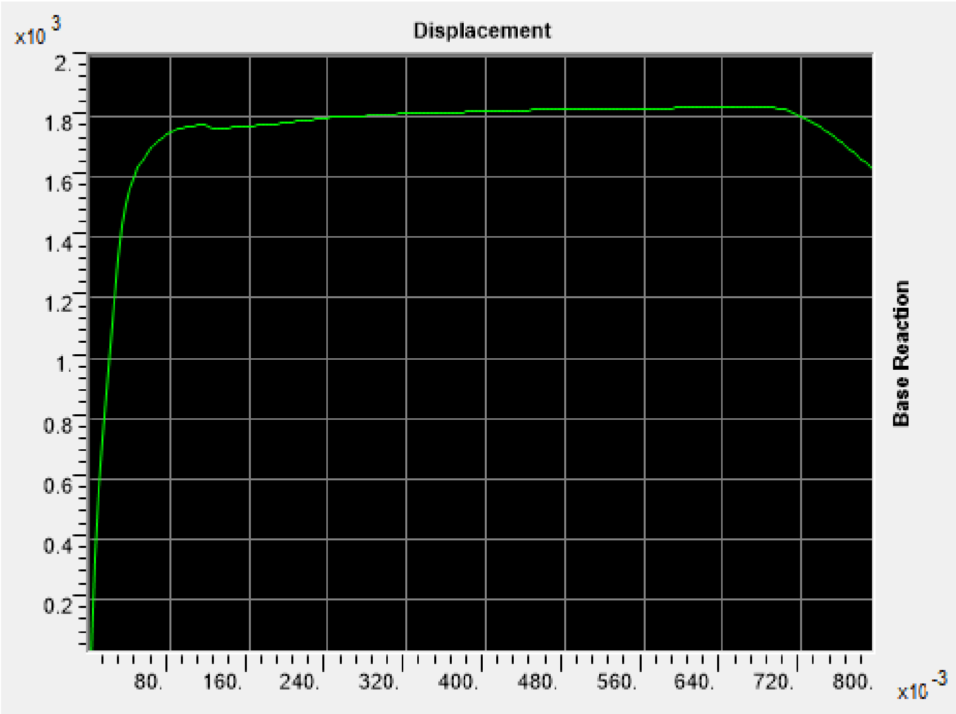


Figure E.5 Pushover Curve for (P2-H1-F1) 1A_post2000 Bridge Pier

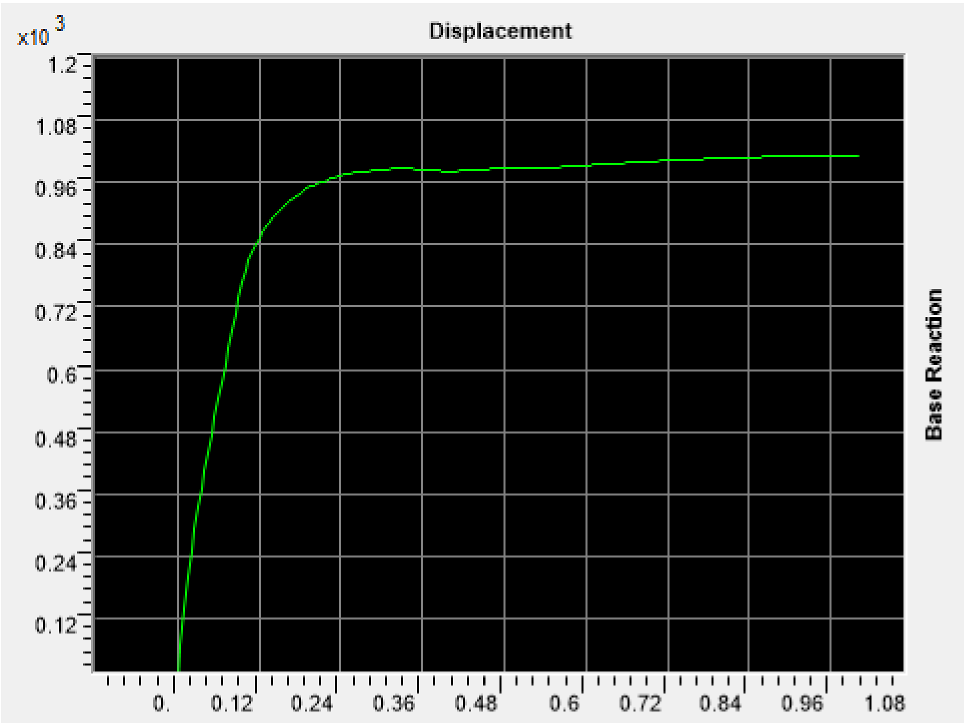


Figure E.6 Pushover Curve for (P2-H1-F1) 9A_post2000 Bridge Pier

APPENDIX F:

Pushover Curve of the Pier Columns

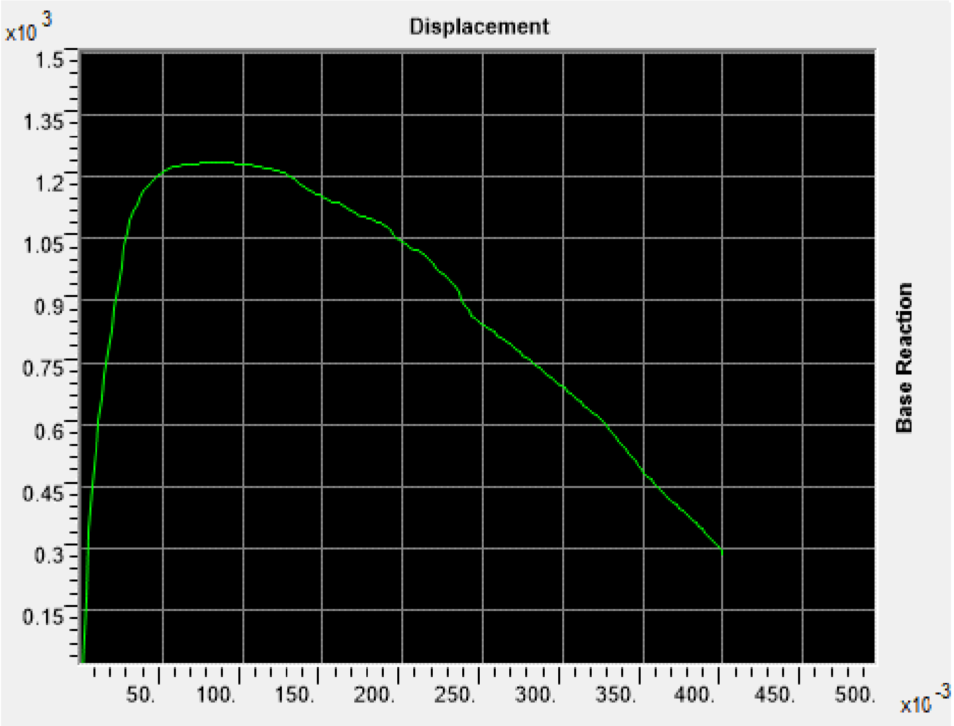


Figure F.1 Pushover Curve for 1A_pre2000 Bridge Pier

Figure F.2 Pushover Curve for 1A_post2000 Bridge Pier

APPENDIX F:

Pushover Curve of the Pier Columns

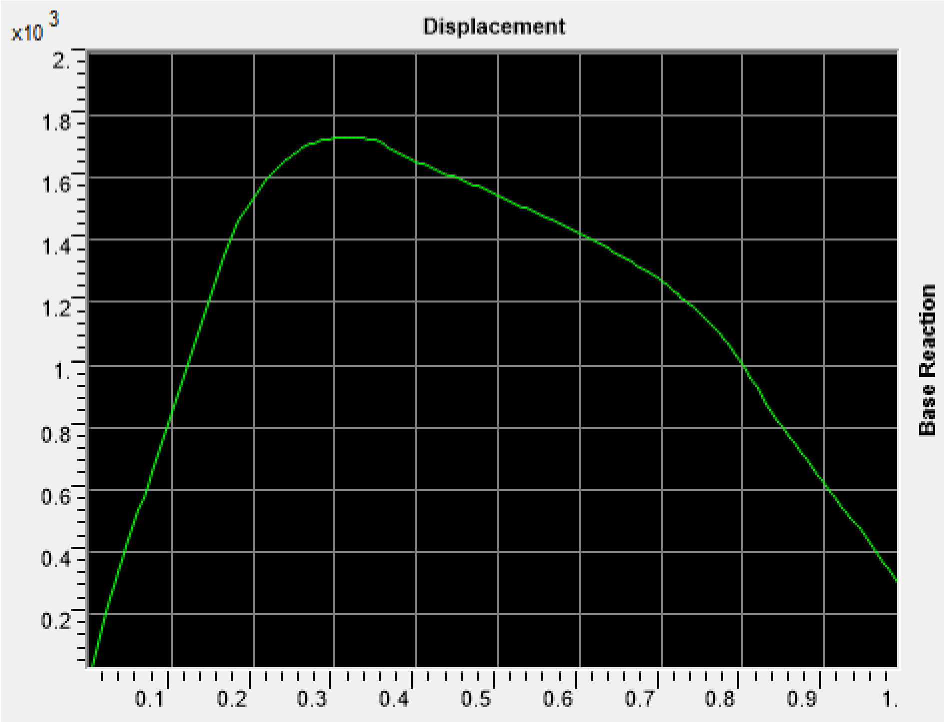


Figure F.3 Pushover Curve for 9A_pre2000 Bridge Pier

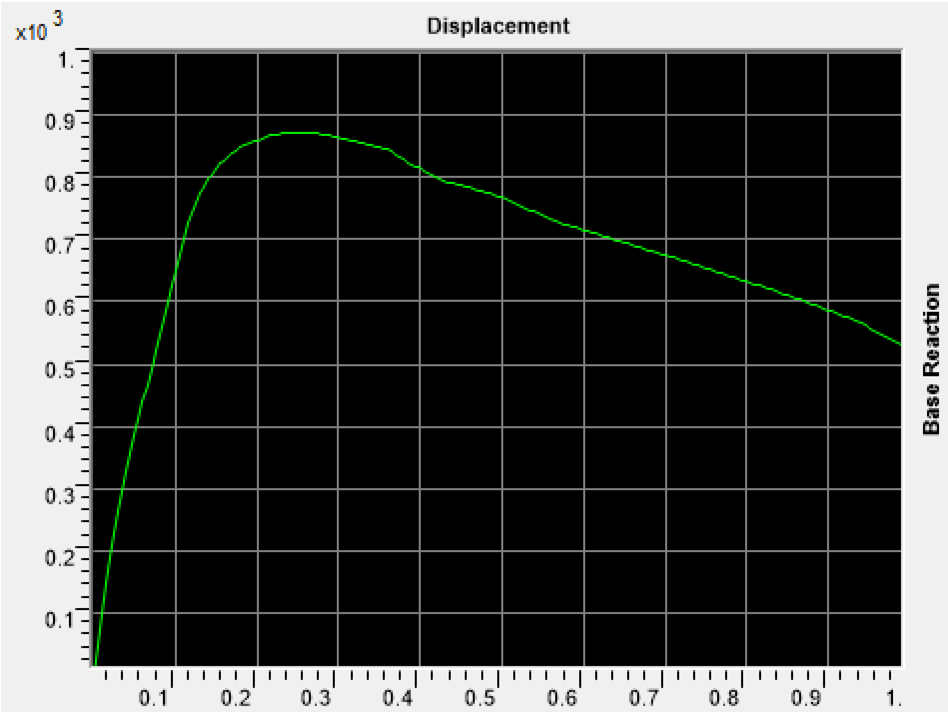


Figure F.4 Pushover Curve for 9A_post2000 Bridge Pier

APPENDIX G:

Hinge Results of the Pier Columns

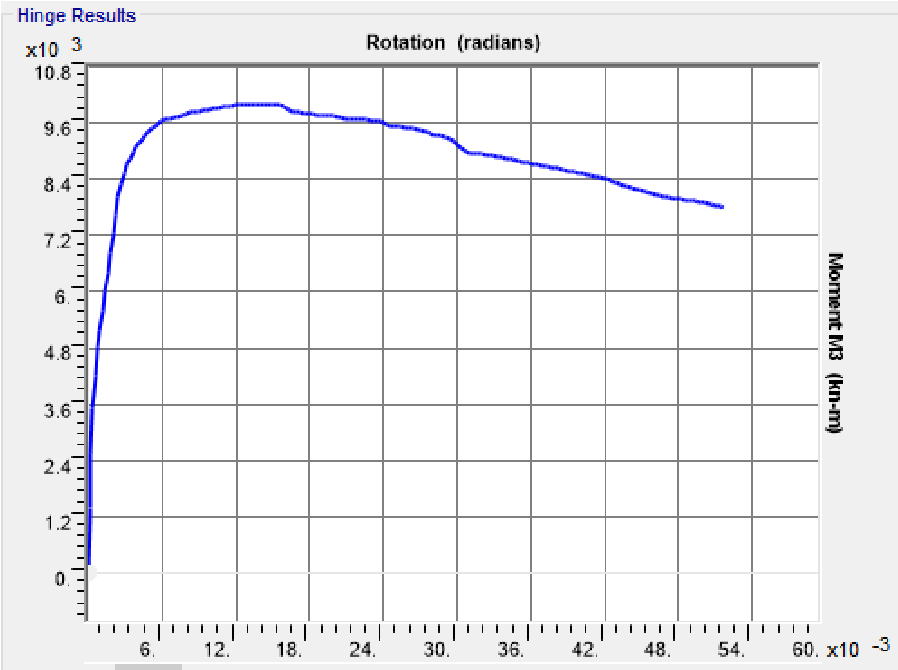


Figure G.1 Hinge Results for 1A_pre2000 Bridge Pier

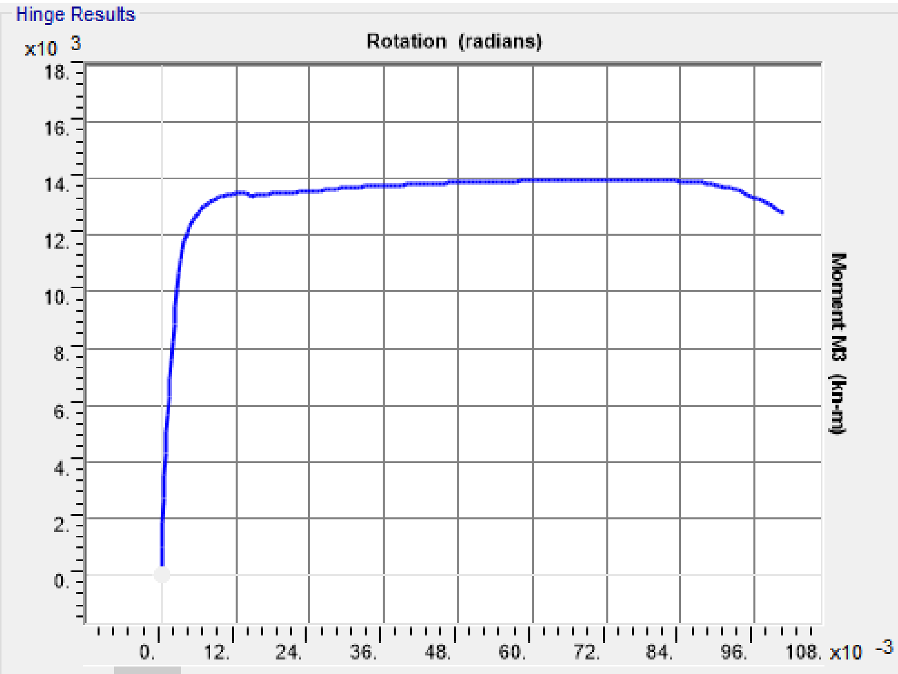


Figure G.2 Hinge Results for 1A_post2000 Bridge Pier

APPENDIX G:

Hinge Results of the Pier Columns

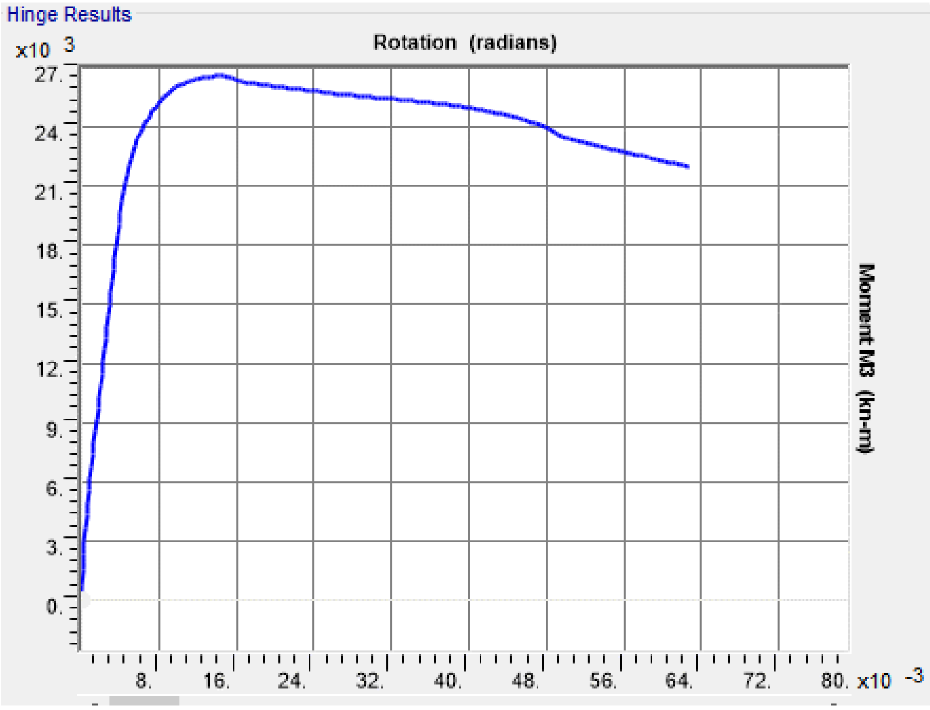


Figure G.3 Hinge Results for 9A_pre2000 Bridge Pier

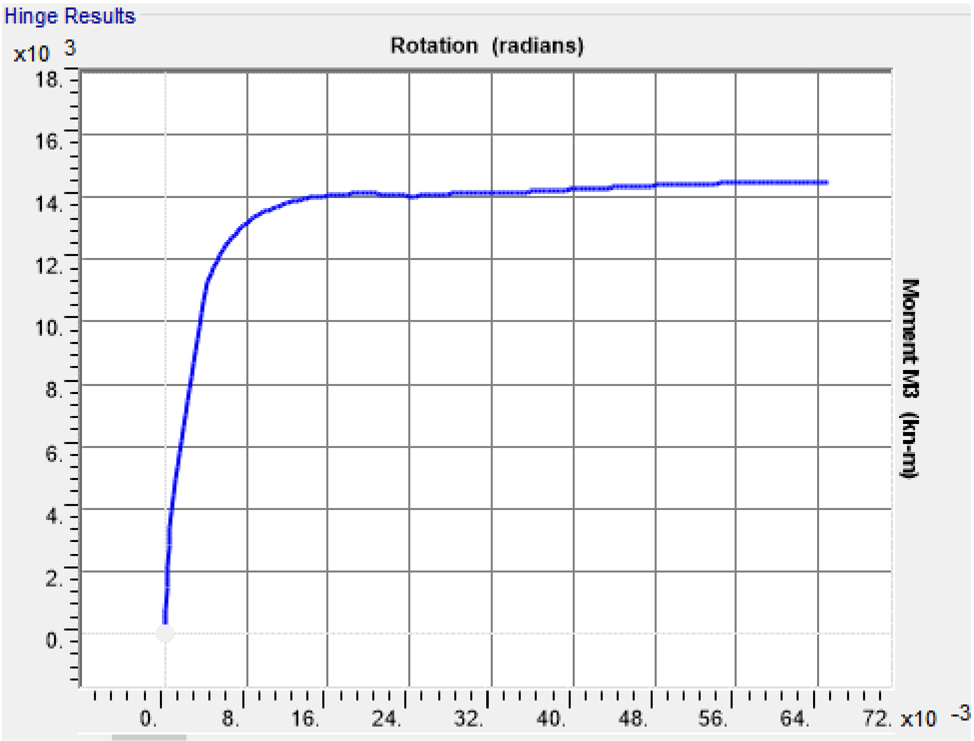


Figure G.4 Hinge Results for 9A_post2000 Bridge Pier