STRUCTURAL PERFORMANCE OF SOME EXISTING BRIDGES OF BANGLADESH UNDER EARTHQUAKE LOAD

by

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A thesis submitted to the Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, in partial fulfillment of the requirements for the degree

of

MASTER OF SCIENCE IN CIVIL ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING
BANGLADESH UNIVERSITY OF ENGINEERING AND TECHNOLOGY, DHAKA, BANGLADESH

November, 2009

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Declaration

I hereby certify that the research work reported in this thesis has been performed by me and that this work has not been submitted elsewhere for any other purpose, except for publication.

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ACKNOWLEDGEMENT

The author wishes to express his deepest gratitude to Dr. Ahsanul Kabir, Professor, Department of Civil Engineering, BUET, for his continuous guidance, invaluable suggestions and affectionate encouragement at every stage of this study.

The author wishes to express his profound gratitude to Engr. Sabbir Siddique, Director, Design Planning & Management Consultants Ltd. for his continuous support and encouragement to complete the thesis work.

The author wishes to express his sincere gratitude to all of his colleagues who shared their workload to help complete this study.

The author wishes to express his sincere gratitude to Engr. Ehshan Hafiz and Engr. Ashik Hossain who helped me to collect some references.

ABSTRACTS

Seismic design provisions of AASHTO codes are usually used for designing bridges in Bangladesh. Both dynamic and equivalent static methods of analysis are recommended in AASHTO code. In Bangladesh, the static analysis is mainly followed. A structure may behave either elastically or plastically under earthquake load depending on its intensity. However, the structure is allowed to behave nonlinearly under the design level of earthquake but the performance of the structures incorporating this nonlinearity are not evaluated during the static design procedure. It is, therefore, necessary to have a clear understanding of structural response under earthquake force following a method popularly known as performance evaluation .

Elasto-plastic response of three important existing bridges under seismic load has been evaluated in this study. Before carrying out the task, existing literatures on relevant field is thoroughly reviewed. Elasto-plastic analysis has been conducted following performance based design method. The performance level of the structure under different load intensities are determined. Here, a performance level means the damage level, which can be assessed and judged.

Three bridges from three different seismic zones of Bangladesh are selected for study. Non-linear static pushover analysis are performed for these structures to evaluate its performance under seismic load. The analysis has been carried out following guide lines of Applied Technology Council (ATC) and Federal Emergency Management Agency (FEMA). Capacity Spectrum method has been employed to determine the performance levels. One of the study bridge located near Sylhet town meet Life Safety Performance Level in its longitudinal direction and Damage Control Performance Level in transverse direction. At this performance level, severe damage of the bridge without collapse could be expected under design level of earthquake. The other two bridges located near Barisal town and Dhaka city meet Immediate Occupancy Performance Level. In the Immediate Occupancy Performance Level, only minor repairable cracks may develop.

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ABBREVIATIONS

ACI	American Concrete Institute
ADRS	Acceleration Displacement Response Spectrum
ATC	Applied Technology Council
BNBC	Bangladesh National Building Code
C	Conforming transverse reinforcement
CP	Collapse prevention performance
EPA	Effective peak acceleration
FEMA	Federal Emergency Management Agency
IO	Immediate occupancy performance
LS	Life safety performance
NC	Non-conforming transverse reinforcement
RSA	Response Spectrum Analysis
SAP	Structural Analysis Program of CSI
UBC	Uniform Building Code

NOTATIONS

ω_n	Natural frequency at n^{th} mode
ζ	Critical damping
$P_{eff}(t)$	Effective earthquake force at time instant t
ω'	Radial frequency of the effective first mode
a	Acceleration due to gravity
A(t)	Pseudo acceleration
A'_s	Compression Steel area
A_g	Gross concrete area
A_s	Tensile Steel area
b_w	Width of beam stem
c	Damping coefficient
C_A	Seismic coefficient
Ct	Numerical coefficient
C_V	Seismic coefficient
d	Lateral displacement
f'_c	28 days cylinder strength of concrete
f_D	Force due to damping
f_I	Force due inertia
F_n	Lateral force at level n
f_S	Inertia force
$f_{s(t)}$	Force at time instant t
F_t	Concentrated force on roof top for accommodating higher mode
F_x	Lateral force at level x
f_y	Yield strength of steel
g	Acceleration due to gravity
h_n	Height at level n
h_x	Height at level x
Hz	Unit for frequency
I	Second moment of Inertia
k	Stiffness of a system

m	Mass of a system
M_3	Moment about major axis
$M_{b(t)}$	Moment at base at time instant t
N_A	Near source coefficient for seismic source
N_V	Near source coefficient for seismic source
P	Axial force
$P_{(t)}$	Force at time instant t
P_c	Axial force contributed by concrete
PF_1	Modal participation factor for the first mode
P_i	Total gravity load at level i
Py	Axial force up to yield
Q	Lateral load
Q_y	Lateral load up to yield level
R	Response modification factor
S_a	Spectral acceleration
S_{ai}	Spectral acceleration at time instant i
S_d	Spectral displacement
S_{di}	Spectral displacement at time instant i
T	Time period
T'	Effective time period
T_A	Coefficient
T_S	Coefficient
u	Displacement
u(t)	Displacement at time instant t
u_g	Displacement due to ground acceleration
u_t	Total displacement
V	Base shear
$V_{b(t)}$	Shear force at base at time instant t
V_i	Total calculated share force at level i
W	Seismic dead weight
Z	Zone coefficient
$\dot{U}$	Velocity

ΔT	Time increment
Δ_y	Yield displacement
$\Phi_{1, Roof}$	Roof level amplitude for the first mode
α_1	Modal mass coefficient for the first mode
ϕ_{i1}	Amplitude of mode 1 at level i
ρ	Steel ratio
ρ'	Compression steel ratio
ρ_{bal}	Balanced steel ratio

Chapter 1

INTRODUCTION

1.1 General

Road communication is becoming very popular means of transportation system in the world particularly in the developing countries like Bangladesh. Bridges are an integral part of roads or railways. It reduces the travel time significantly. Consequently its contribution to the national economy is great. Also its construction is very difficult and costly. Being a riverine country, huge numbers of bridges over rivers and khals are to be constructed involving huge investment of national fund. So its design should be optimized to sustain all anticipated loadings during its life span without loosing its serviceability. Codes describe all such loadings and the methods of its application. But it adopted a different philosophy in case of seismic loading because of its uncertainty both in its occurrence and magnitude. Thus design of bridge for seismic loading is non-comprehensive because actual structural response under seismic load is not adequately addressed in the AASHTO code procedure for seismic design. Investigation of failure strategy during earthquake depict that formation of mechanism at particular locations of lateral load resisting members is responsible for collapse of the structure. So the structural capacity needs to be evaluated upto plastic range for earthquake resistant design. In 1971 an organization called Applied Technological Council (ATC) was established to assist structural engineers to solve some unsettled issues of codes. They are publishing reports on respective issues. Among their publications, Report ATC 6 and Report ATC 40 launched a revised philosophy for seismic design. Now the structures are designed for seismic load following this method of analysis. This procedure of design is known as Performance Based Design. The first step of this method of design is to identify the locations of yielding of the structure and the final step is to design retrofit considering global impact of mechanism. Performance of bridges particularly the major bridges under seismic load should checked to investigate its functionality during a design earthquake. Some works on this issue for building structures in Dhaka city have been carried out. But no works for bridges have yet been done even at the academic level. So this research is an attempt to initiate this work on bridges of Bangladesh.

In general bridges give the impression of being rather simple structural systems. In fact, it has always occupied a special place in the affection of structural engineers. The reason behind this proposition is its structural form which is simple to carry out its functional requirement. As such, structural solutions can often be developed that are both functionally efficient and aesthetically satisfying.

Bridges, in particular those constructed in reinforced or prestressed concrete, have not performed as well as might be expected under seismic attack. In recent earthquakes, modern bridges designed for seismic resistance collapsed or were severely damaged when subjected to ground shaking of an intensity lower than current code intensities. This unexpectedly poor performance, as reported from damage investigation, is mainly attributed to the design philosophy coupled with the lack of attention to design details.

Bridges are typically more sensitive to soil-structure interaction effects than other structures. Dynamic response to ground shaking may be less predictable, particularly for long bridges, as a consequence of nonsynchronous seismic input at different supports, resulting from traveling-wave effects. Further, bridges are founded on saturated soil, depending on its type, which may have a potentiality for liquefaction. Also large relative displacements of supports could occur due to fault dislocation during an earthquake.

In the recent times, Bangladesh was hit by the earthquake several times whose magnitude was measured as high as 6.0 [Munaz and Hiroyuki, (2007)] on Richter scale. So this become a growing concerns that the earthquakes of higher magnitude are likely to happen in this region in near future. Thus the capacity levels of the existing bridges need to be investigated to assess the expected damage level of the existing bridges.

1.2 Objectives

The primary aim of this research is to evaluate the seismic response of some existing bridges at different earthquake zones of Bangladesh. To this end the following objective has been set:

(i) To perform Non-Linear Static Analysis (Elasto-Plastic/Pushover Analysis) of three existing bridges of Bangladesh designed as per AASHTO code provisions.

(ii) To study performances of the selected bridges to assess the level of seismic performance objective met by them.

1.3 Methodology

Response of structure under seismic loading is dynamic. So this behavior should be solved by the theory of structural dynamics. But this dynamic problem can also be solved by an equivalent static analysis to avoid analysis complexity and precision needed to perform dynamic analysis. The difference of ultimate result is too small to consider for simple structure like bridge. The work volume is confined within this equivalent static analysis.

Elastic procedures for structural analysis is sufficient as long as the loading pattern is ensured. But in case of uncertain loading like seismic load, ultimate load carrying capacity needs to be evaluated for economic reason. In the evaluation of the structural response beyond the elastic limits, linear analyses method can neither predict the failure mechanisms nor account for redistribution of forces during progressive yielding. This fact makes the elastic procedures insufficient to assess and evaluate seismic response of the bridges in particular and the structures in general. Nonlinear procedures of analysis can overcome this problem and is able to show the performance level of the structures under any loading level.

Structural analyses for seismic load lies in the assumption that during earthquake the structure will be allowed to deform inelastically upto certain limit and the structure will sustain the load of higher magnitude than elastic limit. But most of the codes do not clearly explain how the capacities of a deformed structure could be evaluated. So it remain uncertain how much damage will occur if the structure be struck by the expected seismic force if it is designed as per the guide lines of the present codes.

This limitation of code provision lets the code committees to think further for improving the code guidelines to evaluate real response of a structure when it is hit by the actual seismic force. Based on these guidelines, seismic design procedure follows performance-based design method. A number of tools are now available for a more rational analysis of the structures under seismic loading [ATC-40, (1996); FEMA-356, (2000)]. One of the main advantages of performance-based designs is its ability to show the performance situation of the structure and its components under different load intensities. The performance situation means that the damage level, if any, can be assessed and a judgment can be made as to which degree this structure can continue in service.

Performance based design needs to know the result upto nonlinear analysis of the structures. Various methods like (1) the Capacity Spectrum Method (CSM) [ATC, (1996)]; (2) the Displacement Coefficient Method (DCM) [FEMA-273, (1997)]; and (3) Modal Pushover Analysis [Chopra and Goel (2001)] are being practiced. However, these methods were developed to apply the Non-Linear Static Procedure (NSP) for buildings only.

Bridge researchers and engineers are currently investigating similar concepts and procedures to develop simplified procedures for performance-based seismic evaluation of bridges [Barron, (2000); Dutta, (1999); Shinozuka et al. (2000)]. Few studies were performed to apply the Non-Linear Pushover (NSP) analysis for bridges. In those studies, the CSM was implemented to estimate the demand. CSM is also used in this study.

In order to achieve the selected objectives, the research has been initiated by studying seismic provisions of current codes of practices in Bangladesh and available literatures on Performance Based Design of bridges. Three existing bridges in the three seismic zones have been selected and their detail engineering drawings have been collected. These are Shah Paran Bridge over the river Surma, Meghna Bridge over the river Meghna and Doarika Bridge at Doarika near Barisal Town. The elasto-plastic analysis of these bridges are done upto a particular level of deformation to evaluate Capacity Spectrum of the bridge under seismic loading. This type of elasto-plastic analysis is now termed as nonlinear static pushover analysis. The analysis demonstrates progressive damage of elements by inserting appropriate hinges as the structure is laterally pushed through. Geometric non-linearity (P-Δ effect) is included in the analysis. The resulting capacity curves (Base shear vs. deck/pier deflection) are to be superimposed on earthquake demand curves in the same domain. The point of intersection of these curves represents structure's performance level. A guideline for retrofit design on the basis of this performance level is mentioned in ATC or FEMA Specifications. So necessary retrofit may be identified for each structure as per the demand of anticipated earthquake.

The general purpose finite element program SAP 2000 Nonlinear program has been used for modeling the structures and study its behavior in terms of capacity and performance.

1.4 Layout of the thesis

The general background, objectives of the study and methodology of the work have been presented in Chapter 1 to give basic idea of the work being done under the research. In Chapter 2, literature review and response of a bridge during an earthquake has been described. In Chapter 3, concept of seismic performance evaluation of structure and non-linear pushover analysis procedure have been discussed. Chapter 4 described the basic modeling and analysis parameters used in the study. Chapter 5 described the modeling parameters for the bridges under study. Chapter 6 contains analysis and result while comments on analysis result is presented in Chapter 7. Conclusion derived from the present study and recommendation for future work has been presented in Chapter 8

Chapter 2

LITERATURE REVIEW AND RESPONSE OF A BRIDGE DURING EARTHQUAKE

2.1 Introduction

A large number of bridges were designed and constructed at a time when bridge codes had no seismic design provisions, or when these provisions were insufficient according to current standards. From the global report [Rafik and Xin (2003)], it has been observed that most of the bridges so far damaged in earthquakes were constructed before 1971 and had little or no design consideration to seismic resistance was incorporated. The vulnerability of pre-1971 bridges was especially evident in the 1989 Loma Prieta earthquake, the 1994 Northridge earthquake, the 2001 Nisqually earthquake, and 2001 Bhuj earthquake [Siddique (2006)]. Since the 1971 San Fernando earthquake in California, the standards for earthquake design have been strengthened considerably, and bridge structural behavior has been more accurately evaluated. Since then, structural ductility, a crucial element for the survival of bridges under severe earthquakes, has become a key consideration in structural analysis and design. In addition, performance based design has been implemented, thus seismic performance of bridges, once designed by the methods outlined in the Standard Codes of practice, can be evaluated. Incorporating this new concept of design, seismic performance of any bridge can greatly be improved by providing ductility requirement without increasing the cost of re-construction [Rafik and Xin (2003)]. This chapter summarizes the outcome of literature review and fundamentals of structural dynamics with analytical approach adopted by the different codes of practices.

2.2 Damage Strategy

Review of the literature [Rafik and Xin (2003)]; Fu and AlAyed (2004); Priestley et al. (1996); ATC-6-2(1983); Agarwal et al. (2008)] reveals that, there are three basic design deficiencies responsible for bridge damage caused by recent earthquakes. All these three are the direct consequence of the elastic design philosophy adopted for seismic design. Elastic seismic design of the older bridges typically utilised comparatively low allowable stress level corresponding to a seismic force. But this seismic force is just a small fraction of actual force that would be created in a structure. The actual strength can not meet the demand even for moderate seismic excitation although its value is typically 100 to 300% higher than the elastic design force levels as considered.

The consequences of this elastic design approach as found in the investigation are:

Deflection due to seismic loading were underestimated due to the use of gross section stiffness in the computation of displacement. In a column-beam frame, lateral load resisting element is the column. The gross section stiffness of column can not be equal to its cracked section even within elastic range. The degradation of column stiffness due to cracking of section was not considered in estimating deflection.

Since seismic force levels were arbitrarily made low, the ratio of gravity load to seismic load adopted for design was not correct. This led to inappropriate estimation and distribution of moment under combined gravity load and seismic force. Points of contraflexure were thus seriously dislocated. This resulted in improper termination of reinforcement [Priestley et al.(1996)].

Inelastic structural actions and associated concepts of ductility and capacity design are crucial to the survival of inelastic systems. Under severe seismic response, the structure is assumed to behave inelastically. Such behavior of the structure was not considered in the elastic design process. Thus locations of potential hinge formation in the inelastic range were not detailed to sustain strength degradation in the inelastic range. Also member shear strength was not set higher than flexural strength, to avoid the possibility of brittle shear failure [Priestley et al. (1996)].

Earthquakes have a habit of identifying structural weaknesses and concentrating damage at these locations [Zwicky and Bianchi (2007); Priestley et al. (1996); Chopra (2002)]. With building structures, the consequences may not necessarily be disastrous, because of the high degree of redundancy generally inherent in structural systems. This enables alternative load paths to be mobilized if necessary. But, bridges have little or no redundancy in the structural systems, and failure of one structural element or connection between elements is thus more likely to result in collapse than is the case with other structure. This leads to the above-mentioned warning that the structural simplicity of bridges may be a mixed blessing. While the simplicity should lead to greater confidence in the prediction of seismic response, this also results in greater sensitivity to design errors [Priestley et al. (1996)].

The dynamic response of the bridge to earthquake ground motion is the most important cause of earthquake-induced damage to bridge. Failure of the ground and soil beneath bridge foundation is also a cause of damage [ATC 6-2 (1983)].

2.3 Complexity of Earthquake Ground Motion

Real earthquake ground motion at a particular bridge site is vastly more complicated than the simple wave form of motion. The complexity of earthquake ground motion is due to three factors: 1) The seismic waves generated at the time of earthquake fault movement were not all of a uniform character; 2) As these waves pass through the earth on their way from the fault to the bridge site, they are modified by the soil and rock media through which they travel; 3) Once the seismic waves reach the bridge site they undergo further modifications, which are dependent on the characteristics of the ground and soil beneath the bridge. These three factors are referred [Agarwal et al. (2008)] as

- source effects
- path effects
- site effects

2.4 Ground Motion and Bridge Frequencies

The characteristics of earthquake ground motions which have the greatest importance for bridges are the duration, amplitude (of displacement, velocity and acceleration) and frequency of the ground motion. Surface ground motion at the bridge site, then, is actually a complex superposition of vibrations of different frequencies. At any given site, some frequencies usually predominate. The distribution of frequencies in a ground motion is referred to as its frequency content.

The response of the bridge to ground motion is as complex as the ground motion itself, yet typically quite different. However, the bridge vibrations tend to center around one particular frequency, which is known as its natural or fundamental frequency. Natural frequency is a function of mass and stiffness of the system.

When the frequency contents of the ground motion are centered around the bridge's natural frequency, the bridge and the ground motion are said to be in resonance with one another. Resonance tends to increase or amplify the bridge's response. Because of this, bridges suffer the greatest damage from ground motion at a frequency close or equal to their own natural frequency.

2.5 Response Spectra

Bridges can respond in widely differing manners to the same earthquake ground motion depending on their geometry. Conversely, any given bridge will act differently during different earthquakes, which gives rise to the need of concisely representing the range of responses of a bridge to ground motion of different frequency contents. Such a representation is known as a response spectrum. A response spectrum is a kind of graph which plots the maximum response values of acceleration, velocity and displacement against period or frequency. Response spectra are very important "tools" in earthquake engineering [Chopra (2002)].; Siddique (2006)].

Figure 2.1 shows a highly simplified version of a response spectrum. Even though highly simplified, it does show how bridge response characteristics vary with bridge frequency or period: as bridge period lengthens, accelerations decrease and displacement increases. On the other hand, bridges with shorter periods (i.e higher natural frequencies), undergo higher accelerations but smaller displacements. In the subsequent chapters, it has been described in more detail.

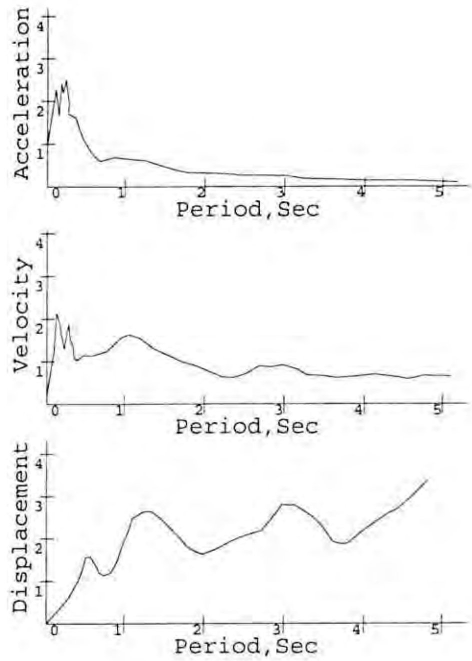


Figure 2.1 Response Spectrum

The amount of acceleration which a bridge undergoes during an earthquake is a critical factor in determining how much damage it will suffer. The spectrum described in Fig. 2.1 provides an indication of how accelerations are related to frequency characteristics.

2.6 Earthquake Forces on Structure

Earthquake force is the inertia force. It is induced on the structure due to the oscillatory motion of its body by the excitation of foundation. It can be derived using Newton's Laws of Motion and the concept of dynamic equilibrium for a mass 'm' acted upon by a ground acceleration $\ddot{u}_g(t)$, that the effective force, $P_{eff}(t) = -m\ddot{u}_g(t)$ [Chopra, (2002)]. This effective force is represented by equivalent static load at different levels of structure and at deck level of bridge in particular. Earthquake load is a dynamic load. Due to earthquake load being transient in nature, a structure may vibrate in different mode shapes and the load on the structure, its intensities and direction are dependent on these mode shapes.

For example, the Fig. 2.2 below shows first three fundamental modes of a bridge bent.

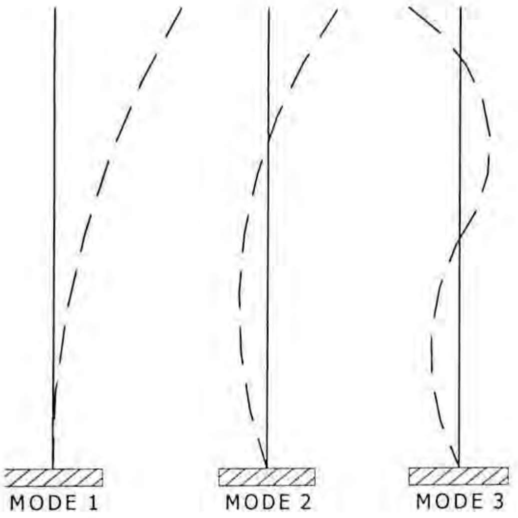


Figure 2.2 Fundamental Mode of a typical bridge bent

From Fig. 2.2 it is seen that different mode shape of the structure causes different load intensities and direction to the structure. If only the first mode is considered and assumed linear mode shape then the structure experiences a triangular shaped lateral load. Equivalent Static Load method as adopted in structure codes is simple approximation of first mode of vibration with the mode shape considered as linear.

However, for bridge with higher time period (flexible ones), the effect of higher modes become important. For regular shaped and non slender bridges the equivalent static method as approached by AASHTO gives an estimate of seismic demand on the bridges that may cause limited damage but very low probability of damage.

2.6.1 Response Spectrum Analysis

The seismic force generated in structures varies according to their dynamic properties even though they stand on the same ground and are subjected to the same seismic motion.

The response spectrum is schematically depicted in the Fig. 2.3. Three types of single-degree-of freedom systems with the same damping constants *h1* but different natural periods Fig. 2.3(b) are subjected in the same manner to the earthquake motion shown in the Fig. 2.3(a). However, each point mass shows a different response according to the relation between properties of earthquake motion and its natural period under the single-degree-of-freedom system.

Mass having a comparatively shorter natural period *T1* vibrates rapidly while mass having a longer natural period *T3* vibrates slowly [Chopra (2002)]. This situation is illustrated in the Fig. 2.3(c). The plot in the Fig. 2.3(d) shows the maximum response value for a given time interval and the natural period. If the vibration characteristics continuously varied for the systems corresponding to extremely rigid structures with a very short natural period to flexible structures having long natural periods, then plot the maximum response values, a response spectrum for damping constant *h1* is obtained.

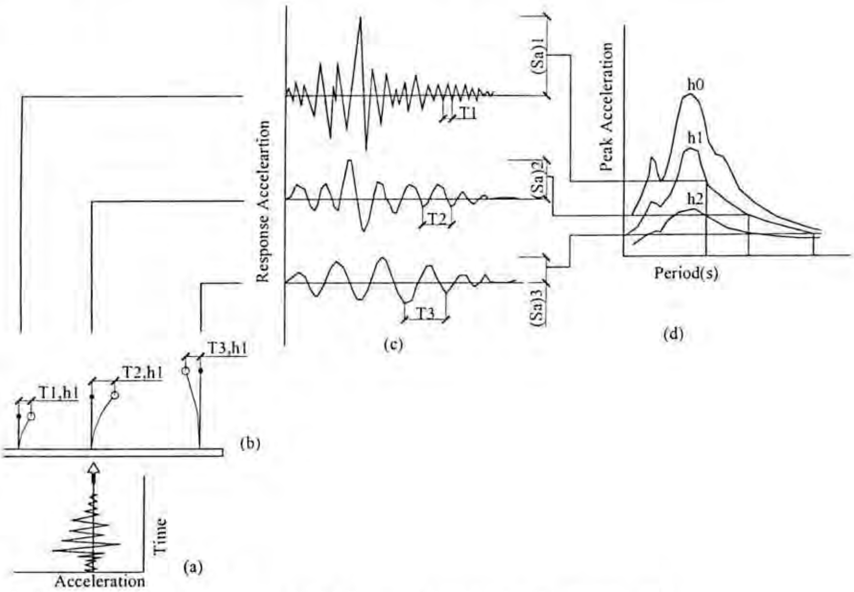


Figure 2.3 Response of different fundamental period

So, knowing the period of the structure, peak spectral acceleration of the structure can be estimated.

Response spectrum analysis (RSA) is a procedure for computing the statistical maximum response of a structure to a base excitation (or earthquake). Each of the vibration modes that are considered may be assumed to respond independently as a single-degree-of-freedom system. Various design codes specify response spectra which determine the base acceleration applied to each mode according to its period. Having determined the response of each vibration mode to the excitation, it is necessary to obtain the response of the structure by combining the effects of each vibration mode. Because the maximum response of each mode will not necessarily occur at the same instant, the statistical maximum response is taken as the square root of the sum of the squares of the individual response.

Now for a given $u_g(t)$, the deformation response u(t) of the system depends only on the natural frequency ω_n or natural period T_n of the system and its damping ratio, ζ. Thus any two systems having the same values of T_n and ζ will have the same deformation response u(t) even though one system may be massive and with higher stiffness than the other.

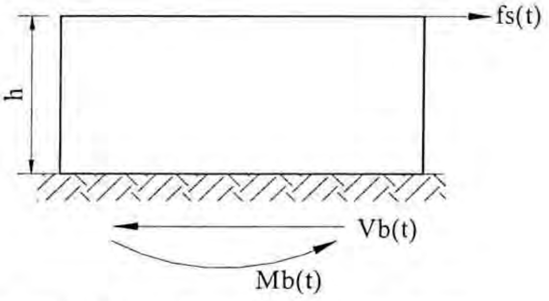


Figure 2.4 Equivalent static force

Once the deformation response history u(t) has been evaluated by dynamic analysis of the structure, the internal forces can be determined by static analysis of the structure at each time instant. Preferred approach in earthquake engineering is based on the concept of the equivalent static force f_S. Equivalent static force, f_S at any time instant t, may be defined as

$$f_S(t) = ku(t)$$

where k is the lateral stiffness of the frame. Expressing k in terms of mass gives

$$f_S(t) = m\omega^2{}_n u(t) = mA(t)$$

where $A(t) = \omega^2{}_n u(t)$

The equivalent static force is m times A(t), the pseudo-acceleration. The pseudo-acceleration response A(t) of a system can readily be computed from the deformation response u(t).

2.6.2 Time History Analysis

Earthquake excitation is time dependent, highly irregular and arbitrary in nature. Usually earthquake excitation in the form of acceleration or displacement or velocity is recorded for a time interval of 0.02 to 0.005 seconds. In this dynamic analysis procedure the response of a structure at every time interval is recorded for the whole earthquake period and the statistical average is represented. Because of its inherent complexities of the procedure and nondeterministic nature of the input ground motion, the analysis procedure has not become popular in the design houses for designing of the structures.

2.7 Provision for Earthquake Load Analysis in AASHTO

Prior to 1971, American Association of State Highway and Transportation Officials [AASHTO (1992, 1996 & 2002)] specifications for seismic design of bridges were based in part on the lateral force requirement of structure developed by the Structural Engineers Association of California. Through an workshop in 1979 AASHTO developed the current standards for seismic design of bridges. Like other codes AASHTO also stipulates equivalent static load method for seismic analysis of bridges. They classified the bridges on the basis of seismic performance category [SPC] which they designated as SPC A, B, C & SPC D categories based on Acceleration Coefficient and Importance Classification. This classification is intended for the need of extent of analysis requirement. The selected bridges taken under study fall in the categories of SPC B and SPC C. Before publication of BNBC, Bridge designers of Bangladesh used an arbitrarily value of Acceleration Coefficient to estimate seismic force using AASHTO specification. Such difficulty no longer exists after publication of BNBC in 1993. Unlike other codes AASHTO has different provisions for analysis procedures for structures subjected to earthquake.

AASHTO developed two analysis procedures to calculate elastic seismic forces.

They are:

i. PROCEDURE – I: Single Mode Spectral Method and

ii. PROCEDURE – II: Multimode Spectral Method

2.7.1 Single Mode Spectral Method

In this method the dynamic response of a bridge has been reduced to a statics after introducing an inertia force. Bridge are generally continuous system. The equation of motion for such a system is conveniently formulated following the principal of energy. The end result in this method is represented by an equivalent static load at deck level incorporating dynamic response. This method estimates an equivalent static force to be applied at deck considering seismicity of the region with the first mode behavior of the structure.

2.7.1.1 Calculation of Seismic Force

The design equivalent earthquake load denoted by P_e having the unit of force/unit length in a given direction is determined from the following relation.

$P_e = \beta C_s w v_s / \gamma$

w = dead wt. of superstructure and its tributary substructure (force/unit length)

v_s = static displacement (unit of length) corresponding to uniform loading, P_o

$C_s = 1.2AS/T^{2/3}$ (unitless)

$T = 2\pi\sqrt{\gamma / P_o \alpha g}$ (unit of time)

P_o = uniform loading to be applied over the length of the bridge (force/unit length) and is arbitrarily set equal to 1

$\alpha = \int v_s dx, dy$

$\beta = \int wv_s dx,dy$

$\gamma = \int wv_s(x)^2 dx,dy$

It is to be noted that the above formula has been derived using energy principal and equation of motion as described in section 2.6 above.

2.7.1.2 Distribution of Earthquake Load

Equivalent seismic force, P_e as mentioned above is to be applied at deck and after performing static analysis, the bridge components are designed following code stipulated load combination.

2.7.2 Multimode Spectral Method

It is dynamic analysis using Response Spectrum. It is usually carried out by a computer program because of involving huge volume of calculation. This method is briefly described below:

The uncoupled normal mode equation of motion are of the form

$$\ddot{u} + 2\zeta_i\omega_i\ddot{u}\,(t) + \omega^2_i\, u_i(t) = Pi(t)/Mi \quad (i= 1,2,3......n)$$

where, u_i , ω_i and ζ_i are the mode amplitude, frequency and damping ratios, respectively, and the modal load Pi(t) and the participating mass or generalized mass Mi.

2.7.2.1 Response Spectrum for Multimode Analysis Procedure

In earthquake engineering response spectrum is a plot of peak value of response quantity as function of natural period of vibration period T_n of the system, or related parameter such as circular frequency, ω_n or cyclic frequency f_n. Each such plot is for SDF system having a fixed damping ratio, ζ.

Response spectrum to be used in dynamic analysis shall be either of the following:

(i) Site Specific Design Spectra: A site specific response spectra shall be developed based on the geologic, tectonic, seismologic, and soil characteristics associated with the specific site.

(ii) Normalized Response Spectra: In absence of a site-specific response spectrum, the normalized response spectra shall be used.

The normalized response spectrum curves provided in the BNBC code are prepared for three different soil types with 5% of the critical damping (Fig. 2.5).

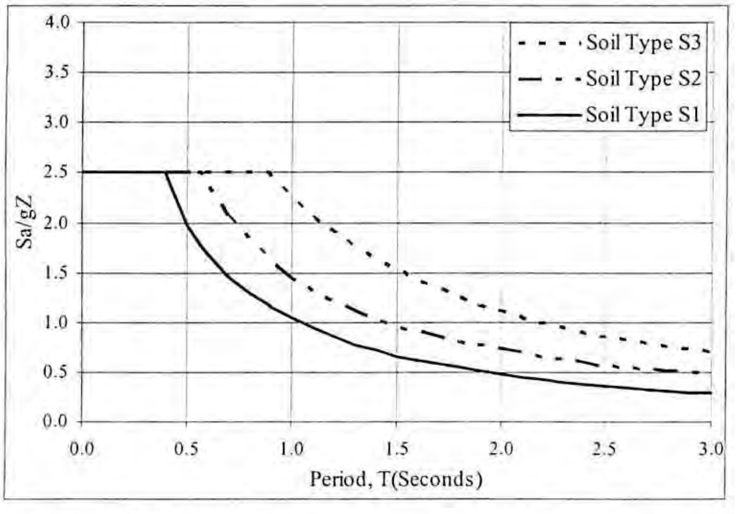


Figure 2.5 Normalized Response Spectrum Curves for 5% Damping of BNBC, 1993

Chapter 3

NONLINEAR STATIC (PUSHOVER) ANALYSIS AND CONCEPT OF SEISMIC EVALUATION OF STRUCTURES

3.1 Introduction

Earthquake response of a concrete structure can be evaluated either by linear or non-linear methods. Nonlinear time history method of analysis predicts the actual response but this method is very complicated and considered impractical for general use. The central focus of this thesis is to introduce the simplified procedure which is termed as Pushover Analysis. Pushover analysis, as the name implies, is the process of pushing horizontally, with a prescribed monotonically incremental loading pattern until a structure reaches a limit state of deformation beyond elastic range. The analysis generate "pushover" or capacity curve of a structure. This is a plot of progressive lateral displacement as a function of the increasing level of force applied to the structure. Pushover analysis is a simplified static non-linear analysis method which use capacity curve and reduced response spectrum, required to incorporate inelastic behavior, to estimate maximum displacement of a bridge without collapse under a given level of earthquake.

An elastic analysis gives a good indication of the elastic capacity of structures. Beyond elastic range, it can neither predict failure mechanisms nor include the redistribution of forces during progressive yielding of the structure when shaken by an earthquake excitation. Inelastic analyses procedures help demonstrate how structures really work by identifying modes of failure and potential for progressive collapse. The use of inelastic procedures for design helps engineers understand better how structures will behave when subjected to major earthquakes exceeding the elastic capacity of the structure. Application of this resolves some of uncertainties associated with code provisions for seismic design procedures.

3.2 Methods to Perform Simplified Nonlinear Analysis

Two key elements of a performance-based design procedure are demand and capacity. Demand is the representation of the earthquake ground motion. Capacity is the representation of the structure's ability to resist the seismic forces. The performance is dependent on the manner that the capacity is able to handle the demand. In other words, the structure must have the capacity to resist the demands of the earthquake such that the performance of the structure is compatible with the objectives of the design.

Simplified nonlinear analysis procedures using pushover methods, such as the capacity spectrum method and the displacement coefficient method, require determination of three primary elements: capacity, demand (displacement) and performance. Each of these elements is briefly discussed below.

3.2.1 Capacity

The overall capacity of a structure depends on the strength and deformation capacities of the individual components of the structure. In order to determine capacities beyond the elastic limits, some form of nonlinear analysis, such as the pushover procedure, is required. This procedure perform a series of elasto-plastic analysis following iterative method of non-linear analysis to generate a force-displacement capacity diagram of the overall structure. In every step, the mathematical model of the structure is modified to account for reduced resistance of yielded components. A lateral force distribution is again applied until additional components yield. This process is continued until the structure becomes unstable or until a predetermined limit of deformation is reached. The capacity curve approximates how structures behave after exceeding their elastic limit.

3.2.2 Demand (displacement)

Ground motions during an earthquake produce complex horizontal displacement pattern in structures that may vary with time. Tracking this motion at every time-step to determine structural design requirements is judged impractical. Traditional linear analysis methods use lateral forces to represent a design condition. For nonlinear methods it is easier and more direct to use a set of lateral displacements as a design condition. For a given structure and ground motion, the displacement demand is the estimate of the maximum expected response(Damage strategy) of a structure during a ground motion.

3.2.3 Performance

Once a capacity curve and demand displacement is defined, a performance check can be done. A performance check verifies that structural and nonstructural components are not damaged beyond the acceptable limits of the performance objective for the forces and displacement imposed by the displacement demand.

3.2.4 Static Non-linear (Pushover) Analysis

The outcome of pushover analysis is the development of the capacity curve. The capacity curve is derived from an progressive elasto-plastic static analysis for a structure. The graphical presentation of the analytical result is known as capacity curve. Thus the capacity curve is simply a plot of the reactions, "V," on the structure at various increments of applied lateral loading against the lateral deflection of the structure at deck level. If a bridge had infinite linear elastic capacity, this capacity curve would be a straight line with a slope equal to the global stiffness of the structure. Since real bridge do not have infinite linear elastic capacities, the capacity curve typically consists of a series of straight-line segments with decreasing slope. This shape of the curve represents the progressive degradation in structural stiffness that occurs as the bridge is subjected to increased lateral displacement associated with yielding and damage. The slope of a straight line drawn from the origin of the plot for this curve to a point on the curve at any lateral displacement, "*d*," represents the secant or "effective" stiffness of the structure when pushed laterally to that displacement.

A typical capacity curve of a hypothetical structure is shown in Fig. 3.1. In Fig. 3-1, the discrete points indicated by the symbol '♦' represent the occurrence of important events in the lateral response history of the structure. Such an event may be the initiation of yield in a particulars structural element or a particular type of damage, such as spalling of cover concrete on a column or shear failure. Each point is determined by a different

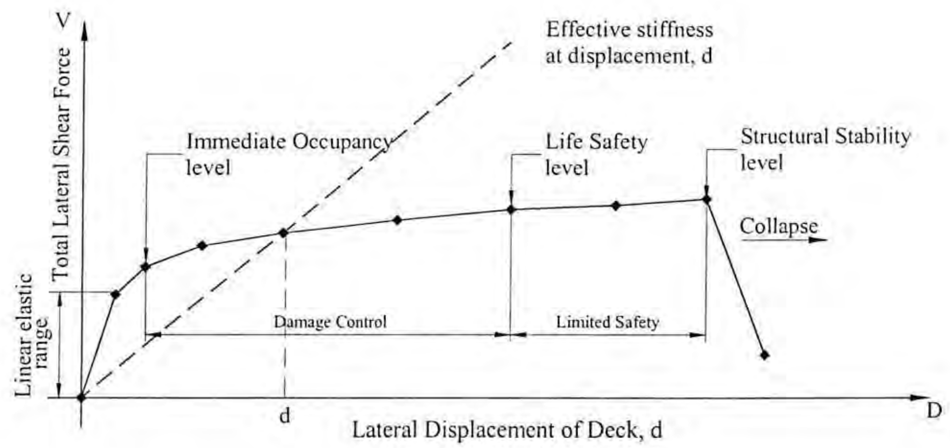


Figure 3.1 Typical capacity curve

analysis sequence. Then, by evaluating the cumulative effects of damage sustained at each of the individual events, and the overall behavior of the structure's increasing lateral displacements, it is possible to determine and indicate on the capacity curve those total structural lateral displacements that represent limits on the various structural performance levels, as has been done in Fig. 3-1.

The process of defining lateral deformation points on the capacity curve at which specific structural performance levels may be said to have occurred requires the exercise of considerable judgment on the part of the engineer. For each of the several structural performance levels and global performance levels defined in Section 3.6 to 3.8, defines global system response limits as well as acceptance criteria for the individual structural elements that make up typical bridge in particular and a structure in general. These acceptance criteria generally consist of limiting values of element deformation parameters, such as the plastic chord rotation of a beam or shear angle of a pier. These limiting values have been selected as reasonable approximate estimates of the average deformations at which certain types of element behavior such as cracking, yielding, spalling, or crushing, may be expected to occur. As the incremental static nonlinear analyses are performed, the engineer must monitor the cumulative deformations of all important structural elements and evaluate them against the acceptance criteria set before.

The point on the capacity curve at which the first element exceeds the permissible deformation level for a structural performance level does not necessarily represent that the structure as a whole reaches that structural performance level. Most structures contain many elements and have considerable redundancy. Consequently, the onset of unacceptable damage to a small percentage of these elements may not represent an unacceptable condition with regard to the overall performance of the structure. When determining the points along the capacity curve for the structure at which the various structural performance levels may have said to be reached, the engineer must view the performance of the structure as a whole and consider the importance of damage predicted for the various elements on the overall behavior of the structure.

The methodology described by ATC-40, incorporates the concept of "primary" and "secondary" elements to assist the engineer in making these judgments. Primary elements are those that are required as part of the lateral force resisting system for the structure. All other elements are designated as secondary elements. For a given performance level, secondary elements are generally permitted to sustain more damage than primary elements since degradation of secondary elements does not have a significant effect on the lateral load resisting capability of the structure. If in the development of the capacity curve it is determined that a few elements fail to meet the acceptance criteria for a given performance level at an increment of lateral loading and displacement, the engineer has the ability to designate these "nonconforming" elements as secondary, enabling the use of more liberal acceptance criteria for these few elements. But the structure with fewer redundancy like bridges may not enjoy this opportunity.

3.2.5 Capacity Spectrum Method

The capacity spectrum method, a nonlinear static procedure(pushover analysis), provides a graphical representation of the global force-displacement capacity curve of the structure (i.e. pushover curve) and compares it to the response spectra, a representations of the earthquake demands. This method is a very useful tool in the evaluation and retrofit design of existing steel and concrete structures. The graphical representation provides a clear picture of how a structure responds to earthquake ground motion, and, as illustrated below, it provides an immediate and clear picture of how various retrofit strategies, such as adding stiffness or strength, will affect the structural response to earthquake demands.

The capacity spectrum curve for the structure is obtained by transforming the capacity curve from lateral force (V) vs. lateral displacement (d) coordinates to spectral acceleration (Sa) vs. spectral displacement (Sd) coordinates using the modal shape vectors, participation factors and modal masses obtained from a modal analysis of the structure. In order to compare the Structure's capacity to the earthquake demand, it is required to plot the response spectrum and the capacity spectrum on the same plot. The conventional response spectrum plotted in spectral acceleration vs. period coordinate has to be changed in to spectral acceleration vs. spectral displacement coordinate. This form of response spectrum is known as acceleration displacement response spectrum (ADRS).

a. ADRS Format

Capacity Spectrum method requires plotting the capacity curve in spectral acceleration and spectral displacement domain. This representation of spectral quantities is knows as Acceleration- Displacement-Response-Spectra in brief ADRS. Spectral quantities like spectral acceleration, spectral displacement and spectral velocity is related to each other to a specific structural period T. Standard codes of practices usually provide response spectrum in spectral acceleration *vs*. period format which is the conventional format.

Each point on the curve defined in the Fig. 3.2 is related to spectral displacement by mathematical relation, $S_d = \frac{1}{4\pi^2} S_a T^2$. Converting using this relation, response spectrum in ADRS format may be obtained.

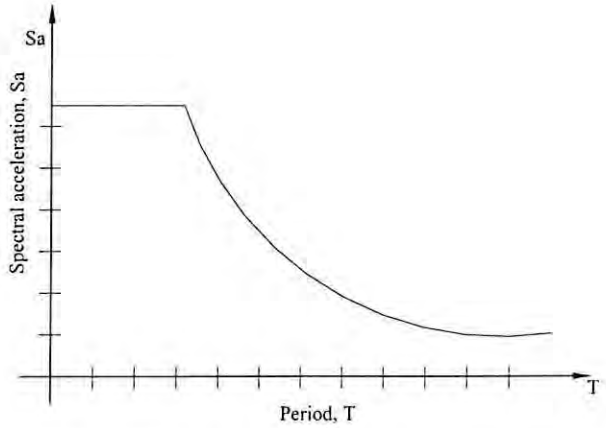


Figure 3.2 Code specified response spectrum in Spectral acceleration vs. Period.

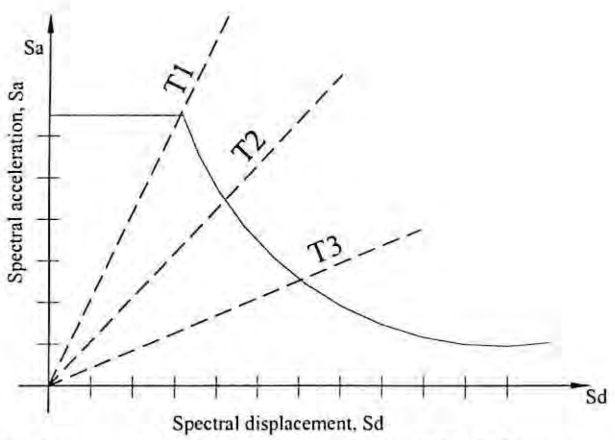


Figure 3.3 Response spectrum in ADRS format

Any line from the origin of the ADRS format represent a constant period T_i which is related to spectral acceleration and spectral displacement by the mathematical relation, $T = 2\pi\sqrt{\frac{S_d}{S_a}}$.

b. Capacity Spectrum

Capacity spectrum is a simple representation of capacity curve in ADRS domain. A capacity curve is the representation of Base Shear to deck/bent displacement. In order to develop the capacity spectrum from a capacity curve it is necessary to do a point by point conversion to first mode spectral coordinates.

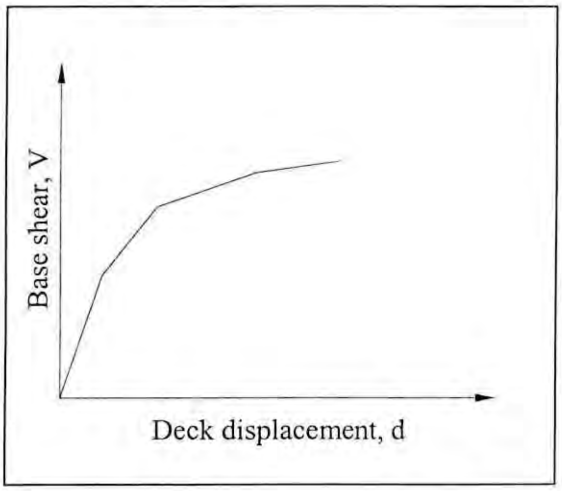


Figure 3.4 A Typical Capacity Curve

Any point corresponding values of base shear, V_i and roof deflection, Δ_i may be converted to the corresponding point of spectral acceleration, S_{ai} and spectral displacement, S_{di} on the capacity spectrum using relation,

$$S_{ai} = \frac{V_i / W}{\alpha_1} \text{ and } S_{di} = \frac{\Delta_{Deck}}{PF_1 \times \Phi_{1,Deck}}$$

Modal participation factor, PF_1 is calculated using equation,

$$PF_1 = \left[\frac{\sum_1^N (w_i \phi_{i,1}) / g}{\sum_1^N (w_i \phi^2_{i,1}) / g} \right]$$

Modal mass coefficient for the first mode, α_1 is calculated using equation,

$$\alpha_1 = \frac{\left[\sum_1^N (w_i \phi_{i,1}) / g \right]^2}{\left[\sum_1^N w_i / g \right] \left[\sum_1^N (w_i \phi^2_{i,1}) / g \right]}$$

Where:

Where:

PF_1	=	modal participation factor for the first natural mode.
α_1	=	modal mass coefficient for the first natural mode
$\Phi_{1, Dexk}$	=	deck level amplitude of the first mode.
w_i/g	=	mass assigned to level i
ϕ_{i1}	=	amplitude of mode 1 at level i
N	=	level N, the level which is the uppermost in the main portion of the structure
V	=	base shear
W	=	bridge dead weight plus likely live loads
Δ_{deck}	=	deck displacement
S_a	=	spectral acceleration
S_d	=	spectral displacement

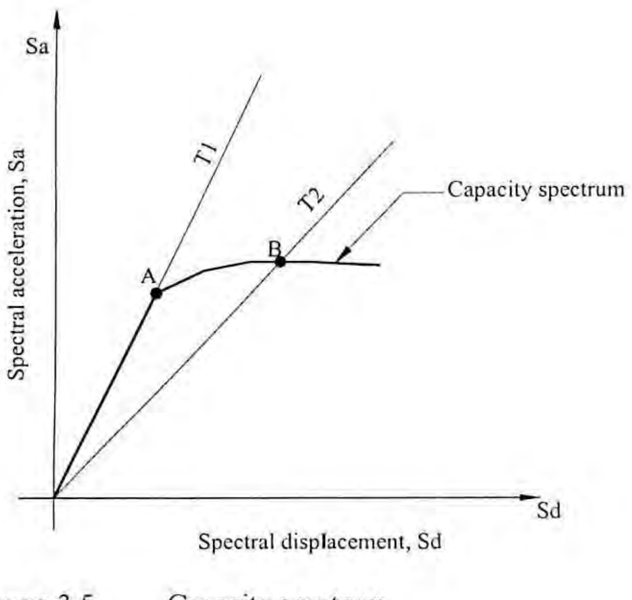


Figure 3.5 Capacity spectrum

Fig. 3.5 shows a typical capacity spectrum converted from capacity curve of Fig. 3.4 of a hypothetical structure. It is seen in the capacity spectrum that up to some displacement corresponding to point A, the period is constant T_1. That is the structure is behaving elastically. As the structure deflects more to point B, it goes to inelastic deformation and its period lengthens to T_2.

When the capacity curve is plotted in S_a vs. S_d coordinates, radial lines drawn from the origin of the plot through the curve at various spectral displacements have a slope (ω'), where, ω' is the radial frequency of the effective (or secant) first-mode response of the structure if pushed by an earthquake to that spectral displacement.

Using the relationship $T' = \dfrac{2\pi}{\omega'}$, it is possible to calculate, for each of these radial lines, the effective period of the structure if it is pushed to a given spectral displacements.

Fig. 3-6 is a capacity spectrum plot obtained from the capacity curve of a hypothetical structure shown in Fig. 3-1 and plotted with the effective modal periods shown.

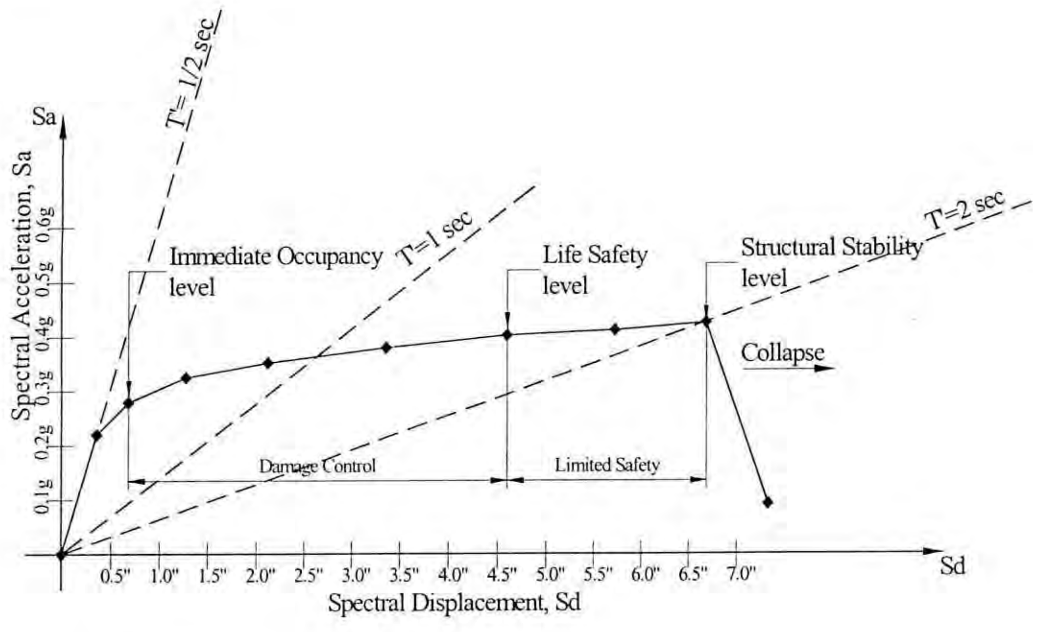


Figure 3.6 Typical Capacity Spectrum

The particular structure represented by this plot would have an elastic period of approximately ½ second. As it is pushed progressively further by stronger ground motion, this period lengthens. The structure represented in Fig. 3.1 and Fig. 3.6 would experience collapse before having its stiffness degraded enough to produce an effective period of 2 seconds.

The capacity of a particular structure and the demand imposed upon it by a given earthquake motion are not independent. One source of this mutual dependence is evident from the capacity curve itself. As the demand increases, the structure eventually yields and, as its stiffness decreases, its period lengthens. Conversion of the capacity curve to spectral ordinates (ADRS) makes this concept easier to visualize. Since the seismic accelerations depend on period, demand also changes as the structure yields. Another source of mutual dependence between capacity and demand is effective damping. As a structure yield in response to seismic demand it dissipates energy with hysteretic damping.

The capacity spectrum method initially characterizes seismic demand using an elastic response spectrum. This spectrum is plotted in spectral ordinates (ADRS) format showing the spectral acceleration as a function of spectral displacement. This format allows the demand spectrum to be "overlaid" on the capacity spectrum for the structure. The intersection of the demand and capacity spectra, if located in the linear range of the capacity, would define the actual displacement for the structure; however this is not normally the case as most analyses include some inelastic nonlinear behavior. To find the point where demand and capacity are equal, a point on the capacity spectrum need to be selected as an initial estimate. Using the spectral acceleration and displacement defined by this point, reduction factors may be calculated to apply to the 5% elastic spectrum to account for the hysteretic energy dissipation, or effective damping, associated with the specific point. If the reduced demand spectrum intersects the capacity spectrum at or near the initial assumed point, then it is the solution for the unique point where capacity equals demand. If the intersection is not reasonably close to the initial point, then a new point somewhere between may be assumed and repeat the process until a solution is reached. This is the performance point where the capacity of the structure matches the demand or the specific earthquake.

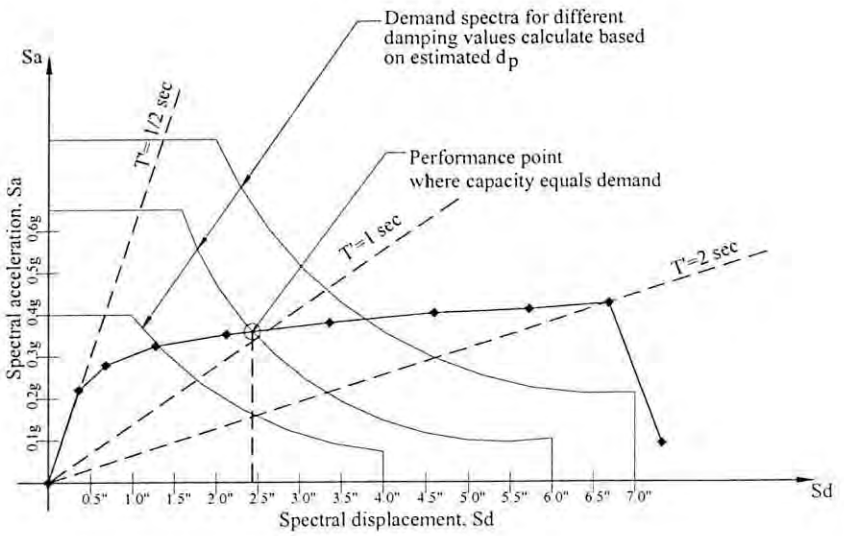


Figure 3.7 Determination of performance point

Once the performance point has been determined, the acceptability of a rehabilitation design to meet the project performance objectives can be judged by evaluating where the performance points falls on the capacity curve. For the structure and earthquake represented by the overlay indicated in Fig. 3.7, the performance point occurs within the central portion of the damage control performance range as shown in Fig. 3-6, indicating that for this earthquake this structure would have less damage than permitted for the Life Safety level and more than would be permitted for the Immediate Occupancy level. With is information, the performance objective and/or the effectiveness of the particular rehabilitation strategy to achieve the project performance objectives can be judged.

3.2.6 Displacement Coefficient Method

Another procedure for calculating demand displacement is 'Displacement Coefficient Method' which provides a direct numerical process for calculating the displacement demand. Displacement Coefficient Method has not been explored. Performance analysis of the structures under this thesis was made using Capacity Spectrum Method.

3.3 Seismic Performance Evaluation

The seismic evaluation procedures ends at the comparison between earthquake demand that is placed on a structure to its capacity to meet this demand. Code provision design procedure have expressed these demand and capacity as forces. In this case base shear (total horizontal force at the lowest level of the structure) is the basic parameter that is used for this purpose. The base shear demand that would be generated by a given earthquake is calculated, and compares it to the base shear capacity of the structure. If the structure were subjected to a force equal to this base shear, some inelastic deformation might occur in some structural elements, but the structure would not collapse. If the demand generated by the earthquake is less than the capacity then the design is deemed acceptable.

The first formal seismic design procedures recognized that the earthquake accelerations would generate forces proportional to the weight of the structure. Over the years, experimental observation about the actual behavior of real structures in earthquakes and theoretical understanding of structural dynamics advanced. The basic procedure was modified to reflect the fact that the demand generated by the earthquake accelerations was also a function of the stiffness of the structure and its material properties.

3.4 Nonlinear Static Procedure for Capacity Evaluation of Structures

Instead of comparing forces, nonlinear static procedures use displacements to compare seismic demand to the capacity of a structure. This approach incorporates contribution of ductility to the inelastic capacity. The inelastic capacity i.e capacity of a deformed structure is then a measure of its ability to dissipate earthquake energy. The current trend of seismic analysis is focused towards this simplified inelastic procedures.

The recommended methodology is on the development of inelastic capacity curve for the structure. This curve is a plot of the horizontal movement of a structure as it is pushed to one side vs. its shear resistance at the base. Initially the plot is a straight line as the structure moves linearly. As the parts of the structure yield the plot begins to curve as the structure softens. The development of capacity curve is now accomplished through a mathematical model under a computer structural software. A monotonically incremental of load in a pattern determined by incorporating the dynamic response of the structure under seismic load is applied to the structure. The corresponding deck displacements and support reactions build the capacity curve of the bridge. The capacity curve defines the capacity of the structure exclusively without caring of any specific seismic demand. In this sense it replaces the base shear capacity of conventional procedures. When an earthquake displaces the structure laterally, its response is represented by a point on this curve. A point on the curve defines a specific damage state of the structure, since the deformation of its entire components can be related to the global displacement of the structure.

In fact the inelastic capacity of a particular structure and the demand imposed upon it by a given earthquake motion are interrelated. One source of this mutual dependence is evident from the capacity curve itself. As the demand (deformation) increases the structure eventually yields and, as its stiffness decreases, its period lengthens. Since the seismic accelerations to be induced on the structure depend on its period of oscillation. With the progressive yielding of the structural elements, period of the structure changes. Another source of mutual dependence between capacity and demand is effective damping. As structure yields in response to seismic demand, it dissipates energy with hysteretic damping. Bridges having large, stable hysteretic loops during cyclic yielding dissipate more energy than those with pinched loops caused by degradation of strength and stiffness. Since the energy that is dissipated need not be stored in the structure, the damping has the effect of diminishing displacement demand.

3.5 Bridge Performance Levels and Ranges

The Performance of a bridge under any particular earthquake event is dependent on a wide range of parameters. These parameters are defined [ATC-40 & 6-2, (1996); FEMA 356, (2000)] qualitatively in terms of (i) the safety afforded by the structure to the occupants during and after the event; (ii) the cost of restoring the structure to pre-earthquake condition; and (iv) overall economic, architectural, or historic impacts on the larger community. These performance characteristics are directly related to the extent of damage that could be sustained by the structure.

The Federal Emergency Management Agency (FEMA) in its report 'Prestandard and Commentary for the Seismic Rehabilitation of Structures [FEMA-356, (2000)] defines the structural performance level of a structure to be selected from four discrete structural performance levels and two intermediate structural performance ranges. The discrete Structural Performance Levels are

Immediate Occupancy (S-1),

Life Safety (S-3),

Collapse Prevention (S-5), and

Not Considered (S-6).

The intermediate Structural Performance Ranges are the

Damage Control Range (S-2) and the Limited Safety Range (S-4)

The definition of these performance ranges are given by FEMA (FEMA-356, 2000).

Acceptance criteria for performance within the Damage Control Structural Performance Range may be obtained by interpolating the acceptance criteria provided for the Immediate Occupancy and Life Safety Structural Performance Levels. Acceptance criteria for performance within the Limited Safety Structural Performance Range may be obtained by interpolating the acceptance criteria provided for the Life Safety and Collapse Prevention Structural Performance Levels. The performance levels and ranges, as per FEMA [FEMA-356, (2000)], are described in the sections that follow.

3.5.1 Immediate Occupancy Structural Performance Level (S-1)

Structural Performance Level S-1, Immediate Occupancy, may be defined as the post-earthquake damage state of a structure that remains safe to occupy, essentially retains the pre-earthquake design strength and stiffness of the structure, and is in compliance with the acceptance criteria specified in this standard for this Structural Performance Levels defined in Table 3.1 to Table 3.3

Structural Performance Level S-1, Immediate Occupancy, means the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical and lateral-force-resisting systems of the structure retain nearly all of their pre-earthquake strength. The risk of life-threatening injury as a result of structural damage is very low, and although some minor structural repairs may be appropriate, these would generally not be required prior to re-occupancy.

3.5.2 Damage Control Structural Performance Level (S-2)

Structural Performance Range S-2, Damage Control, may be defined as the continuous range of damage states between the Life Safety Structural Performance Level (S-3) and the Immediate Occupancy Structural Performance Level (S-1) defined in Table 3.1 to Table 3.3.

Design for the Damage Control Structural Performance Range may be desirable to minimize repair time and operation interruption, as a partial means of protecting valuable equipment and contents, or to preserve important historic features when the cost of design for immediate occupancy is excessive.

3.5.3 Life Safety Structural Performance Level (S-3)

Structural Performance Level S-3, Life Safety, may be defined as the post-earthquake damage state that includes damage to structural components but retains a margin against onset of partial or total collapse in compliance with the acceptance criteria specified in FEMA [FEMA-356, (2000)] for this Structural Performance Level defined in Table 3.1 to Table 3.3.

Structural Performance Level S-3, Life Safety, means the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the structure. Injuries may occur during the earthquake; however, the overall risk of life-threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure; however, for economic reasons this may not be practical. While the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to re-occupancy.

3.5.3.1 Limited Safety Structural Performance Level (S-4)

Structural Performance Range S-4, Limited Safety, may be defined as the continuous range of damage states between the Life Safety Structural Performance Level (S-3) and the Collapse Prevention Structural Performance Level (S-5) defined in Table 3.1 to Table 3.3.

3.5.3.2 Collapse Prevention Structural Performance Level (S-5)

Structural Performance Level S-5, Collapse Prevention, may be defined as the post-earthquake damage state that includes damage to structural components such that the structure continues to support gravity loads but retains no margin against collapse in compliance with the acceptance criteria specified in FEMA [FEMA-356, (2000)] for this Structural Performance Level defined in Table 3.1 to Table 3.3.

Structural Performance Level S-5, Collapse Prevention, means the post-earthquake damage state in which the structure is on the verge of partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral-force resisting system, large permanent lateral deformation of the structure, and—to a more limited extent—degradation in vertical-load-carrying capacity. However, all significant components of the gravity load resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling hazards from structural debris may exist. The structure may not be technically practical to repair and is not safe for re-occupancy, as aftershock activity could induce collapse.

3.6 Target Structure Performance Levels

Structure performance is a combination of the both structural and nonstructural components. Tables 3.1, Table 3.2 and Table 3.3 (FEMA-356, 2000) describe the approximate limiting levels of structural damage that may be expected of structure evaluated to the levels defined for a target seismic demand. These tables represent the physical states of mathematical calculation of different performance levels.

Table 3-1 Damage Control and Structure Performance Levels(FEMA-356, 2000)

	Target Structure Performance Levels			
	Collapse Prevention Performance Level	Life Safety Performance Level	Immediate Occupancy Performance Level	Operational Performance Level
Overall Damage	Severe	Moderate	Light	Very Light
General	Little residual stiffness and strength, but load-bearing columns and walls function. Large permanent drifts. Some exits blocked. Infills and unbraced parapets failed or at incipient failure. Structure is near collapse	Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls or tipping of parapets. Some permanent drift. Damage to partitions. Structure may be beyond economical repair.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All systems important to normal operation are functional.
Nonstructural components	Extensive damage	Falling hazards mitigated but many architectural, mechanical and electrical systems are damaged.	Equipment and contents are generally secure, but may not operate due to mechanical failure or lack of utilities.	Negligible damage occurs. Power and other utilities as available, possibly from standby sources.
Comparison with performance intended for structures designed under the NEHRP Provisions, for the Design Earthquake	Significantly more damage and greater risk.	Somewhat more damage and slightly higher risk.	Less damage and lower risk.	Much less damage and lower risk.

Table 3-2 Structural Performance Levels and Damage Vertical Elements (FEMA-356, 2000)

		Structural Performance Levels		
		Collapse Prevention	Life Safety	Immediate Occupancy
Elements	Type	S-5	S-3	S-1
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some non-ductile columns. Severe damage in short columns	Extensive damage to beams. Spalling of cover and shear cracking (<1/8" width) for ductile columns. Minor spalling in non-ductile columns. Joint cracks <1/8" wide.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns	Minor spalling in non-ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in Joint <1/6" width.
	Drift	4% transient or permanent	2% transient; 1% permanent	1% transient; negligible permanent
	Secondary	Same as primary	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact	Same as primary
	Drift	5% transient or permanent	2.5% transient; 1% permanent	0.7% transient; negligible permanent
Concrete Walls	Primary	Major flexural and shear cracks and voids. Sliding at joints. Extensive crushing and buckling of reinforcement. Failure around openings. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Some boundary element stress, including limited buckling of reinforcement. Some sliding at joints. Damage around openings. Some crushing and flexural cracking. Coupling beams: extensive shear and flexural cracks; some crushing, but concrete generally remains in place.	Minor hairline cracking of walls, <1/16" wide. Coupling beams experience cracking <1/8" width.
	Secondary	Panels shattered and virtually disintegrated	Major flexural and shear cracks. Sliding at joints. Extensive crushing. failure around openings. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Minor hairline cracking of walls. Some evidence of sliding at construction joints. Coupling beams experience cracks <1/8" width. Minor spalling.
	Drift	2% transient or permanent	1% transient; 0.5% permanent	0.5% transient; negligible permanent
	Secondary	Same as primary	Some connection failures but no elements dislodged	Minor crushing and spalling at connections
Foundations	General	Major settlement and tilting	Total settlements <6" and differential settlements <1/2" in 30ft.	Minor settlement and negligible tilting.

1. Damage states indicated in this table are provided to allow an understanding of the severity of damage that may be sustained by various structural elements when present in structures meeting the definitions of the Structural Performance Levels. These damage states are not intended for use in post-earthquake evaluation of damage or for judging the safety of, or required level of repair to, a structure following an earthquake.

2. Drift values, differential settlements, crack widths, and similar quantities indicated in these tables are not intended to be used as acceptance criteria for evaluating the acceptability of a rehabilitation design in accordance with the analysis procedures provided in this standard; rather, they are indicative of the range of drift that typical structures containing the indicated structural elements may undergo when responding within the various Structural Performance Levels. Drift control of a rehabilitated structure may often be governed by the requirements to protect nonstructural components. Acceptable levels of foundation settlement or movement are highly dependent on the construction of the superstructure. The values indicated are intended to be qualitative descriptions of the approximate behavior of structures meeting the indicated levels.

3. For limiting damage to frame elements of infilled frames, refer to the rows for concrete or steel frames.

Table 3-3 Structural Performance Levels and Damage[1,2] – Horizontal Elements(FEMA-356, 2000)

	Structural Performance Levels		
	Collapse Prevention	Life Safety	Immediate Occupancy
Elements	S-5	S-3	S-1
Metal Deck Diaphragms	Large distortion with buckling of some units and tearing of many welds and seam attachments	Some localized failure of welded connections of deck to framing and between panels. Minor local buckling of deck	Connections between deck units and framing intact. Minor distortions.
Wood Diaphragms	Large permanent distortion with partial withdrawal of nails and extensive splitting of elements	Some splitting at connections. Loosening of sheathing. Observable withdrawal of fasteners. Splitting of framing and sheathing.	No observable loosening or withdrawal of fasteners. No splitting of sheathing or framing.
Concrete Diaphragms	Extensive crushing and observable offset across many cracks.	Extensive cracking (<1/4" width). Local crushing and spalling	Distributed hairline cracking. Some minor cracks of larger size (<1/8" width).
Precast Diaphragms	Connections between units fail. Units shift relative to each other. Crushing and spalling at joints.	Extensive cracking (<1/4" width). Local crushing and spalling.	Some minor cracking along joints.

1. Damage states indicated in this table are provided to allow an understanding of the severity of damage that may be sustained by various structural elements when present in structures meeting the definitions of the Structural Performance Levels. These damage states are not intended for use in post-earthquake evaluation of damage or for judging the safety of, or required level of repair to, a structure following an earthquake.

2. Drift values, differential settlements, crack widths, and similar quantities indicated in these tables are not intended to be used as acceptance criteria for evaluating the acceptability of a rehabilitation design in accordance with the analysis procedures provided in this standard; rather, they are indicative of the range of drift that typical structures containing the indicated structural elements may undergo when responding within the various Structural Performance Levels. Drift control of a rehabilitated structure may often be governed by the requirements to protect nonstructural components. Acceptable levels of foundation settlement or movement are highly dependent on the construction of the superstructure. The values indicated are intended to be qualitative descriptions of the approximate behavior of structures meeting the indicated levels.

3.7 Response Limit

To determine whether a structure meets a specified performance objective, response quantities from a nonlinear analysis are compared with limits given for appropriate performance levels [ATC-40, (1996) and FEMA-356, (2000)]. The response limits fall into two categories:

3.7.1 Global structure acceptability limits

These response limits include requirements for the vertical load capacity, lateral load resistance, and lateral drift. Table 3.4 gives the limiting values for different performance level.

a. Gravity Loads

The gravity load capacity of the structure must remain intact for acceptable performance at any level. Where an element or component loses capacity to support gravity loads, the structure must be capable of redistributing its load to other elements or components of the existing system.

b. Lateral Loads

Strength of some structural components are subjected to degradation over multiple load cycles. If a significant number of these components degrade, the overall lateral force resistance of the structure can be affected seriously. The lateral load resistance of the structure and the resistance to the effects of gravity loads acting on deformed shape of structure should not degrade by more than 20 percent of the maximum resistance of the structure for the extreme case.

Two effects can lead to loss of lateral load resistance with increasing displacement. The first is gravity loads acting through lateral displacements, known as the P-Δ

effect. The P-Δ effect is most prominent for flexible structures with little redundancy and low lateral load strength relative to the structure weight. The second effect is degradation of resistance of individual components of the structure under the action of reversed deformation cycles. When lateral load resistance of the structure degrades with increasing displacement, there is a tendency for displacements to accumulate in one direction. This tendency is especially important for long-duration events. The following table presents deformation limits of various performance levels. Maximum inelastic deformation is defined as the portion of the maximum total deformation beyond the effective yield point. For structural stability, the maximum total deformation at deck at the performance point should not exceed the quantity 0.33V/p, where V is the total calculated shear force at base and P is the total gravity load (i.e. dead plus likely live load) of deck system.

Table 3-4 Deformation Limits [ATC-40, (1996)]

	Performance Level			
Inter story Drift Limit	Immediate Occupancy	Damage Control	Life Safety	Structural Stability
Maximum total drift	0.01	0.01 ~0.02	0.02	$0.33\frac{V_i}{P_i}$
Maximum inelastic drift	0.005	0.005~0.015	No limit	No limit

3.7.2 Element and Component Acceptability Limit

a. Deformation and Force Controlled Actions

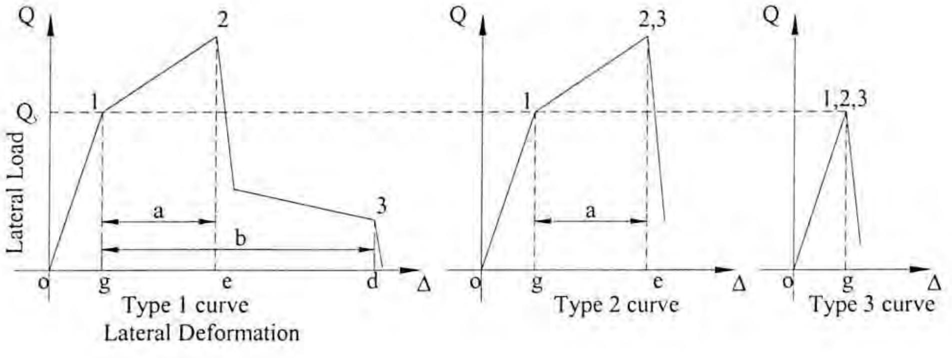


Figure 3.8 Component Force versus Deformation Curves (FEMA-356, 2000)

All structural actions may be classified as either deformation controlled or force-controlled using the component force versus deformation curves shown in Figure 3.8. The Type 1 curve depicted in Fig. 3.8 is representative of ductile behavior where there is an elastic range (point 0 to point 1 on the curve) followed by a plastic range (points 1 to 3) with non-negligible residual strength and ability to support gravity loads at point 3. The plastic range includes a strain hardening or softening range (points 1 to 2) and a strength-degraded range (points 2 to 3). Primary component actions exhibiting this behavior shall be classified as deformation-controlled if the strain-hardening or strain-softening range is such that o*e* > 2og; otherwise, they shall be classified as force controlled. Secondary component actions exhibiting Type 1 behavior shall be classified as deformation-controlled for any *e/g* ratio. The Type 2 curve is representative of ductile behavior where there is an elastic range (point 0 to point 1 on the curve) and a plastic range (points 1 to 2) followed by loss of strength and loss of ability to support gravity loads beyond point 2. Primary and secondary component actions exhibiting this type of behavior shall be classified as deformation-controlled if the plastic range is such that $e > 2g$; otherwise, they shall be classified as force controlled which is the representative of simply supported deck-girder bridge . The Type 3 curve depicted in Figure 3.8 is representative of a brittle or non-ductile behavior where there is an elastic range (point 0 to point 1 on the curve) followed by loss of strength and loss of ability to support gravity loads beyond point 1. Primary and secondary component actions displaying Type 3 behavior shall be classified as force-controlled [FEMA-356, (2000)].

b. Deformation-Controlled and Force- Controlled Behavior

Acceptance criteria for primary components that exhibit Type 1 behavior are typically within the elastic or plastic ranges between points 0 and 2, depending on the performance level. Acceptance criteria for secondary elements that exhibit Type 1 behavior can be within any of the performance ranges. Acceptance criteria for primary and secondary components exhibiting Type 2 behavior will be within the elastic or plastic ranges, depending on the performance level. Acceptance criteria for primary and secondary components exhibiting Type 3 behavior will always be within the elastic range. Table 3-5 provides some examples of possible deformation- and force-controlled actions in common framing systems.

Table 3-5 Examples of Possible Deformation-Controlled and Force-Controlled Actions(FEMA-356, 2000)

Component	Deformation-Controlled Action	Force-Controlled Action
Moment Frames		
Beam	Moment (M)	Shear (V)
Columns	M	Axial load (P), V
Joints	-	V[1]
Shear Walls	M, V	P
Braced Frames		
Braces	P	--
Beams	--	P
Pier/pile	--	P
Shear Link	V	P, M
Connections	P, V, M[3]	P, V, M
Diaphragms	M, V[2]	P, V, M

1. Shear may be a deformation-controlled action in steel moment frame connection
2. If the diaphragm carries lateral loads from vertical seismic resisting elements above the diaphragm level, then M and V shall be considered force-controlled actions.

3.	Axial, shear, and moment may be deformation-controlled actions for certain steel and wood connections.

A given component may have a combination of both force- and deformation-controlled actions. Each element must be checked to determine whether its individual components satisfy acceptability requirements under performance point forces and deformations. Together with the global requirements, acceptability limits for individual components are the main criteria for assessing the calculated structure response.

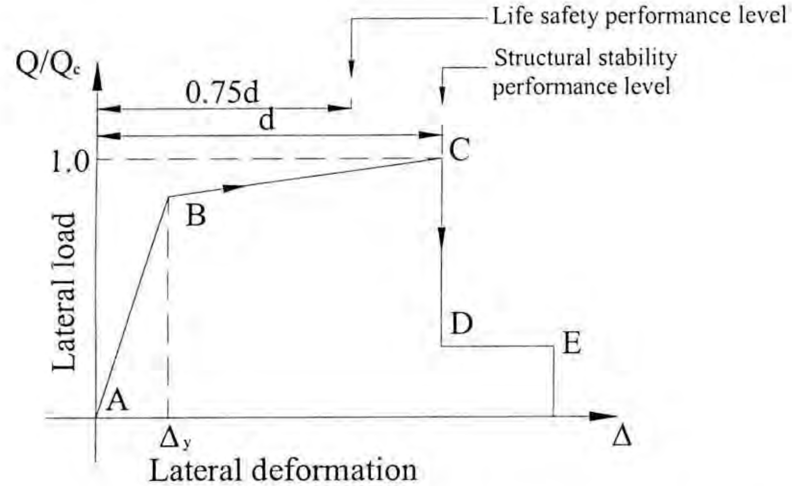


Figure 3.9 Force-deformation action and acceptance criteria

The Fig. 3.9 shows a generalized load – deformation relation appropriate for most concrete components. The relation is described by linear response from *A* (unloaded component) to an effective yield point *B*, linear response at reduced stiffness from *B* to *C*, sudden deduction in lateral load resistance to *D*, response at reduced resistance to *E*, and final loss of resistance thereafter. The following main points relate to the depicted load-deformation relation:

Point A corresponds to the unloaded condition. The analysis must recognize that gravity loads may induce initial forces and deformations that should be accounted for in the model. Therefore, lateral loading may commence at a point other than the origin of the load-deformation relation.

Point B has resistance equal to the nominal yield strength. The slope from B to C, ignoring the effects of gravity loads acting through lateral displacements, is usually taken as between 5% and 10% of the initial slope. This strain hardening, which is observed for most reinforce concrete component, may have an important effect on the redistribution of internal forces among adjacent components.

The abscissa at C corresponding to the deformation at which significant strength degradation begins.

The drop in resistance from C to D represents initial failure of the component. The residual resistance from D to E may be non-zero in some cases and may be effectively zero in others. Where specific information is not available, the residual resistance usually may be assumed to be equal to 20% of the nominal strength.

Point E is a point defining the maximum deformation capacity. Deformation beyond that limit is not permitted because gravity load can no longer be sustained.

Table 3.6 to Table 3.12 give the acceptance criteria for Nonlinear Procedures for the individual components [ATC-40, (1996)] used in prepare acceptance model of individual structural elements of a structure that is to be evaluated for finding seismic performance under this thesis.

Table 3-6 Numerical Acceptance Criteria for Plastic Hinge Rotations in Reinforced Concrete Beams, in radians [ATC-40, (1996)]

			Performance Level[3]				
			Primary			Secondary	
Component Type			**IO**	**LS**	**SS**	**LS**	**SS**
1. Beams Controlled by Flexure[1]							
$\dfrac{\rho - \rho'}{\rho_{bal}}$	Transverse Reinforcement[2]	$\dfrac{V}{b_w d\sqrt{f'_c}}$ [4]					
≤0.0	C	≤3	0.005	0.020	0.025	0.020	0.050
≤0.0	C	≥6	0.005	0.010	0.020	0.020	0.040
≥0.5	C	≤3	0.005	0.010	0.020	0.020	0.030
≥0.5	C	≥6	0.005	0.005	0.015	0.015	0.020
≤0.0	NC	≤3	0.005	0.010	0.020	0.020	0.030
≤0.0	NC	≥6	0.000	0.005	0.010	0.010	0.015
≥0.5	NC	≤3	0.005	0.010	0.010	0.010	0.015
≥0.5	NC	≥6	0.000	0.005	0.005	0.005	0.010
2. Beams controlled by shear[1]							
Stirrup spacing ≤ d/2			0.0	0.0	0.0	0.010	0.020
Stirrup spacing > d/2			0.0	0.0	0.0	0.005	0.010
3. Beams controlled by inadequate development or splicing along the span[1]							
Stirrup spacing ≤ d/2			0.0	0.0	0.0	0.010	0.020
Stirrup spacing > d/2			0.0	0.0	0.0	0.005	0.010
4. Beams controlled by inadequate embedment into beam-column joint[1]							
			0.0	0.01	0.015	0.020	0.030

1. When more than one of the conditions 1,2,3 and 4 occur for a given component, use the minimum appropriate numerical value from the table.

2. Under the heading "transverse reinforcement," 'C' and 'NC' are abbreviations for conforming and non-conforming details, respectively. A component is conforming if within the flexural plastic region: (1) closed stirrup are spaced at ≤d/3 and 2) for components of moderate and high ductility demand the strength provided by the stirrup (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered non-conforming.

3. Linear interpolation between values listed in the table is permitted.
   IO = Immediate Occupancy
   LS = Life Safety
   SS = Structural Stability

4. V = Design Shear

Table 3-7 Numerical Acceptance Criteria for Plastic Hinge Rotations in Reinforced Concrete Columns, in radians [ATC-40, (1996)]

			Performance Level[4]				
			Primary			Secondary	
Component Type			IO	LS	SS	LS	SS
1. Columns Controlled by Flexure[1]							
$\frac{P}{A_g f'_c}$ [5]	Transverse Reinforcement[2]	$\frac{V}{b_w d \sqrt{f'_c}}$ [6]					
≤0.1	C	≤3	0.005	0.010	0.020	0.015	0.030
≤0.1	C	≥6	0.005	0.010	0.015	0.010	0.025
≥0.4	C	≤3	0.000	0.005	0.015	0.010	0.025
≥0.4	C	≥6	0.000	0.005	0.010	0.010	0.015
≤0.1	NC	≤3	0.005	0.005	0.010	0.005	0.015
≤0.1	NC	≥6	0.005	0.005	0.005	0.005	0.005
≥0.4	NC	≤3	0.000	0.000	0.005	0.000	0.005
≥0.4	NC	≥6	0.000	0.000	0.000	0.000	0.000
2. Columns controlled by shear[1,3]							
Hoop spacing ≤d/2, or $\frac{P}{A_g f'_c} \leq 0.1$			0.000	0.000	0.000	0.010	0.015
Other cases			0.000	0.000	0.000	0.00	0.000
3. Columns controlled by inadequate development or splicing along the clear height[1,3]							
Hoop spacing ≤d/2			0.0	0.0	0.0	0.010	0.020
Hoop spacing >d/2			0.0	0.0	0.0	0.005	0.010
4. Columns with axial loads exceeding 0.70 [1,3]							
Conforming reinforcement over the entire length			0.0	0.0	0.005	0.005	0.010
All other cases			0.0	0.0	0.0	0.0	0.0

1. When more than one of the conditions 1,2,3 and 4 occur for a given component, use the minimum appropriate numerical value from the table. See Chapter 9 for symbol definitions.

2. Under the heading "transverse reinforcement," 'C' and 'NC' are abbreviations for conforming and non-conforming details, respectively. A component is conforming if within the flexural plastic hinge region: (1) closed hoops are spaced at ≤d/3 and 2) for components of moderate and high ductility demand the strength provided by the stirrup (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered non-conforming.

3. To qualify, (1) hoops must not be lap spliced in the cover concrete, and (2) hoops must have hooks embedded in the core or must have other details to ensure that hoops will be adequately anchored following spalling of cover concrete.

4. Linear interpolation between values listed in the table is permitted.
   IO = Immediate Occupancy
   LS = Life Safety
   SS = Structural Stability

5. P = Design axial load

6. V = Design shear force

Table 3-8 Numerical Acceptance Criteria for Chord Rotations for Reinforced Concrete Coupling Beams

Component Type		Performance Level[3] Primary IO	Primary LS	Primary SS	Secondary LS	Secondary SS
1. Coupling beams controlled by flexure						
Longitudinal reinforcement and transverse reinforcement[1]	$\frac{V}{b_w d\sqrt{f'_c}}$					
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤3	0.006	0.015	0.025	0.025	0.040
Conventional longitudinal reinforcement with conforming transverse reinforcement	≥6	0.005	0.010	0.015	0.015	0.030
Conventional longitudinal reinforcement with non-conforming transverse reinforcement	≤3	0.006	0.012	0.020	0.020	0.035
Conventional longitudinal reinforcement with non-conforming transverse reinforcement	≥6	0.005	0.008	0.010	0.010	0.025
Diagonal reinforcement	N/A	0.006	0.018	0.030	0.030	0.050
2. Coupling beams controlled by shear						
Longitudinal reinforcement and transverse reinforcement[1]	$\frac{V}{b_w d\sqrt{f'_c}}$ [2]					
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤3	0.006	0.012	0.015	0.015	0.024
Conventional longitudinal reinforcement with conforming transverse reinforcement	≥6	0.004	0.008	0.010	0.010	0.016
Conventional longitudinal reinforcement with non-conforming transverse reinforcement	≤3	0.006	0.008	0.010	0.010	0.020
Conventional longitudinal reinforcement with non-conforming transverse reinforcement	≥6	0.004	0.006	0.007	0.007	0.012

1. Conventional longitudinal steel consists of top and bottom steel parallel to the longitudinal axis of the beam. The requirements for conforming transverse reinforcement are: (1) closed stirrups are to be provided over the entire length of the beam at spacing not exceeding d/3; and (2) the strength provided by the stirrups (Vs) should be at least three-fourths of the design shear.

2. V = the design shear force on the coupling beam in pounds, b_w = the web width of the beam, d = the effective depth of the beam and f'_c = concrete compressive strength in psi.

3. Linear interpolation between values listed in the table is permitted.
IO = Immediate occupancy; LS = Life Safety; SS = Structural Stability

Table 3-9 Numerical Acceptance Criteria for Reinforced Concrete Column Axial Hinge [FEMA-356, (2000)]

	Plastic Deformation[1]				
	Primary			Secondary	
Component Type	IO	LS	SS	LS	SS
1. Braces in Tension	$7\Delta_T$	$9\Delta_T$	$11\Delta_T$	$11\Delta_T$	$13\Delta_T$

[1] Δ_T is the axial deformation at expected tensile yielding load.

Table 3-10 Numerical Acceptance Criteria for Total Shear Angle in Reinforced Concrete Beam-Columns Joints, in radians [ATC-40, (1996)]

			Performance Level[4]				
			Primary[6]			**Secondary**	
Component Type			**IO**	**LS**	**SS**	**LS**	**SS**
1. Interior joints							
$\frac{P}{A_g f'_c}$ [2]	Transverse Reinforcement[1]	$\frac{V}{V_n}$ [3]					
≤0.1	C	≤1.2	0.0	0.0	0.0	0.020	0.030
≤0.1	C	≥1.5	0.0	0.0	0.0	0.015	0.020
≥0.4	C	≤1.2	0.0	0.0	0.0	0.015	0.025
≥0.4	C	≥1.5	0.0	0.0	0.0	0.015	0.020
≤0.1	NC	≤1.2	0.0	0.0	0.0	0.015	0.020
≤0.1	NC	≥1.5	0.0	0.0	0.0	0.010	0.015
≥0.4	NC	≤1.2	0.0	0.0	0.0	0.010	0.015
≥0.4	NC	≥1.5	0.0	0.0	0.0	0.010	0.015
2. Other joints							
$\frac{P}{A_g f'_c}$ [2]	Transverse Reinforcement[1]	$\frac{V}{V_n}$ [3]					
≤0.1	C	≤1.2	0.0	0.0	0.0	0.015	0.020
≤0.1	C	≥1.5	0.0	0.0	0.0	0.010	0.015
≥0.4	C	≤1.2	0.0	0.0	0.0	0.015	0.020
≥0.4	C	≥1.5	0.0	0.0	0.0	0.010	0.015
≤0.1	NC	≤1.2	0.0	0.0	0.0	0.005	0.010
≤0.1	NC	≥1.5	0.0	0.0	0.0	0.005	0.010
≥0.4	NC	≤1.2	0.0	0.0	0.0	0.000	0.000
≥0.4	NC	≥1.5	0.0	0.0	0.0	0.000	0.000

1. Under the heading "transverse reinforcement," 'C' and 'NC' are abbreviations for conforming and non-conforming details, respectively. A joint is conforming if closed hoops are spaced at $\leq h_c/3$ within the joint. Otherwise, the component is considered non-conforming. Also, to qualify as conforming details under condition 2, (1) closed hoops must not be lap spliced in the cover concrete, and (2) hoops must have hooks embedded in the core or must have other details to ensure that hoops will be adequately anchored following spalling of cover concrete.

2. The ratio $\frac{P}{A_g f'_c}$ is the ratio of the design axial force on the column above the joint to the product of the gross cross-sectional and lateral forces.

3. The ratio V/V_n is the ratio of the design shear force to the shear strength for the joint.

4. Linear interpolation between values listed in the table is permitted.

IO = Immediate Occupancy; LS = Life Safety; SS = Structural Stability

5. No inelastic deformation is permitted since joint yielding is not allowed in a conforming structure.

Table 3-11 Numerical Acceptance Criteria for Plastic Hinge Rotation in Reinforced Concrete Two-way Slabs and Slab-Column Connections, in radians [ATC-40, (1996)]

		Performance Level[4]				
		Primary			Secondary	
Component Type		**IO**	**LS**	**SS**	**LS**	**SS**
1. Slabs controlled by flexure and slab column connections[1]						
$\frac{V_g}{V_0}$ [2]	Continuity Reinforcement[3]					
≤ 0.2	Yes	0.01	0.015	0.02	0.030	0.05
≥ 0.4	Yes	0.00	0.000	0.00	0.030	0.04
≤ 0.2	No	0.01	0.015	0.02	0.015	0.02
≥ 0.4	No	0.00	0.000	0.00	0.000	0.00
2. Slabs controlled by inadequate development or splicing along the span[1]						
		0.00	0.00	0.000	0.01	0.02
3. Slabs controlled by inadequate embedment into slab-column joint[1]						
		0.01	0.01	0.015	0.02	0.03

1. When more than one of the conditions 1,2,3 and 4 occur for a given component, use the minimum appropriate numerical value from the table.

2. V_g = the gravity shear acting on the slab critical section as defined by ACI 318, V_o = the direct punching shear strength as defined by ACI 318.

3. Under the heading "Continuity reinforcement" assume 'Yes' where at least one of the main bottom bars in each direction is effectively continuous through the column cage. Where the slab is post-tensioned, assume "Yes" where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, assume "No."

4. Linear interpolation between values listed in the table is permitted.
   IO = Immediate Occupancy
   LS = Life Safety
   SS = Structural Stability

Table 3-12 Numerical Acceptance Criteria for Plastic Hinge Rotations in Reinforced Concrete Walls and Wall Segments Controlled by Flexure, in radians [ATC-40, (1996)]

			Performance Level[4]				
			Primary			Secondary	
Component Type			IO	LS	SS	LS	SS
1. Walls and wall segments controlled by flexure							
$\dfrac{(A_S - A_S')f_y + P}{t_w l_w f_c'}$ [1]	$\dfrac{V}{t_w l_w \sqrt{f_c'}}$ [2]	Boundary Element[3]					
≤0.1	≤3	C	0.005	0.010	0.015	0.015	0.020
≤0.1	≥6	C	0.004	0.008	0.010	0.010	0.015
≥0.25	≤3	C	0.003	0.006	0.009	0.009	0.012
≥0.25	≥6	C	0.001	0.003	0.005	0.005	0.010
≤0.1	≤3	NC	0.002	0.004	0.008	0.008	0.015
≤0.1	≥6	NC	0.002	0.004	0.006	0.006	0.010
≥0.25	≤3	NC	0.001	0.002	0.003	0.003	0.005
≥0.25	≥6	NC	0.001	0.001	0.002	0.002	0.004

1. A_s = the cross-sectional area of longitudinal reinforcement in tension, A_s' = the cross-sectional area of longitudinal reinforcement in compression, f_y = yield stress of longitudinal reinforcement, P = axial force acting on the wall considering design load combinations, t_w = wall web thickness, l_w = wall length, and f'_c = concrete compressive strength.

2. V = the design shear force acting on the wall, and other variables are as defined above.

3. The term "C" indicates the boundary reinforcement effectively satisfies requirements of ACI 318. The term "NC" indicates the boundary requirements do not satisfy requirements of ACI 318.

4. Linear interpolation between values listed in the table is permitted.

IO = Immediate Occupancy

LS = Life Safety

SS = Structural Stability

Chapter 4

SEISMIC DEMAND AND THE BASIC MODELING PARAMETERS

4.1 Introduction

The performance or the damage strategy of a bridge against some specified seismic demand, response quantities from a static non-linear analysis is compared with the global deformation limits. The response limits falls into two categories:

Global acceptability limits: These response limits include requirements for the vertical load capacity, lateral load resistance, and lateral deformation. These are defined in Chapter 3.

Element and component acceptability limits: Each element (deck, pier or foundation) must be checked to determine if its components respond within acceptable limits.

Bridge performance objectives are checked against some predefined seismic demand. Seismic demand for a structure is totally site depended. For analysis, development of site dependent elastic response spectrum is needed. The Federal Emergency Management Agency [FEMA-356, (2000)] and Applied Technology Council [ATC-40, (1996)] has recommended standard procedure to establish seismic demand at a site. This procedure is followed in the subsequent analysis and is discussed next.

4.2 Seismic Demand

Earthquake is an uncertain phenomenon. It is not possible to predict the time and what intensity of earthquake that may hit during life of a structure. It is recognized that neither the complete protection against earthquake of all sizes is economically feasible nor design alone based on strength criteria is justified. Thus the earthquake design philosophy adopted in the codes accepts that:

Under minor but frequent shaking, the main members of the bridge that carry vertical and horizontal forces should not be damaged; however bridge parts that do not carry load may sustain repairable damage.

Under moderate but occasional shaking, the main members may sustain repairable damage, while the other parts of the bridge may be damaged such that they may even have to be replaced after the earthquake.

Under strong but rare shaking, the main members may sustain severe (even irreparable) damage, but the bridge should not collapse.

Severity of earthquakes as classified in ATC-40, 1996 is defined below.

a. The Serviceability Earthquake (SE)

The Serviceability Earthquake (SE) is defined probabilistically as the level of ground shaking that has a 50 percent chance of being exceeded in 50-year period. This level of earthquake ground shaking is typically about 0.5 times of the level of ground shaking of the Design Earthquake. The SE has a mean return period of approximately 75 years. Damage in the non-structural elements is expected during Serviceability Earthquake.

b. The Design Earthquake (DE)

The Design Earthquake (DE) is defined probabilistically as the level of ground shaking that has a 10 percent chance of being exceeded in a 50-year period. The DE represents an infrequent level of ground shaking that can occur during the life of the bridge. The DE has a mean return period of approximately 500 years. Minor repairable damage in the primary lateral load carrying system is expected during Design Earthquake. For secondary elements, the damage may be such that they require replacement.

c. The Maximum Earthquake (ME)

The Maximum Earthquake (ME) is defined deterministically as the maximum level of earthquake ground shaking which may ever be excepted at the bridge site within the known geologic frame work. In probabilistic terms, the ME has a return period of about 1,000 years. During Maximum Earthquake, bridges will be damaged beyond repairable limit but will not collapse.

4.2.1 Development of Elastic Site Response Spectra

Two parameters are used to construct elastic response spectra. These parameters are called Effective Peak Acceleration (EPA) and Effective Peak Velocity (EPV) but their significance is better understood if these are considered as normalizing factors for construction of smoothed elastic response spectra for ground motion of normal duration. For purpose of computing lateral force coefficient, EPA and EPV are replaced by dimensionless acceleration coefficient C_A and C_V, respectively. An elastic response spectrum, for each earthquake hazard level of interest is based on these seismic coefficients. These coefficients for a particular zone are dependent on the seismicity of the area, the proximity of the site to active seismic sources, and site soil profile characteristics.

4.2.1.1 Seismic zone

Bangladesh is divided into three seismic zones as per BNBC. The table below shows the values of zone coefficients of Bangladesh.

Table 4-1 Seismic Zone Factor Z

Zone	1	2	3
Z	0.075	0.15	0.25

4.2.1.2 Seismic Source Type

As per ATC-40 (1996), three types of seismic source may be defined and are given in Table 4-2.

Table 4-2 Seismic Source Type as per ATC-40, 1996

		Seismic Source Definition	
Seismic Source Type	Seismic Source Description	Maximum Moment Magnitude, M	Slip Rate, SR(mm/yr)
A	Faults that are capable to produce large magnitude events and which have a high rate of seismic activity	M ≥7.0	SR≥5
B	All faults other than types A and C	Not applicable	Not applicable
C	Faults that are not capable to producing large magnitude events and which have a high rate of seismic activity	M<6.5	SR<2

4.2.1.3 Near Source Factor

Currently data pertaining to the active faults close to Bangladesh is not available. It is not possible to estimate the seismic source distance from a specific site being considered in this research. But it may be safely assumed that all the sources are located at a distance more than 15 km and the Table 4.3 (ATC-40, 1996) may be used to consider the Near-Source effects for the present..

Table 4-3 Seismic Source Factor

Seismic Source Type	Closed Distance to Known Seismic Source							
	≤2km		5 km		10 km		≥15 km	
	N_A	N_V	N_A	N_V	N_A	N_V	N_A	N_V
A	1.5	2.0	1.2	1.6	1.0	1.2	1.0	1.0
B	1.3	1.6	1.0	1.2	1.0	1.0	1.0	1.0
C	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

1. The near-source factor may be used on the linear interpolation of values for distance other than those shown in the table.

2. The closest distance of the seismic source shall be taken as the minimum distance between the site and the area described by the vertical projecting of source on the surface (i.e., surface projection of fault plane). The surface projecting need not include portions of the source a depths of 10km or greater. The largest value of the near-source factor considering all sources shall be used for design.

4.2.1.4 Seismic Coefficients

For each earthquake hazard level, the structure is assigned a seismic coefficient C_A in accordance Table 4.4 (ATC-40, 1996) and a seismic coefficient C_V in accordance with Table 4.5 (ATC-40, 1996). Seismic coefficient C_A represents the effective peak acceleration (EPA) of the ground. A factor of about 2.5 times C_A represents the average value of peak response of a 5 percent-damped short-period system in the acceleration domain.

The seismic coefficient C_V represents 5 percent-damped response of a 1-second system. C_V divided by period (T) defines acceleration response in the velocity domain. These coefficients are dependent on soil profile type and the product of earthquake zoning coefficient-Z, severity of earthquake-E and near source factor-N (ZEN). The soil profile types are classified in Table 4-6.

Table 4-4 Seismic Coefficient C_A

Soil Profile Type	Shaking Intensity, ZEN[1,2]			
	= 0.075	**= 0.15**	**= 0.20**	**= 0.30**
S_B	0.08	0.15	0.20	0.30
S_C	0.09	0.18	0.24	0.33
S_D	0.12	0.22	0.28	0.36
S_E	0.19	0.30	0.34	0.36
S_F	Site-specific geo-technical investigation required to determine C_A			

1. The value of E used to determine the product, ZEN, should be taken to be equal to 0.5 for the serviceability Earthquake, 1.0 for the Design Earthquake, and 1.25 for the Maximum Earthquake.
2. Seismic coefficient C_A should be determined by linear interpolation for values of the product ZEN other than those shown in the table.

Table 4-5 Seismic Coefficient C_V [ATC-40, (1996)]

Soil Profile Type	Shaking Intensity, ZEN			
	= 0.075	**= 0.15**	**= 0.20**	**= 0.30**
S_B	0.08	0.15	0.20	0.30
S_C	0.13	0.25	0.32	0.45
S_D	0.18	0.32	0.40	0.54
S_E	0.26	0.50	0.64	0.84
S_F	Site-specific geo-technical investigation required to determine C_V			

1. The value of E used to determine the product, ZEN, should be taken to be equal to 0.5 for the serviceability Earthquake, 1.0 for the Design Earthquake, and 1.25 for the Maximum Earthquake.

2. Seismic coefficient C_V should be determined by linear interpolation for values of the product ZEN other than those shown in the table.

Table 4-6 Soil Profile Types [ATC-40, (1996)]

		Average Soil Properties for Top 100 ft of Soil Profile		
Soil Profile Type	Soil Profile Name/Generic Description	Share Wave Velocity, Vs(ft/sec)	Standard Penetration Test, N or N_{CH} for cohesionless soil layers(blow/ft)	Undrained Shear Strength, S_u(psf)
S^1_A	Hard Rock	$V_S > 5,000$	Not Applicable	
S_B	Rock	$2,500 < V_S \leq 5,000$	Not Applicable	
S_C	Very Dense Soil and Rock	$1,200 < V_S \leq 2,500$	$N > 50$	$S_U > 2000$
S_D	Stiff Soil Profile	$600 < V_S \leq 1,200$	$15 \leq N \leq 50$	$1000 \leq S_U \leq 2000$
S^2_E	Soft Soil Profile	$V_S < 600$	$N < 50$	$S_U < 1,000$
S_F	Soil Requiring Site-Specific Evaluation			

1. Soil profile SA is not applicable to site in Dhaka.
2. Soil profile type S_E also include any soil profile with more than 10 feet or soft clay defined as a soil with PI > 20, $W_{MC} \geq 40$ and $S_u < 500$ psf..

4.3 Establishing Demand Spectra

For the purpose of subsequent analysis to be made in this thesis, it is necessary to establish an earthquake demand spectra against which bridge performance will be evaluated. There will be three such demand spectra for the three bridge sites under study.

4.4 Element Hinge Property

It is known that reinforced concrete does not respond elastically to load level about half the ultimate value. When an element is stressed beyond its elastic limit, the element will continue to deform disproportionate to its load, this process is called formation of plastic hinge. Hinge properties of RC members under different loading conditions are different. These are discussed in the next sections.

4.4.1 Concrete Axial Hinge

Concrete axial hinge is formed when the axial load carrying capacity of a section exceeds its elastic limit. The elastic limit for axial capacity is different for tension and compression. The limits are explained in Figure 4.1 referring to Figure 3.9 of Chapter 3

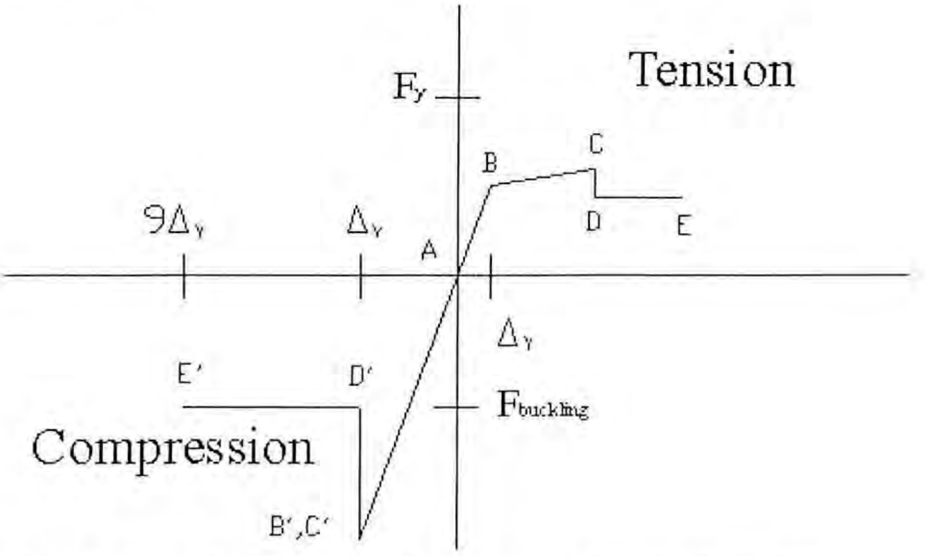


Figure 4.1 Concrete axial hinge property (FEMA-356, 2000)

Axial hinge features used in analysis are explained below:

- $P_y = A_s f_y$
- $P_c = 0.85\ A_c f'_c$
- Slope between points B and C is taken as 10% total strain hardening
- Hinge length assumption for Δy is based on the full length
- Point B, C, D and E are based on recommendation of Federal Emergency Management Agency (Pre-standard and Commentary for the Seismic Rehabilitation of Building, FEMA-356 Table 5.8, Braces in Tension)
- Point B′ = P_c
- Point E′ taken as 9Δy

4.4.2 Concrete Moment Hinge and Concrete P-M-M Hinge

Concrete moment hinge is formed when the flexural moment carrying capacity of a section exceeds its elastic limit. The limits of flexural moment capacity and bi-axial moment with axial load are explained in the Figure 4-2. If hinge is formed by the action of flexural moment only and P-M-M hinge is the hinge formed by the combined action of axial force and biaxial moments. The points A, B, C, D and E is explained in Figure 3.9 of Chapter 3.

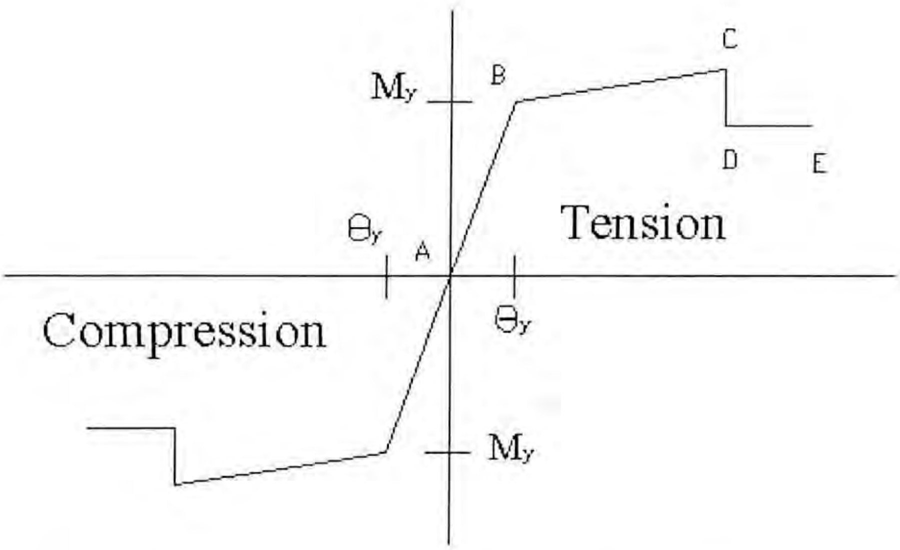


Figure 4.2 Concrete moment and P-M-M hinge property

P-M-M hinge features used in analysis are explained below:

- Slope between points B and C is taken as 10% of total strain hardening
- Points C, D and E based on the recommendation of Advance Technology Council [ATC-40, (1996)]
- M_y is based on reinforcement provided
- P-M-M curve is for major axis moment and is taken to be the same as the Moment curve in conjunction with the definition of Axial–Moment interaction curves.

4.4.3 Concrete Shear Hinge

Concrete shear hinge is formed when the flexural shear carrying capacity of a section exceeds its elastic limit. The elastic limit for flexural shear capacity for coupling beams controlled by flexure and controlled by shear is explained in Fig. 4.5 [ATC-40, (1996)].

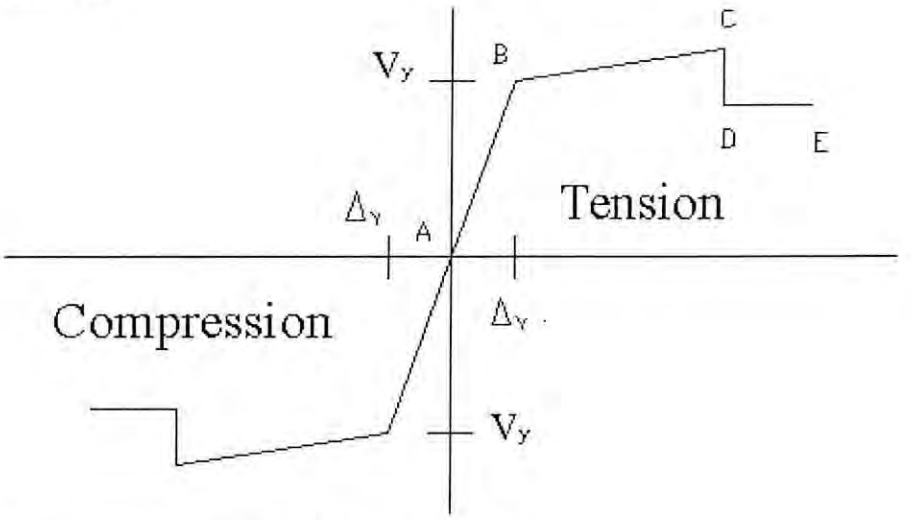


Figure 4.3 Concrete shear hinge property

Shear hinge features used in analysis:

- Slope between points B and C is taken such that shear at C is 10% higher than at B to represent strain hardening of steel
- $V_y = 2A_c\sqrt{(f'_c)} + f_y A_s v_d$

Points C, D and E are based on the recommendation of ATC-40, 1996.

4.5 Concrete Frame Acceptability Limits

To determine the performance objective of a structure, response quantities from a nonlinear static analysis are compared with limits for appropriate performance levels. Fig. 4.4 illustrates a generalized load-deformation relation applied to the structural components for this study. Curve Type I of the Figure 4.4 is used when the deformation is a flexural plastic hinge. Curve type II of the Figure- 4.4 is used when the deformation is inter-story drift, shear angle, sliding shear displacement, or beam-column joint rotation.

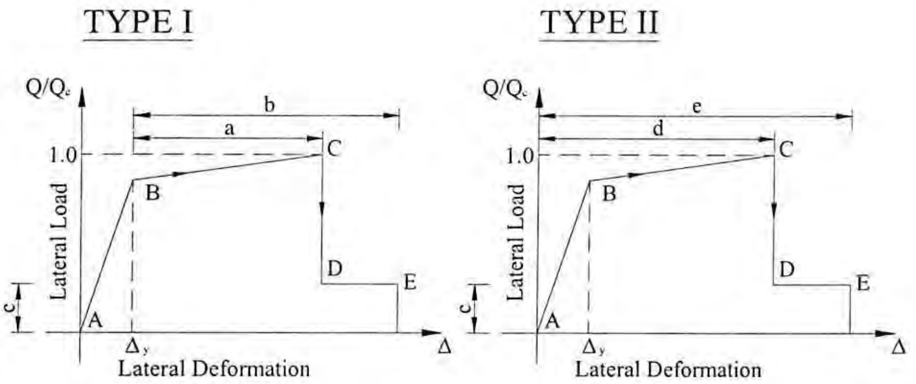


Figure 4.4 Generalized Load-Deformation Relations for Components

Tables 4-7 to 4-10 define the modeling parameter for beam and column in terms of plastic angles within the yielding plastic hinge.

Table 4-7 Modeling Parameters for Nonlinear Procedures – Reinforced Concrete Beams [ATC-40, (1996)]

			Modeling Parameters[3]		
			Plastic Rotation Angle, rad		Residual Strength Ratio
Component Type			a	b	c
1. Beam Controlled by Flexure[1]					
$\frac{\rho - \rho'}{\rho_{bal}}$	Transverse Reinforcement[2]	$\frac{V}{b_w d \sqrt{f'_c}}$ [4]			
≤0.0	C	≤3	0.025	0.05	0.2
≤0.0	C	≥6	0.02	0.04	0.2
≥0.5	C	≤3	0.02	0.03	0.2
≥0.5	C	≥6	0.015	0.02	0.2
≤0.0	NC	≤3	0.02	0.03	0.2
≤0.0	NC	≥6	0.01	0.015	0.2
≥0.5	NC	≤3	0.01	0.015	0.2
≥0.5	NC	≥6	0.005	0.01	0.2
2. Beams controlled by shear[1]					
Stirrup spacing ≤d/2			0.0	0.02	0.2
Stirrup spacing > d/2			0.0	0.01	0.2
3. Beams controlled by inadequate development or splicing along the span[1]					
Stirrup spacing ≤d/2			0.0	0.02	0.0
Stirrup spacing >d/2			0.0	0.01	0.0
4. Beams controlled by inadequate embedment into beam-column joint[1]					
			0.015	0.03	0.2

1. When more than one of the conditions 1,2,3 and 4 occur for a given component, use the minimum appropriate numerical value from the table.

2. Under the heading "transverse reinforcement," 'C' and 'NC' are abbreviations for conforming and non-conforming details, respectively. A component is conforming if within the flexural plastic region: (1) closed stirrup are spaced at ≤d/3 and 2) for components of moderate and high ductility demand the strength provided by the stirrup (Vs) is at least three-fourths of the design shear. Otherwise, the component is considered non-conforming.

3. Linear interpolation between values listed in the table is permitted

4. V = design shear force

Table 4-8 Modeling Parameters for Nonlinear Procedures – Reinforced Concrete Column [ATC-40, (1996)]

			Modeling Parameters[4]		
			Plastic Rotation Angle, rad.		Residual Strength Ratio
Component Type			a	b	c
1. Columns Controlled by Flexure[1]					
$\frac{P}{A_g f'_c}$ [5]	Transverse Reinforcement[2]	$\frac{V}{b_w d \sqrt{f'_c}}$ [6]			
≤0.1	C	≤3	0.02	0.03	0.2
≤0.1	C	≥6	0.015	0.025	0.2
≥0.4	C	≤3	0.015	0.025	0.2
≥0.4	C	≥6	0.01	0.015	0.2
≤0.1	NC	≤3	0.01	0.015	0.2
≤0.1	NC	≥6	0.005	0.005	-
≥0.4	NC	≤3	0.005	0.005	-
≥0.4	NC	≥6	0.0	0.0	-
2. Columns controlled by shear[1]					
Hoop spacing ≤ d/2 or $\frac{P}{A_g f'_c}$ [5] ≤ 0.1			0.0	0.015	0.2
Other cases			0.0	0.0	0.0
3. Columns controlled by inadequate development or splicing along the clear height[1,3]					
Hoop spacing ≤d/2			0.01	0.02	0.4
Hoop spacing >d/2			0.0	0.01	0.2
4. Column with axial loads exceeding 0.40 P_0[1,3]					
Conforming reinforcement over the entire length			0.015	0.025	0.02
All other cases			0.0	0.0	0.0

1. When more than one of the conditions 1,2,3 and 4 occur for a given component, use the minimum appropriate numerical value from the table.

2. Under the heading "transverse reinforcement," 'C' and 'NC' are abbreviations for conforming and non-conforming details, respectively. A component is conforming if within the flexural plastic hinge region: (1) closed hoops are spaced at ≤d/3 and 2) for components of moderate and high ductility demand the strength provided by the stirrup (Vs) is at least three-fourths of the design shear. Otherwise, the component is considered non-conforming.

3. To quality, (1) hoops must not be lap spliced in the cover concrete, and (2) hoops must have hooks embedded in the core or must have other details to ensure that hoops will be adequately anchored following spalling of cover concrete.

4. Linear interpolation between values listed in the table is permitted.

5. P = Design axial load

6. V = design shear force

Table 4-9 Modeling Parameters for Concrete Axial Hinge [FEMA-356, (2000)]

	Modeling Parameters[1]		
	Plastic Deformation		Residual Strength Ratio
Component Type	a	b	c
1. Braces in Tension	$11\Delta_T$	$14\Delta_T$	0.8

[1] Δ_T is the axial deformation at expected tensile yielding load.

Table 4-10 Modeling Parameters for Nonlinear Procedures-Coupling Beams [ATC-40, (1996)]

		Modeling Parameters[3]		
		Chord Rotation, rad		Residual Strength Ratio
Component Type		d	e	c
1. Coupling beams controlled by flexure				
Longitudinal reinforcement and transverse reinforcement[1]	$\frac{V}{b_w d \sqrt{f_c'}}$ [2]			
Conventional longitudinal reinforcement with	≤3	0.025	0.040	0.75
Conforming transverse reinforcement	≥6	0.015	0.030	0.50
Conventional longitudinal reinforcement with non-	≤3	0.020	0.035	0.50
Conforming transverse reinforcement	≥6	0.010	0.025	0.25
Diagonal reinforcement	N/A	0.030	0.050	0.80
2. Coupling beams controlled by shear				
Longitudinal reinforcement and transverse reinforcement[1]	$\frac{V}{b_w d \sqrt{f_c'}}$ [2]			
Conventional longitudinal reinforcement with	≤3	0.018	0.030	0.60
Conforming transverse reinforcement	≥6	0.012	0.020	0.30
Conventional longitudinal reinforcement with non-	≤3	0.012	0.025	0.40
Conforming transverse reinforcement	≥6	0.008	0.014	0.20

1. Conventional longitudinal steel consists of top and bottom steel parallel to the longitudinal axis of the beam. The requirements for conforming transverse reinforcement are: (1) closed stirrups are to be provided over the entire length of the beam at spacing not exceeding d/3; and (2) the strength provided by the stirrups (Vs) should be at least three-fourths of the design shear.

2. V = the design shear force on the coupling beam in pounds, b_w = the web width of the beam, d = the effective depth of the beam and f_c' = concrete compressive strength in psi.

3. Linear interpolation between values listed in the table is permitted.

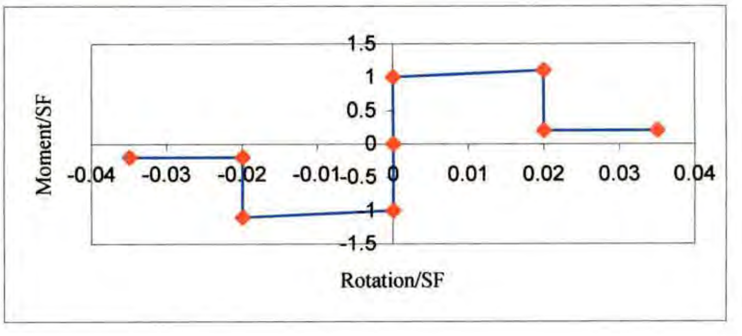
4.6 Hinge Properties for Modeling

Depending upon the longitudinal reinforcement, transverse reinforcement etc. different hinge properties may be modeled based on the modeling parameter defined through Table 4.7 to 4.10. Different points A, B, C etc. are defined in Figure 3.9 of Chapter 3. For the purpose of this study, following properties of concrete hinges have been assumed.

4.6.1 Reinforced Concrete Beams - M3 Hinge

M2 and M3 are the bending moments about two orthogonal axes perpendicular beam axis. Beams controlled by flexure having conforming transverse reinforcement shall have the following hinge properties. This is given in tabular form and is shown graphically as well.

Point	Moment/SF	Rotation/SF[1]
E	0.2	0.035
D	0.2	0.02
C	1.1	0.02
B	1	0
A	0	0
B'	-1	0
C'	-1.1	-0.02
D'	-0.2	-0.02
E'	-0.2	-0.035
Acceptance criteria[2]		
IO	LS	CP
0.005	0.011	0.020



[1] Average values of the four rows of conforming transverse reinforcement (Table 4.7).
[2] Average values of the four rows of conforming transverse reinforcement (Table 3.6).

Scale Factor (SF) is defined as yield moment/rotation or yield shear/displacement.

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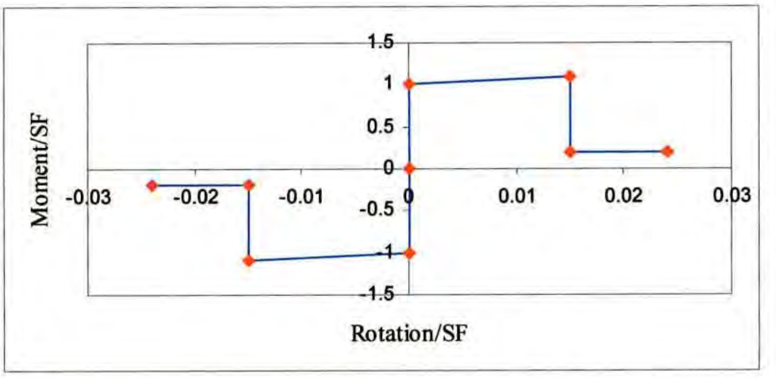
4.6.2 Reinforced Concrete Column - M2/M3 Hinge

Hinge properties for the columns controlled by flexure and conforming transverse reinforcement given in tabular form and is shown graphically also.

Point	Moment/SF	Rotation/SF[3]
E	0.2	0.024
D	0.2	0.015
C	1.1	0.015
B	1	0
A	0	0
B'	-1	0
C'	-1.1	-0.015
D'	-0.2	-0.015
E'	-0.2	-0.024

Acceptance criteria[4]

IO	LS	CP
0.0025	0.0075	0.0150



[3] Average values of the four rows of conforming transverse reinforcement (Table 4.8).
[4] Average values of the four rows of conforming transverse reinforcement (Table 3.7).

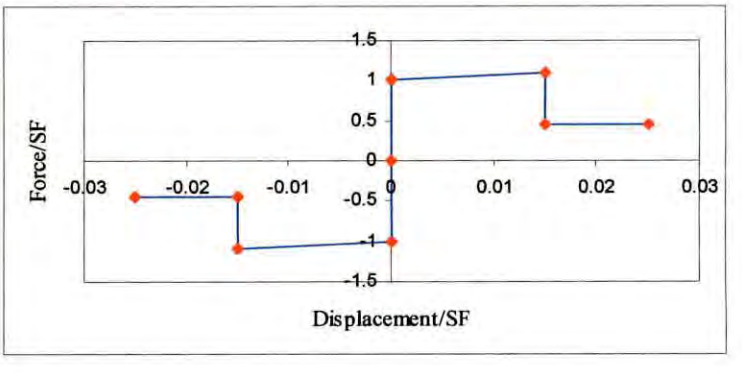
4.6.3 Reinforced Concrete Beams - Shear Hinge

The following tabulated hinge properties is recommended for coupling beams controlled by shear with conventional longitudinal reinforcement conforming transverse reinforcement.

Point	Force/SF	Displacement/SF[5]
E	0.45	0.025
D	0.45	0.015
C	1.1	0.015
B	1	0
A	0	0
B'	-1	0
C'	-1.1	-0.015
D'	-0.45	-0.015
E'	-0.45	-0.025

Acceptance criteria[6]

IO	LS	CP	
0.0050	0.0100	0.0125	1.5

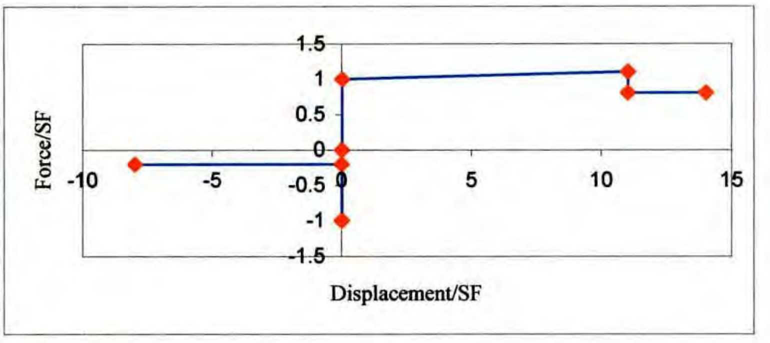


[5] Average values of the two rows of Item 2- Conforming transverse reinforcement (Table 4.10).
[6] Average values of the two rows of Item 2 -Conforming transverse reinforcement (Table 3.8).

4.6.4 Reinforced Concrete Column-Axial Hinge

The following hinge properties is recommended for the column controlled by axial load. This is given in tabular form and also shown graphically.

Point	Force/SF	Displacement/SF[7]
E	0.8	14
D	0.8	11
C	1.1	11
B	1	0
A	0	0
B'	-1	0
C'	-1	-0.01
D'	-0.2	-0.01
E'	-0.2	-8



Acceptance criteria[8]

	IO	LS	CP
+ve	0	7	9
-ve	0	-5	-7

[7] From Table 4.9

[8] From Table 3.9

Chapter 5

MODELING PARAMETERS FOR THE BRIDGES UNDER STUDY

5.1 Introduction

Bangladesh has been seismically divided into three zones. Three bridges are selected from those three zones. Geometric and site specific parameters are different in each bridge. These parameters are described below.

5.2 Shah Paran Bridge

5.2.1 Geometry

A photograph of the bridge is presented in Figure 5.1. The geometry and the properties of the bridge have been taken from approved as-built drawings. The bridge is 392.0m long and have a deck width of 14.0m. It is a two lane two way highway bridge. It is a 9 span simply supported bridge having span arrangement of 2x43.575m (end spans)+ 7x43.55m (intermediate spans).

Figure 5.1 Photographic view of Shah Paran Bridge

Its pier consists of 3 round RCC column of 1.0m dia founded on pile cap resting on 750mm dia cast-in-situ piles. The deck of the bridge is a composition of RCC deck slab and post tensioned pre-stressed girder.

The average cross sectional area of the composite deck is 4.65 m^2 on average. The height of the pier from pile cap top to the bottom pier cap varies from 5m to 11m. The cross section of pier cap is 2.0mx1.5m.and the bracing is 600mm by 1000mm Reinforcement in the deck is nearly 29400 mm^2 (1.2%), while for column it is 17220 mm^2 (2.1%). The reinforcement of pier cap and bracing is 10000 mm^2 and 6000 mm^2 respectively. The deck cross section and the cross section of pier is shown in Figure 5.2

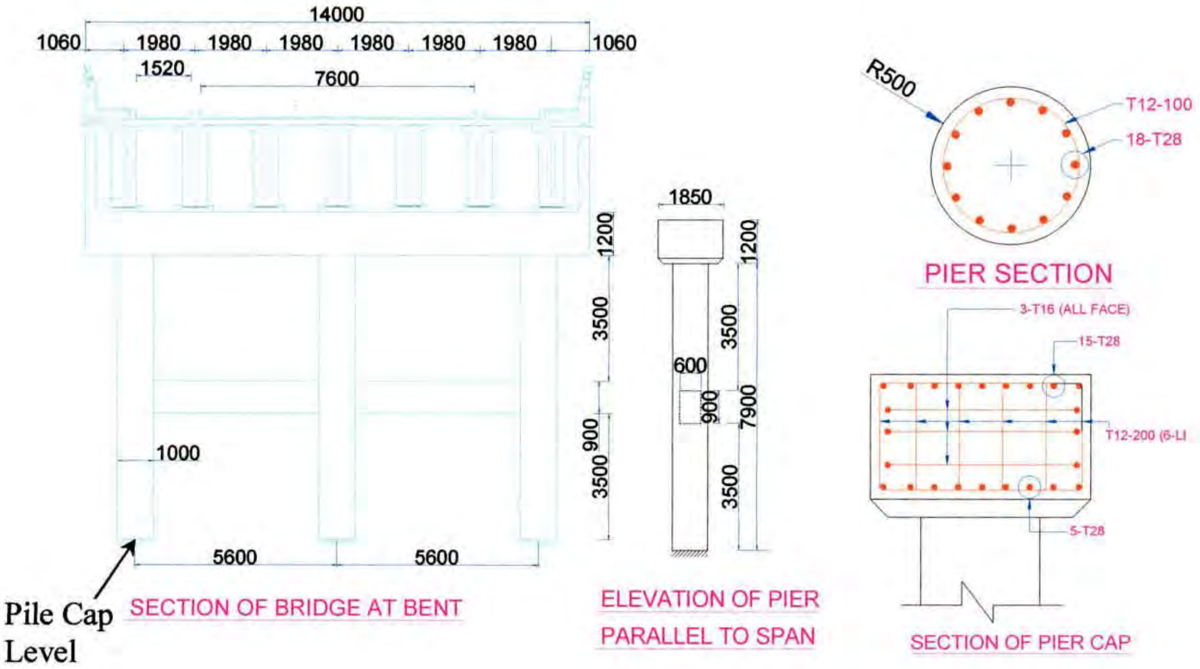


Figure 5.2 Deck and Pier Sections of Shah Paran Bridge

5.2.2 Site Specific Data and Calculation of C_A and C_V

Location of the site	:	Near Sylhet City
Soil profile at the site	:	Soil type S_D as per Table 4.6, stiff soil with shear wave velocity $600<V_S<1200$ ft/sec, $15<=N<=50$ and $1{,}000 \leq S_U \leq 2{,}000$ psf
Earthquake source type:		A – considering the events similar to the great Indian Earthquake in Assam in 12 June, 1897

Near Source Factor : > 15km

Calculation of C_A and C_V			
Seismic Zone Factor, Z	=	0.25	*as per BNBC*
Earthquake Hazard Level, E	=	1	*Design Earthquake*
Near-Source Factor, N	=	1	*>15km, Table 4.3*
Shaking Intensity, ZEN	=	0.25	
For Soil Type S_E, C_A	=	0.3	*From Table 4.4*
For Soil Type S_E, C_V	=	0.5	*From Table 4.5*

The following parameters as shown in Table 5.1 establishes 5% damped elastic response spectra and is graphically shown in Figure 5.3.

Table 5-1 Calculated parameters for constructing Response Spectrum for Shah Paran Bridge

Effective peak ground acceleration (EPA)	=	0.3	g	$= C_A$
Average value of peak response	=	0.75	g	$= 2.5C_A$
Seismic coefficient, C_V	=	0.5	g	$= C_V$
T_S	=	0.667	sec	$= C_V / 2.5C_A$
T_A	=	0.133	sec	$= 0.2T_S$

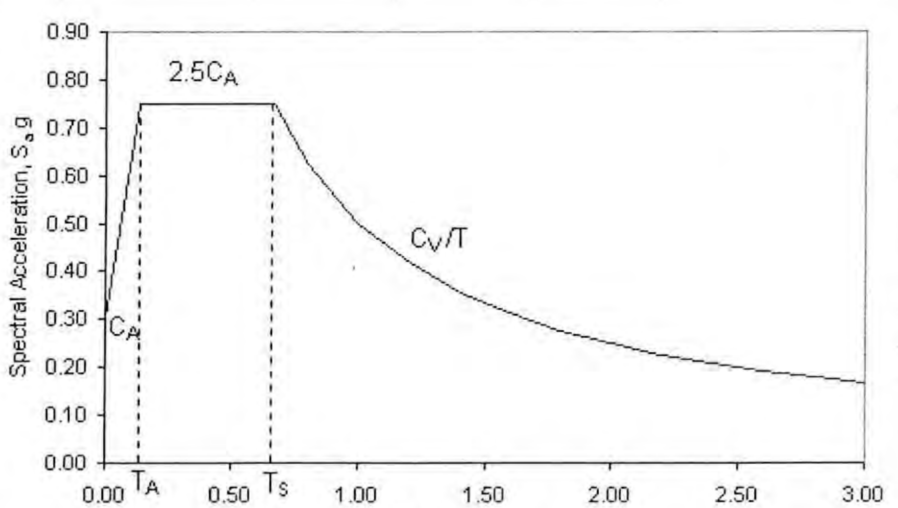


Figure 5.3 5% Elastic Response Spectrum for Shah Paran Bridge

For seismic performance evaluation purpose, this newly constructed site specific 5% elastic response spectra need to be converted in to ADRS format using relation, $S_d = \frac{T^2}{4\pi^2} S_a g$ and is represented in Figure 5.4.

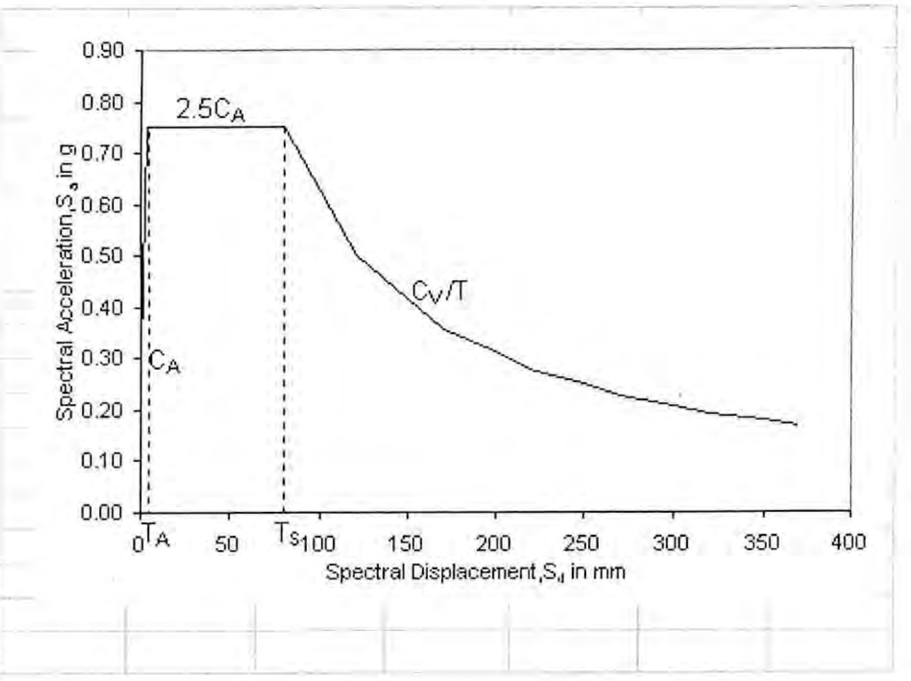


Figure 5.4 5% Elastic Response Spectrum in ADRS Format for Shah Paran Bridge

5.3 Meghna Bridge

5.3.1 Geometry

The geometry and the properties of the bridge have been taken from approved as-built drawings. The bridge is 930.0m long and have a deck width of 9.2 meter. It is a two lane two way highway bridge. It consists of 13 spans having span arrangement of 1x48.5m+9x87m+1x48.5m+1x25m+1x25m. Eleven spans were designed as cast-in-situ post tensioned progressive cantilever box girders and the two spans at one end designed as simply supported post tensioned PC girders supporting RC deck. Its piers consist of two types single unequal but symmetric hexagonal shaped RC column of average dimension of 5.35m x 2.7m dia supporting cantilever and 4m x 1.5m supporting composite deck founded on 12 number of 1.5m dia cast-in-situ piles. The average cross sectional area of the box girder 8.9 m^2 while that of simply supported span is 4.2 m2. A photograph of the bridge is given in Figure 5.5

Figure 5.5 Photographic view of Meghna Bridge

The height of the pier from pile cap top to the bottom of box section or pier cap varies from 21m to 33m. The average cross section of pier cap is 1.5mx1.5m provided in two end spans. Nominal reinforcement is provided in box section with the maximum HT strand of 85000mm^2 while 29400 mm^2 HT strand is provided in the two end spans. 128500 mm^2 (1.1%) reinforcement is provided in column. The reinforcement of pier cap is 10000 mm^2. The typical sectional of the bridge view is shown in Figure 5.6 and cross sections of deck and pier are shown in Figure 5.7

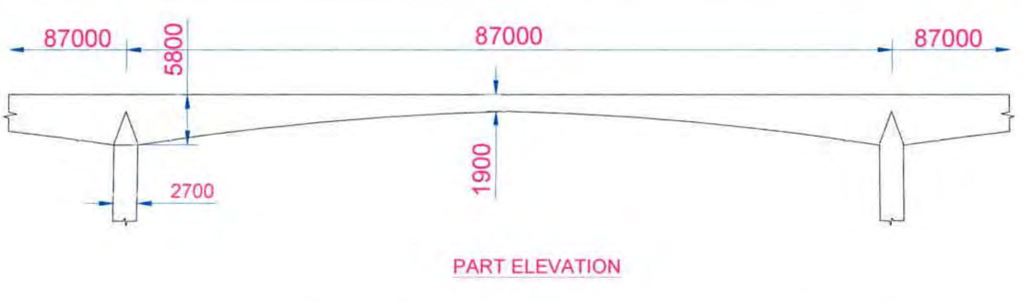


Figure 5.6 Typical Sectional View of Prestressed Box Girder of Meghna Bridge

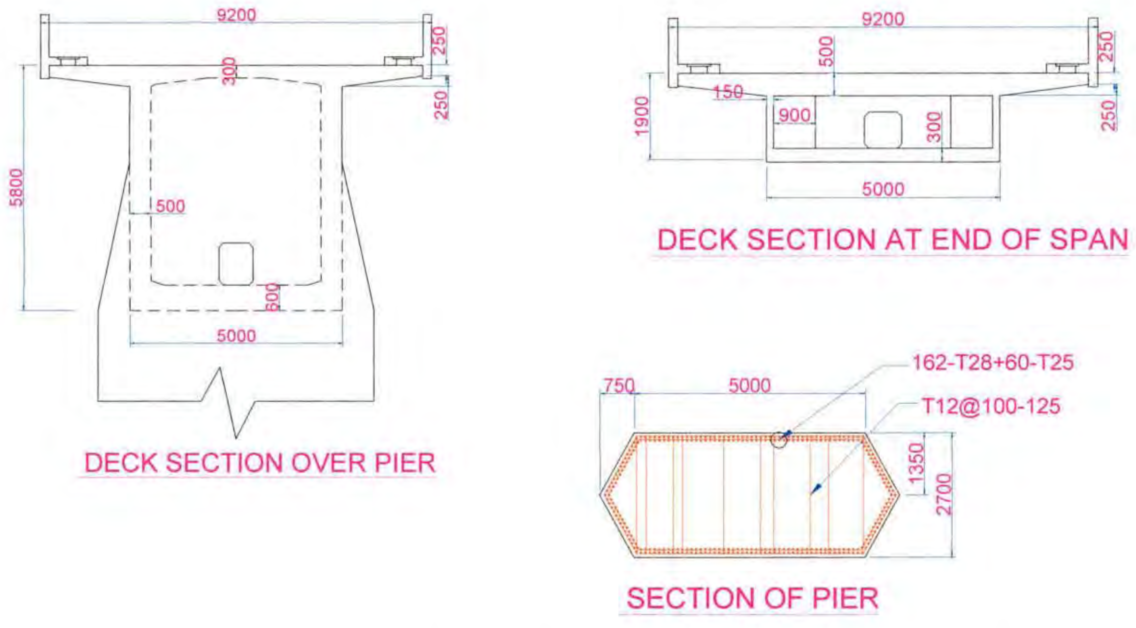


Figure 5.7 Cross section of deck and pier of Meghna Bridge

5.3.2 Site Specific Data and Calculation of C_A and C_V

Location of the site : Near Dhaka City

Soil profile at the site : Soil type S_E as per Table 4.6, soft soil profile with shear wave velocity V_S<600 ft/sec, N < 50 and S_u < 1000 psf

Earthquake source type: A – considering the events similar to the great Indian Earthquake in Assam in 12 June, 1897

Near Source Factor : > 15km

Calculation of C_A and C_V			
Seismic Zone Factor, Z	=	0.15	*as per BNBC*
Earthquake Hazard Level, E	=	1	*Design Earthquake*
Near-Source Factor, N	=	1	*>15km, Table 4.3*
Shaking Intensity, ZEN	=	0.15	
For Soil Type S_E, C_A	=	0.3	*From Table 4.4*
For Soil Type S_E, C_V	=	0.5	*From Table 4.5*

The following parameters as shown in Table 5-2 establishes 5% damped elastic response spectra and is graphically shown in Figure 5.8.

Table 5-2 Calculated parameters for constructing Response Spectrum for Meghna Bridge

Effective peak ground acceleration (EPA)	=	0.15 g	$= C_A$
Average value of peak response	=	0.750 g	$= 2.5C_A$
Seismic coefficient, C_V	=	0.26 g	$= C_V$
T_S	=	0.667 sec	$= C_V / 2.5C_A$
T_A	=	0.133 sec	$= 0.2T_S$

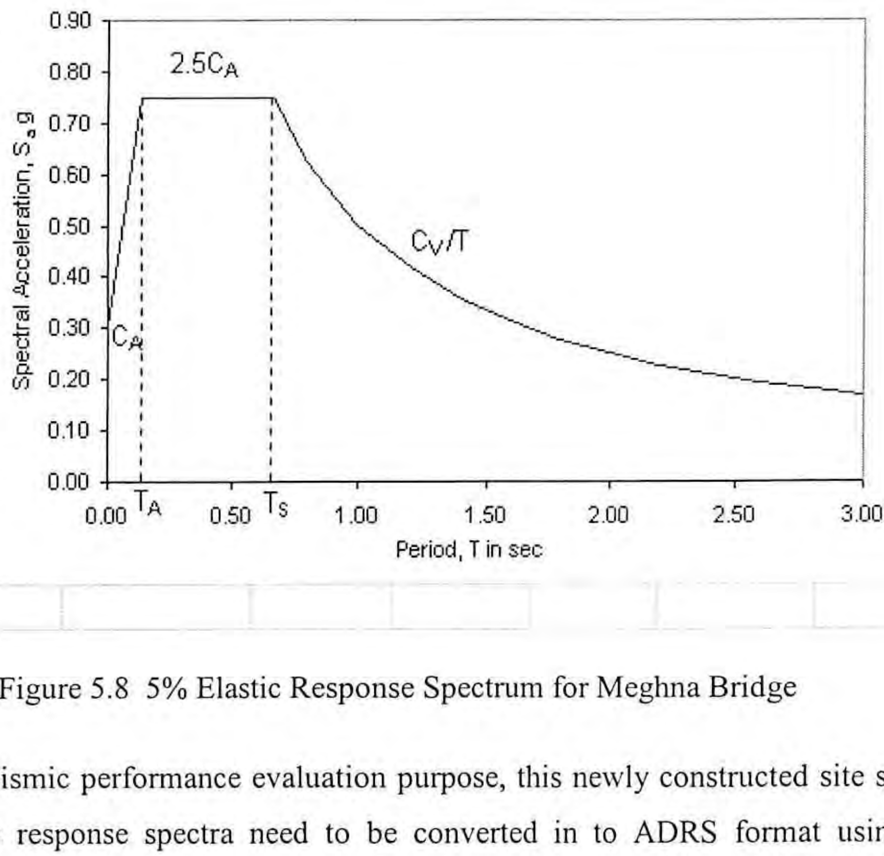


Figure 5.8 5% Elastic Response Spectrum for Meghna Bridge

For seismic performance evaluation purpose, this newly constructed site specific 5% elastic response spectra need to be converted in to ADRS format using relation, $S_d = \frac{T^2}{4\pi^2} S_a g$ and is represented in Figure 5.9.

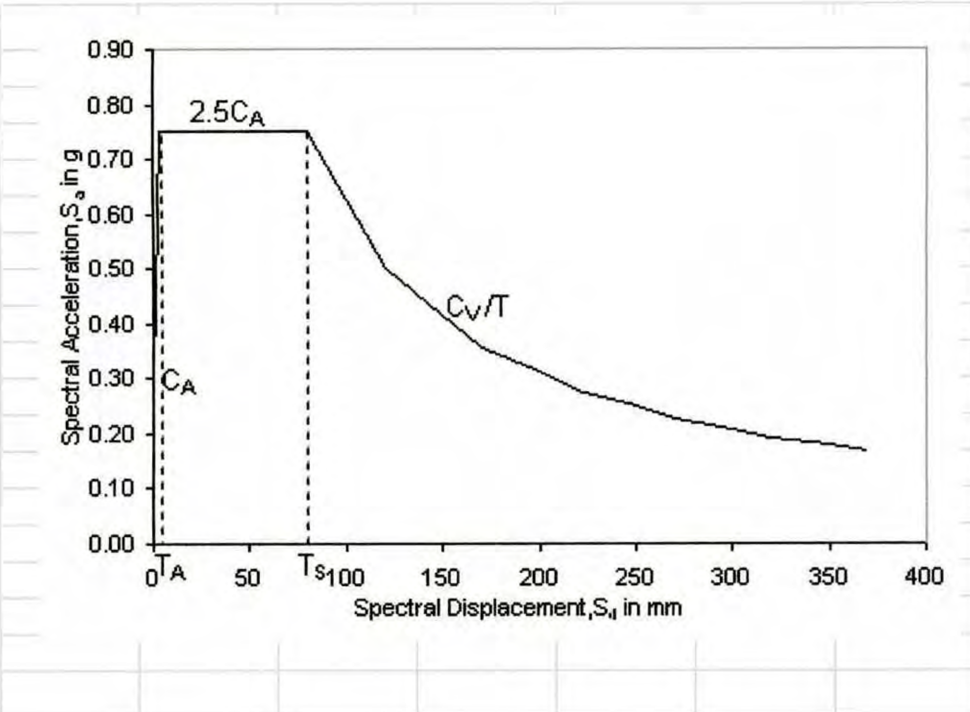


Figure 5.9 5% Elastic Response Spectrum in ADRS Format for Meghna Bridge

5.4 Doarika Bridge

5.4.1 Geometry and Properties

The geometry and the properties of the bridge have been taken from approved as-built drawings. The bridge is 388.0m long and have a deck width of 10.0m. It is a two lane two way highway bridge. A photograph of the bridge is presented in Figure 5.10.

Figure 5.10 Photographic view of Doarika Bridge

Its piers consist of single semi circular(oval shaped) RCC column of average dimension of 4.75m X 1.5m founded on 1.5m dia cast-in-situ piles. The deck of the bridge is a composition of RCC deck slab and post tensioned pre-stressed girder.

The average cross sectional area of the composite deck is 7.84 m^2 on average. The height of the pier from pile cap top to the bottom pier cap varies from 15m to 18m. The average cross section of pier cap is 2.0mx1.5m. HT strand amounting to 14200mm^2 is provided in post tensioned precast girders and the reinforcement in the deck is nearly 25000mm^2 (1.25%), 10250 mm^2 (1.5%) is provided in column. The reinforcement of pier cap is 12000 mm^2. The compressive strength of pier and deck concrete is 25 Mpa while girder concrete is 35 Mpa. The yield strength of all reinforcing bar is 413 Mpa and ultimate strength of HT strand is 1860 Mpa. The cross section of deck along with pier elevation is shown in Figure 5.11.

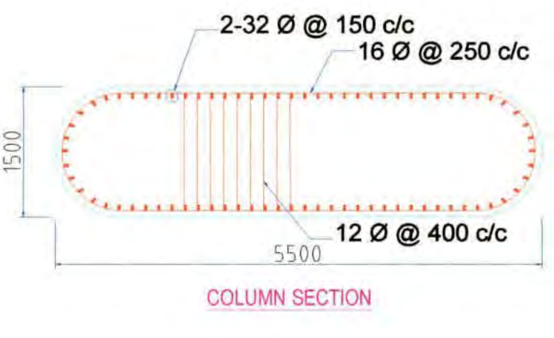


Figure 5.11 Elevation and cross section of Doarika Bridge

5.4.2 Site Specific Data and Calculation of C_A and C_V

Location of the site : At Doarika near Barisal City

Soil profile at the site : Soil type S_E as per Table 4.6, soft soil profile with shear wave velocity V_S<600 ft/sec, N < 50 and S_u < 1000 psf

Earthquake source type: A – considering the events similar to the great Indian Earthquake in Assam in 12 June, 1897

Near Source Factor : > 15km

Calculation of C_A and C_V			
Seismic Zone Factor, Z	=	0.075	*as per BNBC*
Earthquake Hazard Level, E	=	1	*Design Earthquake*
Near-Source Factor, N	=	1	*>15km, Table 4.3*
Shaking Intensity, ZEN	=	0.075	
For Soil Type S_E, C_A	=	0.19	*From Table 4.4*
For Soil Type S_E, C_V	=	0.26	*From Table 4.5*

The following parameters as shown in Table 5-3 establishes 5% damped elastic response spectra and is graphically shown in Figure 5.12.

Table 5-3 Calculated parameters for constructing Response Spectrum for Doarika Bridge

Effective peak ground acceleration (EPA)	=	0.19 *g*	= C_A
Average value of peak response	=	0.475 *g*	= $2.5C_A$
Seismic coefficient, C_V	=	0.26 *g*	= C_V
T_S	=	0.547 *sec*	= $C_V / 2.5C_A$
T_A	=	0.11 *sec*	= $0.2T_S$

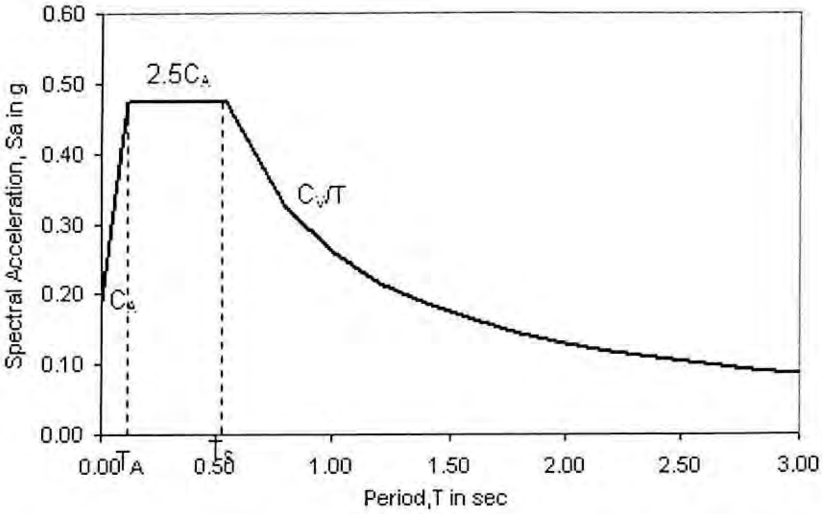


Figure 5.12 5% Elastic Response Spectrum for Doarika Bridge

For seismic performance evaluation purpose, this newly constructed site specific 5% elastic response spectra need to be converted in to ADRS format using relation, $S_d = \frac{T^2}{4\pi^2} S_a g$ and is represented in Figure 5-13

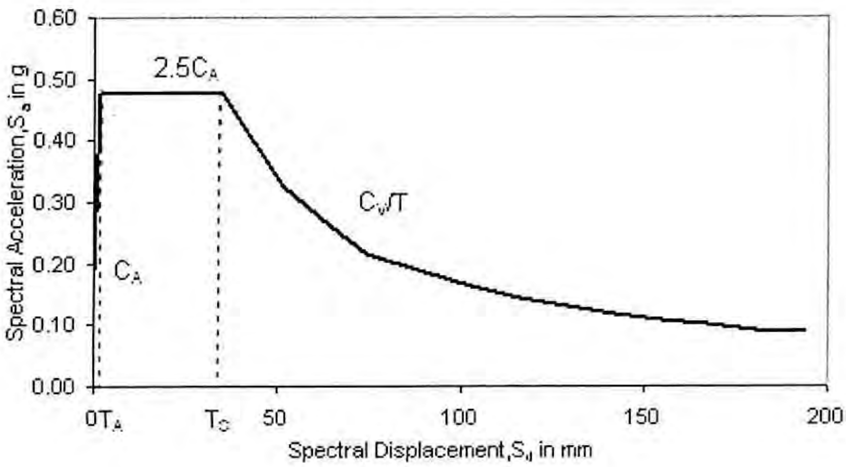


Figure 5.13 5% Elastic Response Spectrum in ADRS Format for Doarika Bridge

Chapter 6

ANALYSIS AND RESULTS

6.1 Introduction

In the previous chapters an outline of the procedure for structural performance evaluation in the light of ATC- 40, ATC-6-2 and FEMA 356 has been described. In this study three existing bridges are modeled using SAP 2000 package software. All these bridges were built by Roads and Highways Department. These are (i) Shah Paran Bridge in Sylhet (ii) Meghna Bridge in Comilla and (iii) Doarika Bridge in Barisal. Structural performances of these bridges have been investigated. These bridges have been modeled following the approved design drawing collected from the design consultants.

The performances of the bridges evaluated by pushover analysis have been presented as capacity curves and capacity spectrums are described below.

6.2 Assumption Pertaining to the Structures Under Analysis

Some assumptions are to be made to transfer a physical structure into a mathematical model. The bridges under study also require some simplifying assumptions for analytical solution.

6.2.1 Loading

The bridges under study were designed following AASHTO codes of specification. The designed sections are used to carryout seismic performance analysis following procedure described in SAP 2000. SAP 2000 established the analysis program following the guidelines of ATC and FEMA. Only loadings as stipulated by AASHTO responsible for creating seismic load on structure have been considered. This includes self weight and superimposed dead loads of the structure mainly. In addition 25% of AASHTO specified Live Load is also considered since all bridges are the major highway bridges. Geometry, material properties and design sections are taken from As-built drawings

6.2.2 Pushover Analysis Parameters

The following assumptions relate to the pushover analysis of the structure: Moment(M3) and Shear(V2) hinges are considered at the ends of cap beams and P-M-M and Shear hinges are considered at the ends of the piers. All hinges conform to ATC-40 guidelines which have been described in Chapter 4 along with the performance limits.

Pushover analysis has been done using load pattern resembling fundamental mode of vibration following equivalent static load as per AASHTO/1996. Geometric non-linearity (P-Δ effect) of the structure is considered with full dead loads and 25% of the live load.

- In each case study, the horizontal displacement of the top most node of the deck above bent has been selected for performance monitoring.
- The general-purpose finite element program SAP 2000 Nonlinear has been used to perform Non-linear Static Pushover analysis for evaluation of seismic performance of the bridges.

6.3 Components of Mathematical Modeling

Earthquake load is dynamic in nature. Earthquake shaking creates inertia force in the structure following Newton's Second Law of motion. The impact of this force is directly related to three components of the structure which are discussed below.

6.3.1 Mass

The mass of a bridge system, which contributes to the seismic response in the form of inertia forces, can be characterized by the weight, W of the moving portion of the bridge divided by the gravitational constant, g. Most of this mass is contributed by the bridge superstructure and is usually expressed as a distributed over the length of the bridge. The in-plane axial and flexural stiffness of a bridge superstructure are usually large enough to assume the superstructure a rigid body dynamic system. With this assumption, the entire bridge mass is concentrated in the superstructure and the mass of the pier is negligible. In the mathematical model the entire superstructure mass is thus considered lumped at the superstructure centroid around a pier in this study.

Out of the study bridges, superstructures of two bridges are supported by the piers through bearings. This bearing is modeled as link element whose properties are so chosen so that it represent the properties of the bearings.

6.3.2 Stiffness

The stiffness quantities are : i) modulus of elasticity, E; ii) second moment of inertia, I and iii) shear area, Av. The reinforced concrete is non-homogeneous material. But for simplicity in analysis a single material property is used without appreciable error. Geometric properties, I and Av, are, however considered incorporating post yield effect. Effective I and Av are used considering plastic hinge penetration length.

6.3.3 Damping

Usually three types of damping encountered by a bridge are i) Coulomb damping; ii) radiation damping and iii) hysteretic damping. Damping is a phenomenon of ability in dissipating energy i.e release of energy. Like other form of structure the most obvious form of damping or energy dissipation is in the form of hysteresis of force-deformation. This phenomenon is described in a little bit detail in the section that follow.

6.4 Analysis Strategy of the Structures

The structure has been modeled as described in Section 5.3.1.2. with the parameters defined in Section 5.2 and Chapter 4. The bridge is analyzed on the designed sections using SAP 2000. Hinges as defined in Chapter 4 have been assigned to the respective members and pushover analyses have been performed to develop capacity curves for each structure. The capacity curves have been transformed to capacity spectrums as per method described in Chapter 3. Site specific seismic demand for the structures has been established in Chapter 4. Accordingly performance points of the structures for the estimated seismic demand have been calculated. Resulting output for each structures are presented in the sections that follow. Hinges states near the performance point have been shown in color code. Different performance levels are determined as per Table 3.6 to Table 3.12 of Chapter 3. It is to recast that to find the performance of the structure for the specific seismic demand the 5% elastic demand needs to be scaled down by some factors to recognize the increase of damping due hysteretic damping for inelastic deformation.

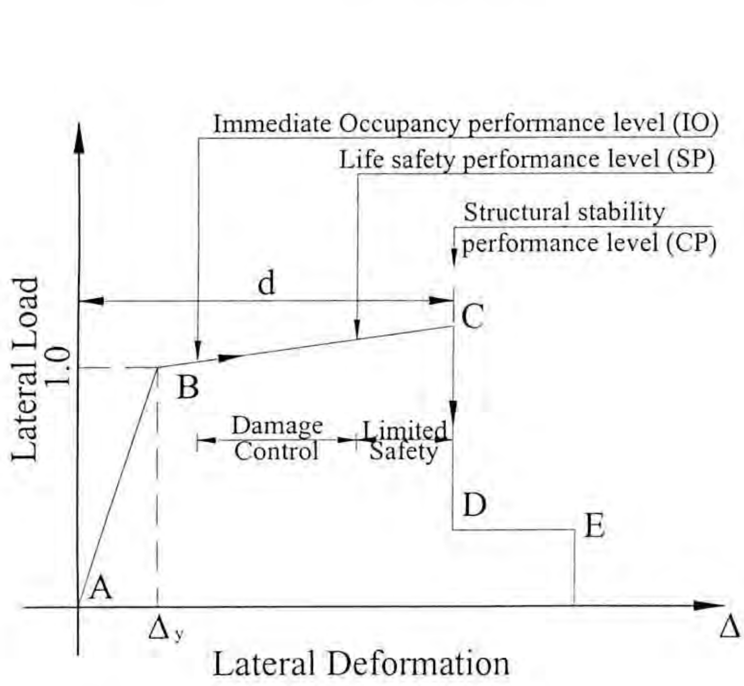


Figure 6.1 Typical load-deformation acceptance criteria (FEMA-356, 2000)

The equivalent viscous damping, β_{eq} resulting from viscous damping and hysteretic damping may be calculated by formula:

$\beta_{eq} = \beta_0 + 0.05$ ... ... (5.1)

Where,

β_0 = hysteretic damping represented as equivalent viscous damping

0.05 = 5% viscous damping inherent in the structure (assumed to be constant)

The term β_0 can be calculate as [Chopra, A.K. (1995)] :

$$\beta_{eq} = \frac{1}{4\pi}\frac{E_D}{E_{SO}} \quad \text{... ... ...} \tag{5.2}$$

Where,

E_D = energy dissipated by damping

E_{SO} = maximum strain energy

The physical significance of the terms E_D and E_{SO} is illustrated in Fig. 5.14.

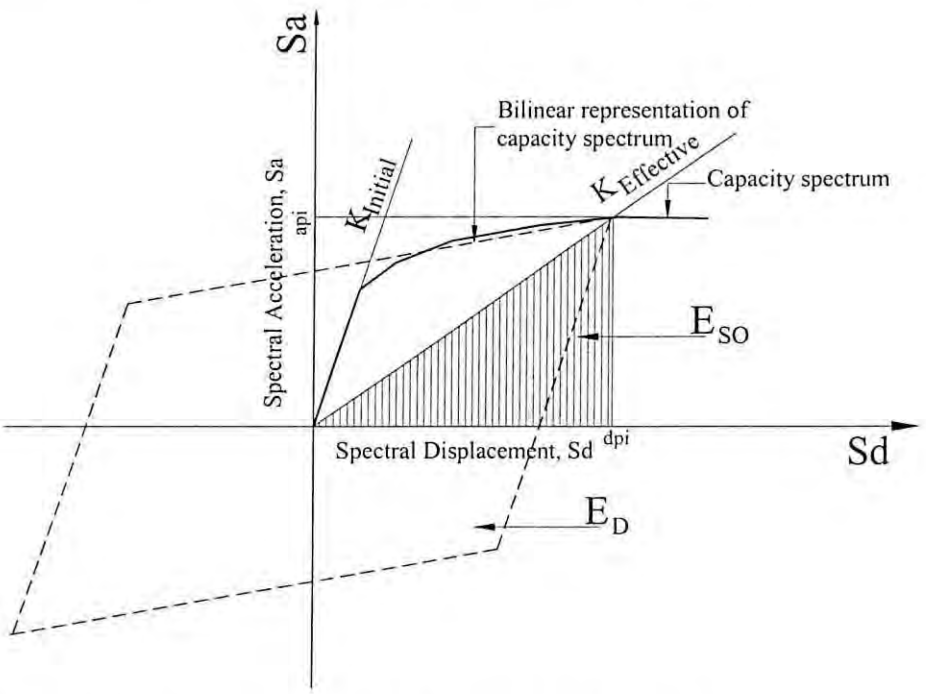


Figure 6.2 Derivation of damping for spectrum reduction

The equation for the reduction factors SR_A and SR_V are given by (ATC 40, 1996):

$$SR_A \approx \frac{3.21 - 0.681\ln(\beta_{eff})}{2.12} \tag{5.3}$$

$$SR_V \approx \frac{3.21 - 0.41\ln(\beta_{eff})}{1.65} \tag{5.4}$$

Where,

SR_A = reduction factor for short-period system in the acceleration domain

SR_V = reduction factor for long-period system in the velocity domain

The result of analysis in the form of graphical representation of the capacity spectrum, hinge stages at performance levels and performance point are shown in the sections that follow.

6.5 Modeling Limitations

i) Only upward component of effective prestress force is considered in the model and input as superimposed deal load. Axial component of prestress force is not taken into consideration as it does not contribute any in creating seismic force;

ii) Only AASHTO Lane Loading is considered because it is the governing Live Load ;

iii) Elastomeric bearing is modeled as an equivalent link;

iv) Piers are assumed as fixed at pile cap;

v) Full scale model for abutments is not done, only supports are modeled as equivalent

6.6 Analysis Domain and Results

The analysis is accomplished in three stages - i) construction of Capacity Curve which is a plot of base shear vs. deck displacement or displacement of pier cap top; ii) transposing of Capacity Curve into ADRS format; iii) identification of performance point following Capacity Spectrum Method(CSM), procedure B as guided by ATC 40. In simpler term, performance point represent earthquake demand i.e. a particular value of displacement caused by a particular magnitude of earthquake and described by drift ratio. On the basis of this ratio structural damage level can be determined using some arbitrarily titles like Immediate Occupancy (IO), Damage Control (DC) etc. already discussed in chapter 3. These results of the analysis are presented bridge wise in the sections that follow.

6.6.1 Shah Paran Bridge

Shah Paran Bridge is a simply supported nine span bridge. The superstructure rest on pier cap through elastomeric bearing. Over each pier there is an expansion joint

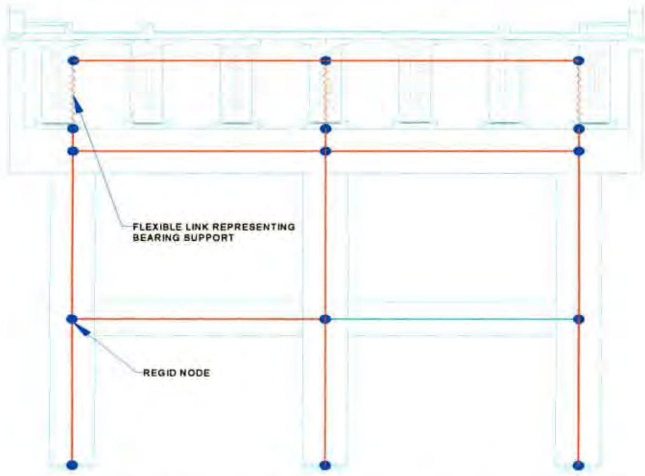


Figure 6.3 Skeleton of a Bridge Bent of Shah Paran Bridge

between the super structure which is free from restraint in all three axes. Deformation of deck top at the highest altitude includes the deformation of bearing itself. Deformation of bearing itself does not generate any reaction at the base of pier. So the base shear as found by analysis corresponds to the deformation of pier cap top which is thus selected as the monitoring node of mathematical model of the bridge.

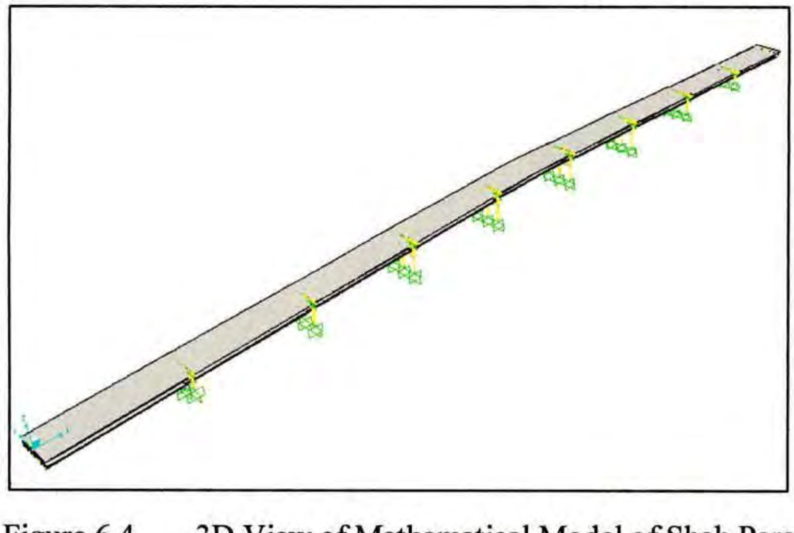


Figure 6.4 3D View of Mathematical Model of Shah Paran Bridge

Figure 6.3 represent a skeleton of a bent elements and Figure 6.4 shows 3D view of mathematical model of full bridge.

6.6.1.1 Capacity Curve of Shah Paran Bridge

Shah Paran Bridge Bents consist of three columns in the transverse direction

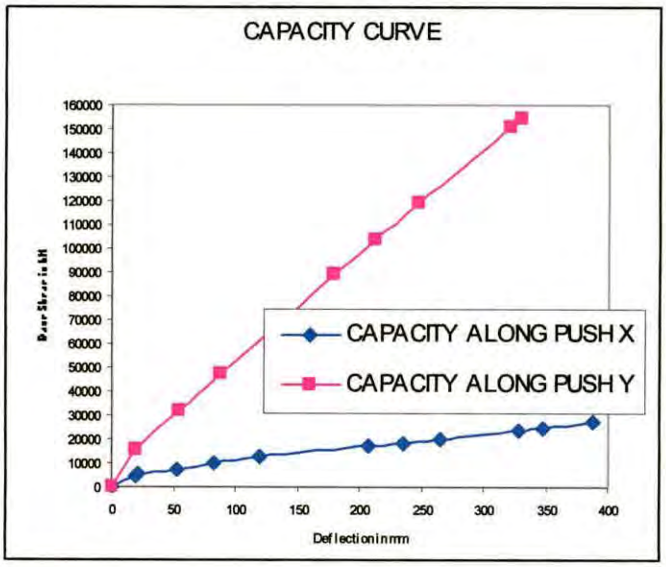


Figure 6.5 Capacity Curve for Push Load of Shah Paran Bridge

connected by a cap beam at top, a bracing at middle and columns are fixed at bottom with pile cap. In the longitudinal direction (X-Direction) the bent acts nearly a cantilever of three columns in a row. Due to framing action in transverse direction (Y-Direction), load resisting capacity is much more higher than that of longitudinal direction as evident from Capacity Curve (Figure 6.5). The Capacity Curve is a plot of deflection of deck (mid span) top vs. base shear or support reactions.

6.6.1.2 Hinge Status of Shah Paran Bridge near performance points when pushed in the longitudinal direction

Plastic hinges are formed in the piers when stress level exceed yield point. Different types of hinges are developed in conformity of hinge properties assigned at the joint at the time of modeling. Yield stress exceeded at 48 nodes out of 192 nodes when pushed in longitudinal direction. Different types of hinges are formed according the stress levels. Among the hinges formed, 24 nos. remain within Immediate Occupancy (IO) level, 16 nos. remain between IO and Life Safety (LS) level and 8 nos. hinges are within Collapse Prevention(CP) level. All the hinges of pier are formed at the joints with the pile cap.

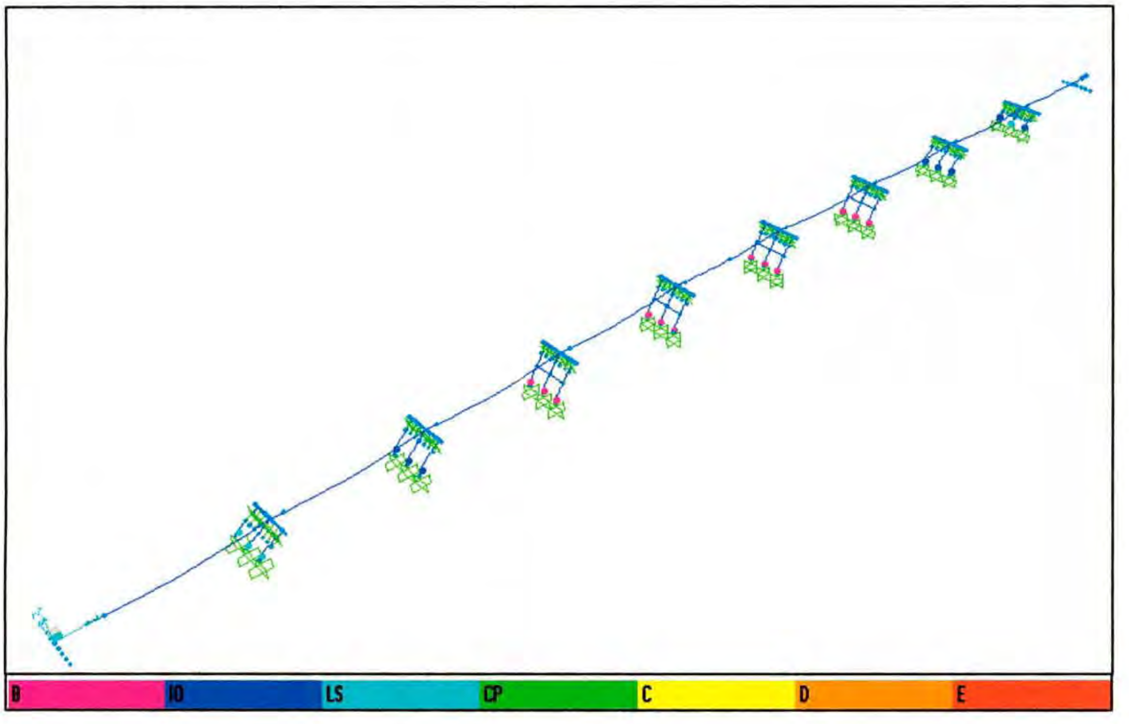


Figure 6.6 Hinges Types formed in the Piers of Shah Paran Bridge at Performance Points when pushed along bridge length

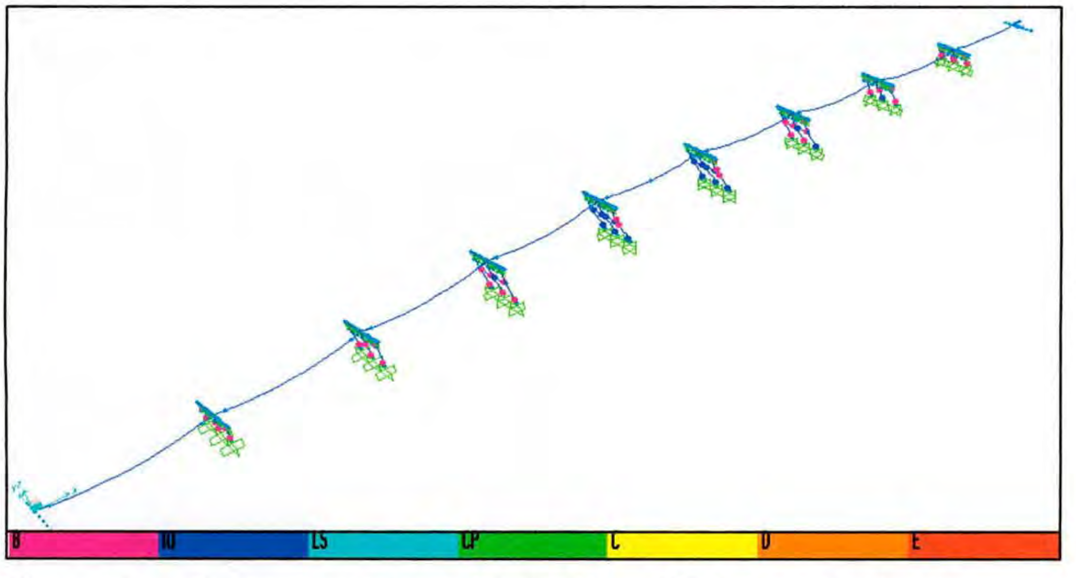


Figure 6.7 Hinges Types Formed in the Piers of Shah Paran Bridge at Performance Points in the transverse direction of the bridge

is pushed in the long direction. Yield stress exceeded at 82 nodes out of 192 nodes when pushed in transverse direction. Different types of hinges are formed according the stress levels.

Among the hinges formed, 74 nos. remain within Immediate Occupancy(IO) level, 4 nos. remain between IO and Life Safety (LS) and 4 nos. hinges are within Collapse Prevention(CP) level. In this case, hinges are formed in the bracings and cap beams in addition to piers because framing action of pier, cap beam and bracings. Figure 6.7 shows the hinge types formed at different location of piers, bracing and cap beams when pushed in the transverse direction.

6.6.1.3 Graphical Representation of performance level along the bridge length of Shah Paran Bridge

Figure 6-8 shows the result graphical representation of performance point when pushed in the longitudinal direction. At the performance point the spectral demand is 227mm. At this demand, the base shear value is about 18000 kN and the corresponding effective damping is 15.4%.

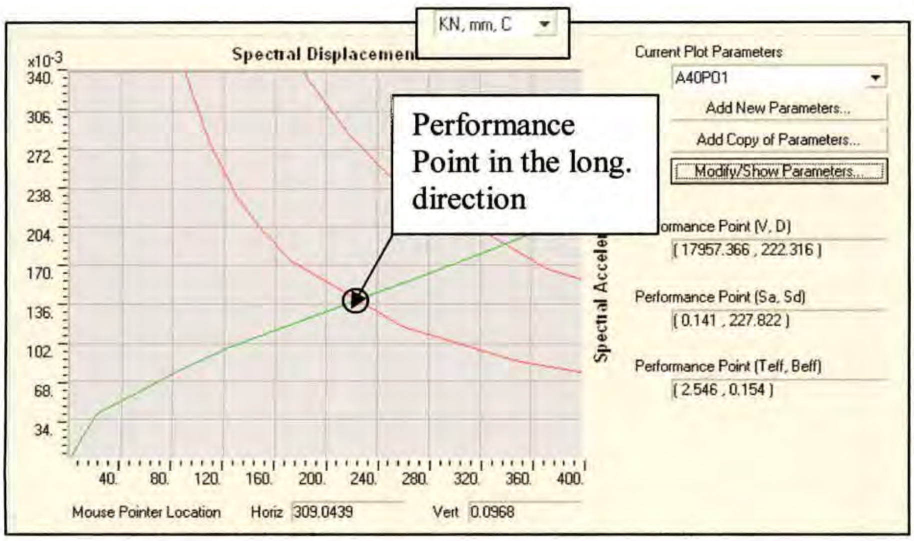


Figure 6.8 Capacity and Demand Spectrum of Shah Paran Bridge in the direction of the bridge length

and effective period is 2.54 sec. Figure 6-9 shows the result graphical representation of performance point when pushed in transverse direction. At the performance point spectral demand is 78mm. At this demand, the base shear become 54227 kN. The effective damping is 9% and the effective period is 0.74 sec at this performance point. The performance objective met by this performance point in both the direction is determined and is discussed in the next chapter.

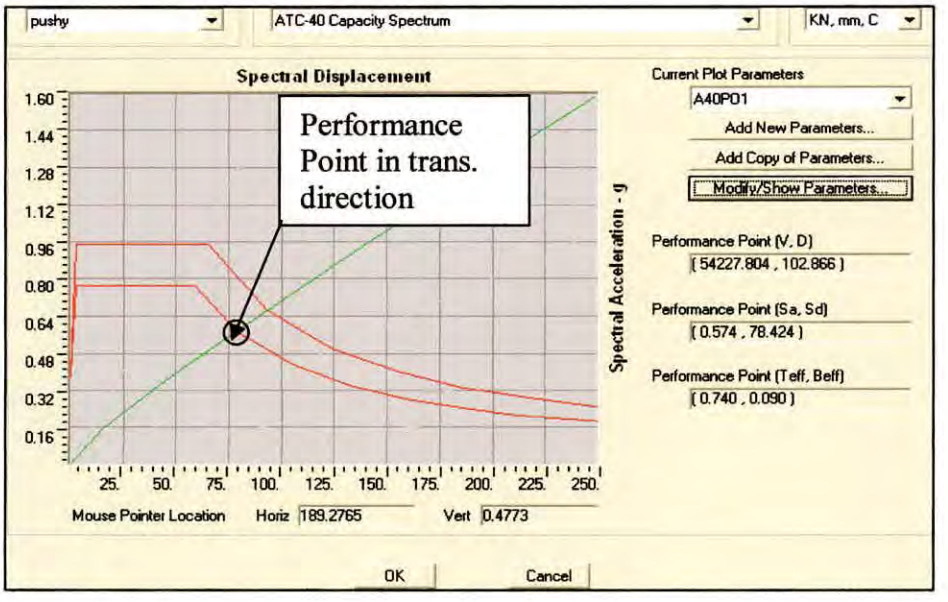


Figure 6.9 Capacity and Demand Spectrum of Shah Paran Bridge in the transverse direction of the bridge

6.6.1.4 Graphical Representation of performance level along both axes of Shah Paran Bridge

Figure 6.10 shows performance points in both the longitudinal and transverse

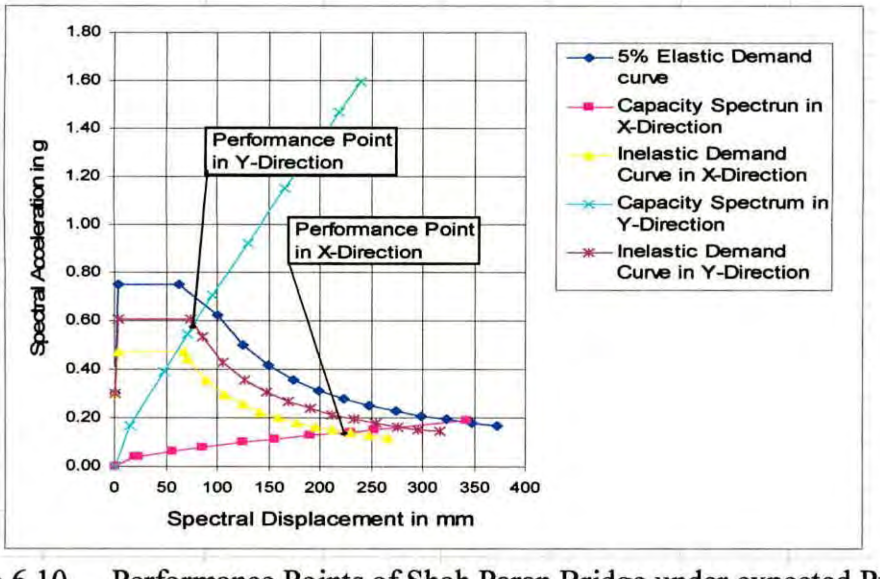


Figure 6.10 Performance Points of Shah Paran Bridge under expected Push Load

directions in the same plot. In the plot, X- Direction is the direction of span and Y-Direction is the transverse direction. From the plot it is clear that the demand in the longitudinal direction is much more than that of transverse direction. So the bridge is seismically stronger in the transverse direction and less prone to damage.

6.6.2 Meghna Bridge

A combination of nine cantilever, two propped cantilever and two simply supported spans form Meghna Bridge. The two cantilever box girders of length 43.5m supported by two piers constitute cantilever span. These two cantilevers are separated by an expansion joint having provision of total movement of 120mm. The span length of propped cantilever is 48.5m. One of these spans is the starting span (Dhaka End) and another span is the 11th span. The 12th and 13th spans are the simply supported spans of length 25m.

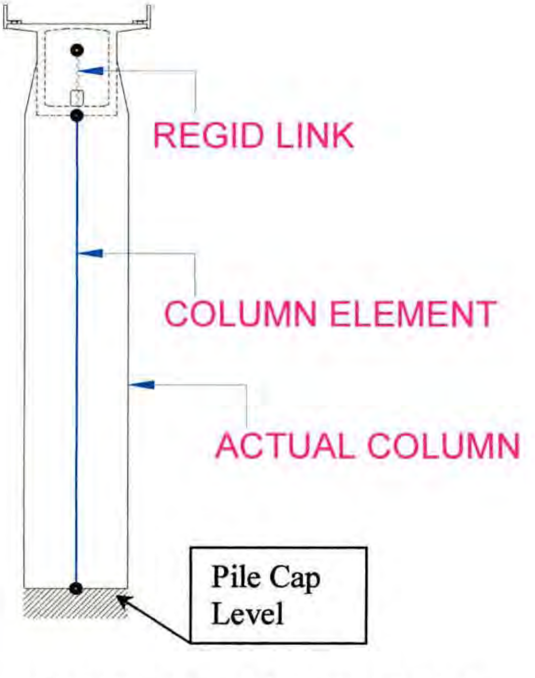


Figure 6.11 Skeleton of a Bridge Bent of Meghna Bridge

Deck top at highest altitude is the observation node for monitoring displacement required to accomplish push over analysis. So base shear as found by analysis corresponds the deformation of this node in the mathematical model of the bridge. Figure 6-11 represent a skeleton of a bent elements and Figure 6-12 shows 3D view of mathematical model for full bridge.

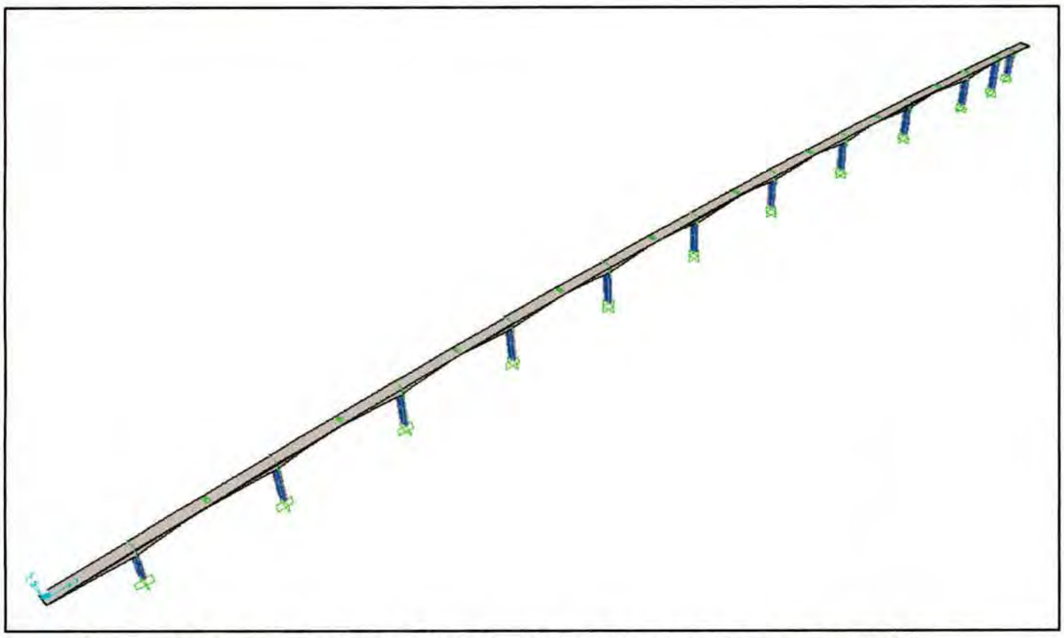


Figure 6.12 3D view of Mathematical Model of Meghna Bridge

6.6.2.1 Capacity Curve of Meghna Bridge

Meghna Bridge Bents consist of a single column whose strong axis is aligned in the transverse direction (Y-Direction).

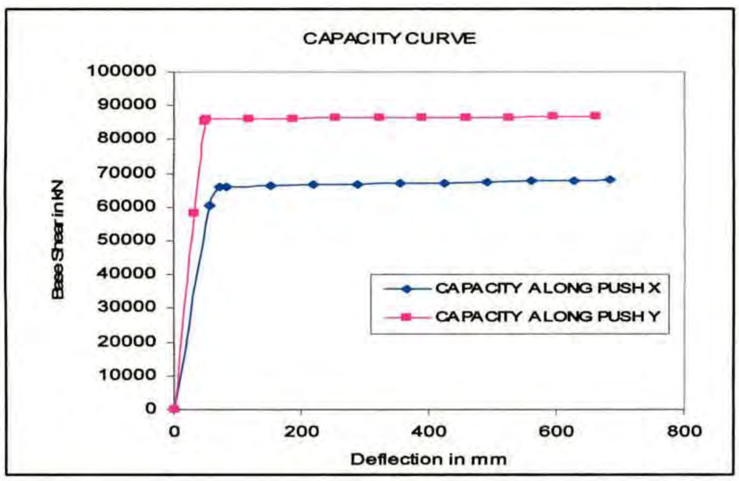


Figure 6.13 Capacity Curve for Meghna Bridge

The bent column is an unequal but symmetrical hexagon. Its bottom end is rigidly connected to the pile cap and top is monolithically cast with the deck. In both the direction the bent acts as a cantilever column carrying lumped mass of superstructure. Since the pier column's major axis is aligned in the transverse direction, so in this direction, load resisting capacity is more than that of longitudinal direction as evident from Capacity Curve (Figure 6-13). The Capacity Curve is a plot of deflection of deck (mid span) top vs. base shear or support reactions.

6.6.2.2 Hinge Status of Meghna Bridge near performance points when pushed in the longitudinal direction

Plastic hinges are formed in the piers when stress level exceed yield point. Hinges are

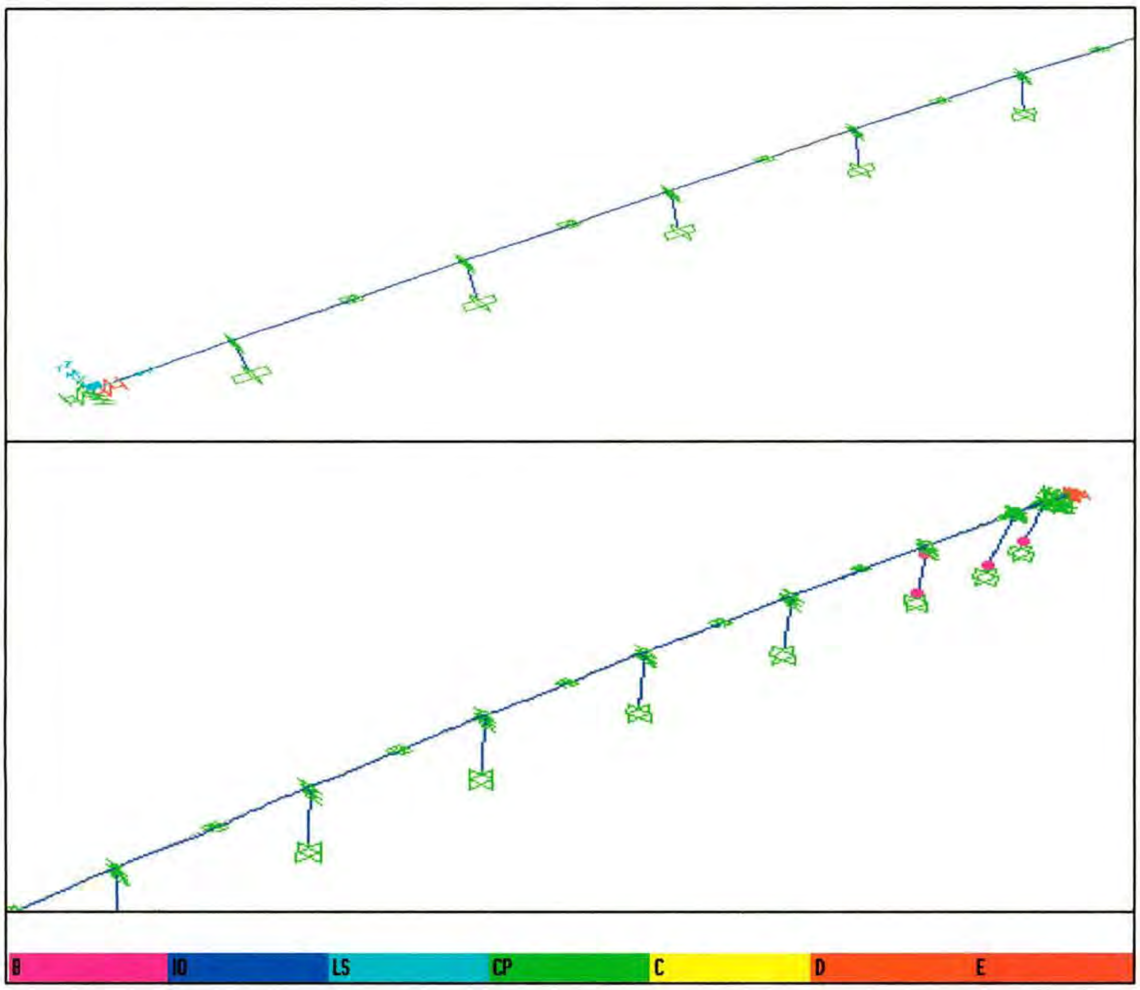


Figure 6.14 Hinges Status near at Performance Points of Meghna Bridge along the bridge length

developed in conformity of hinge properties assigned at the joint at the time of modeling. Figure 6.14 shows the hinge types formed at different location of pier. Yield stress exceeded at 6 nodes out of 136 nodes when pushed in the longitudinal direction. All these 6 nos. remain within Immediate Occupancy(IO) level and are formed at pile cap

6.6.2.3 Hinge Status near performance points when pushed in the transverse direction of Meghna Bridge

Plastic hinges are formed in the piers when stress level exceed yield point. Hinges are developed in conformity of hinge properties assigned at the joint at the time of modeling.

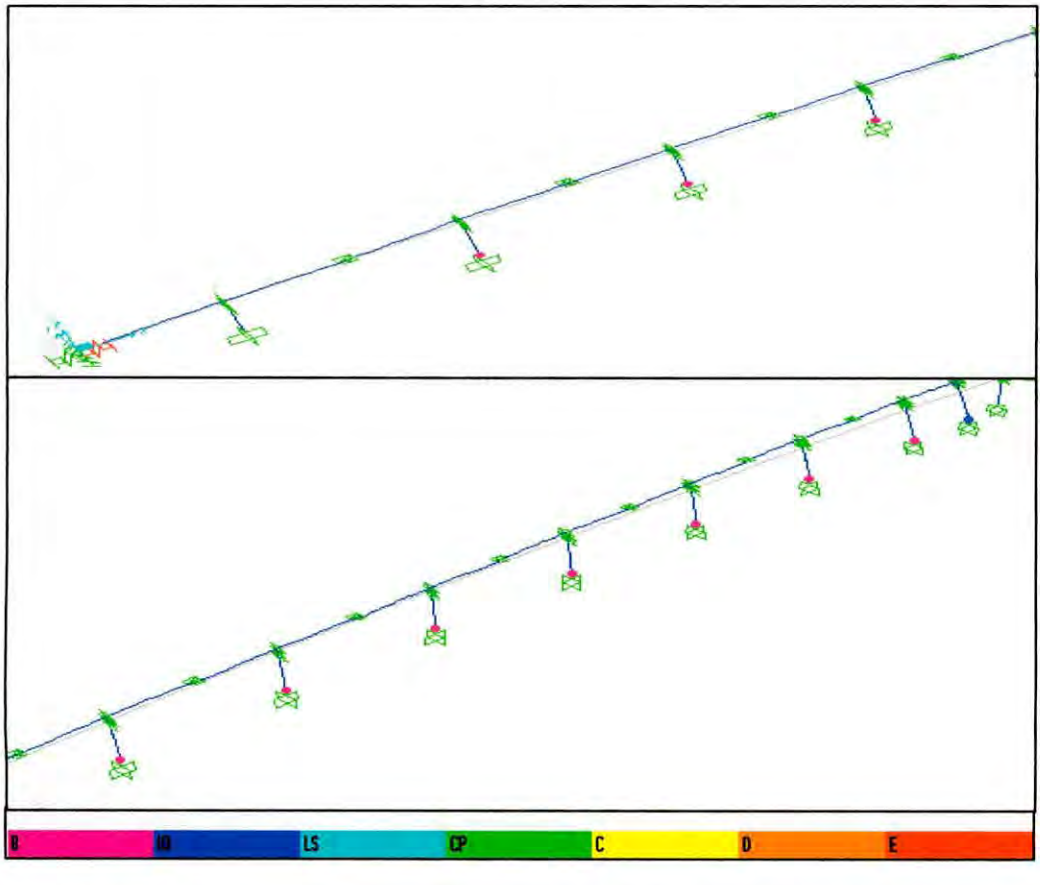


Figure 6.15 Hinges Status near Performance Points of Megna Bridge in the transverse direction

Figure 6-15 shows the hinge types formed at pile cap location of pier. Yield stress exceeded at 18 nodes out of 136 nodes when pushed in the transverse direction. Out of these hinges, 12 nos. remain within Immediate Occupancy(IO) level and 2 nos. lies within IO and LS level.

6.6.2.4 Graphical Representation of performance level along bridge length of Meghna Bridge

Figure 6-16 shows the result in graphical representation of performance point when the bridge is pushed in the longitudinal direction. At the performance point the spectral

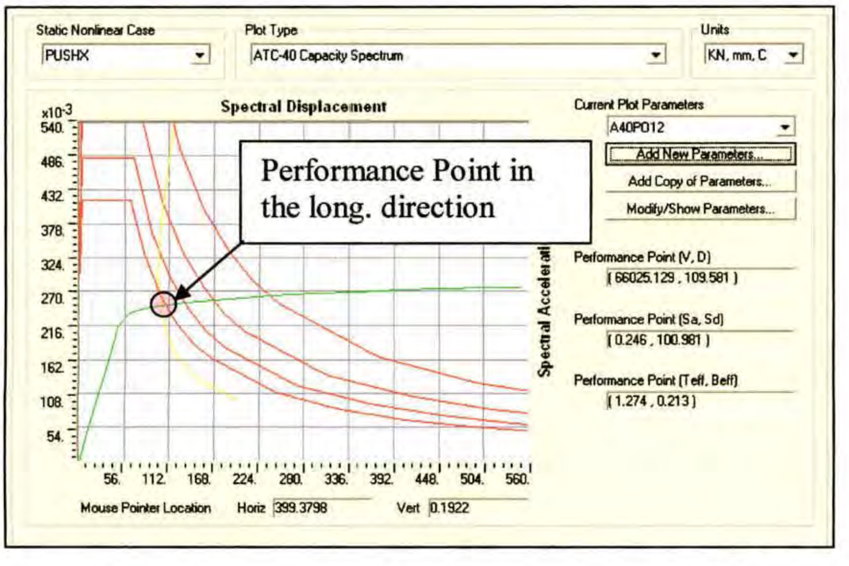


Figure 6.16 Capacity and Demand Spectrum of Meghna Bridge in the longitudinal direction of the bridge.

demand is 101mm. At this demand, the base shear value is about 66025 kN and the effective period is 1.274 sec and corresponding damping is 21.3%.

Figure 6-17 shows the graphical representation of performance point when pushed in transverse direction. At the performance Point the spectral demand is 97mm. At this

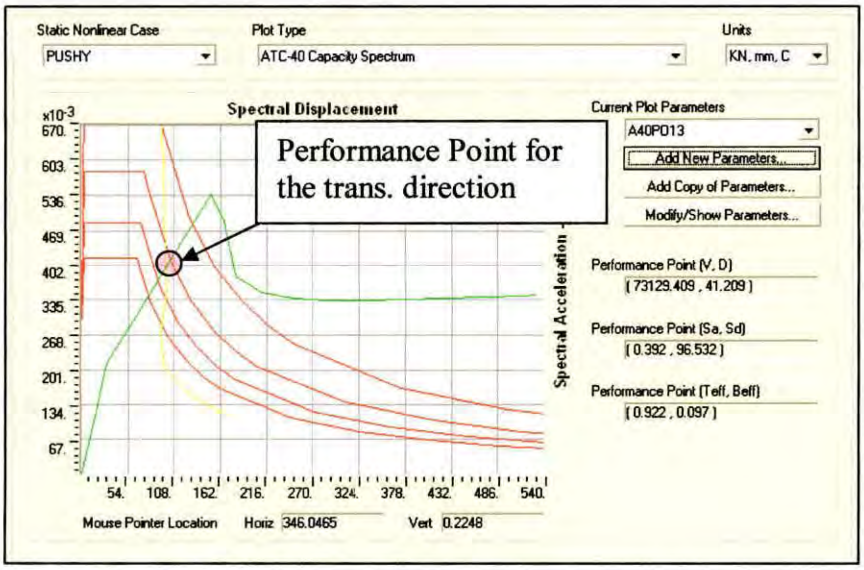


Figure 6.17 Capacity and Demand Spectrum of Meghna Bridge in the transverse direction of the bridge

demand, the base shear value is about 73129 kN. The effective damping is 9.7% and the effective period is 0.922 sec.

6.6.2.5 Graphical Representation of performance level along both axes of Meghna Bridge

Figure 6-18 shows performance points both in the longitudinal and transverse directions in the same plot. In the plot, X-Direction represent longitudinal direction and Y-Direction represent transverse direction.

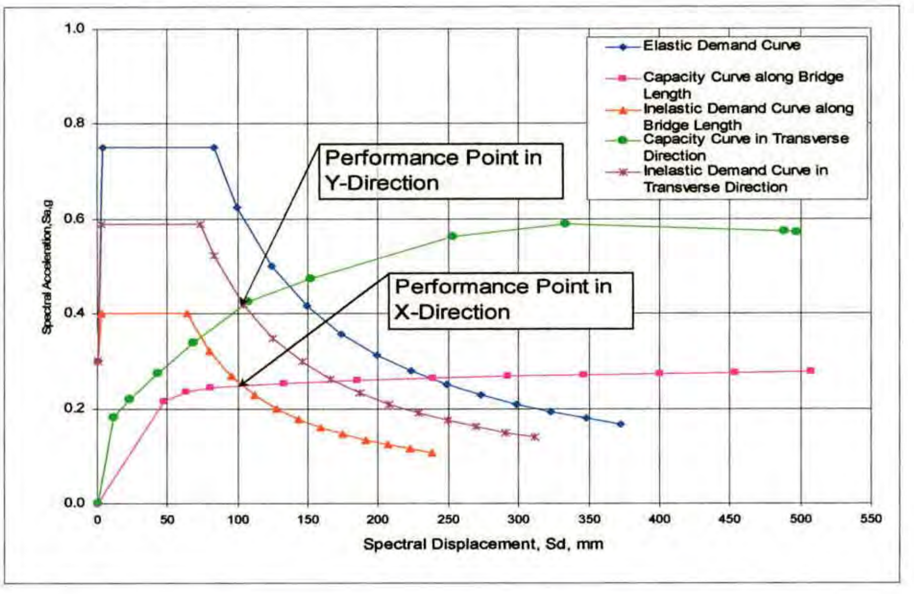


Figure 6.18 Performance Points of Meghna Bridge under expected Push Load

From the plot it is obvious that the demand for both the direction does not vary much due to mainly identical sectional properties of the pier shaft in both direction. Performance status is discussed in the next chapter.

6.6.3 Doarika Bridge

Doarika Bridge is a simply supported eight span bridge. The superstructure rest on pier cap through elastomeric bearing. Over each pier there is an expansion joint between the super structure which is free from restraint in all three axes. Deformation of deck top at highest altitude includes the deformation of bearing itself. But the base

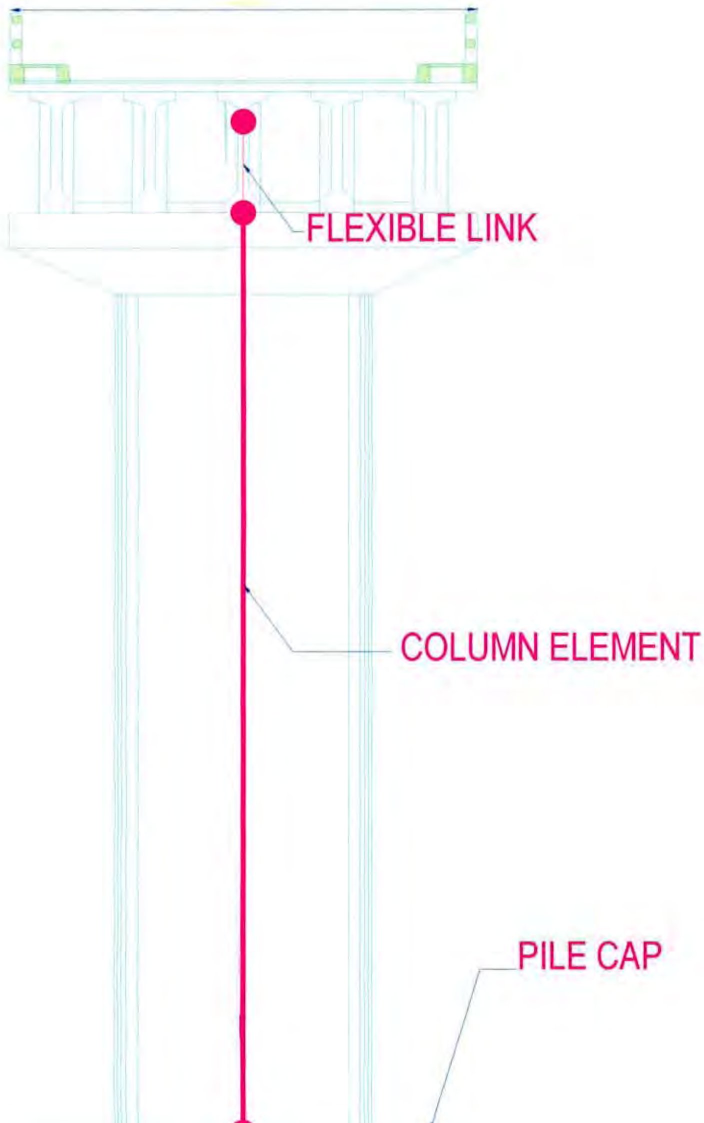


Figure 6.19 Bent Skeleton of Doarika Bridge

shear as found from analysis corresponds to the deformation of pier cap top which is the monitoring node of mathematical model of the bridge. Figure 6-19 represent a skeleton of a bent elements and Figure 6-20 shows 3D view of mathematical model of the full bridge.

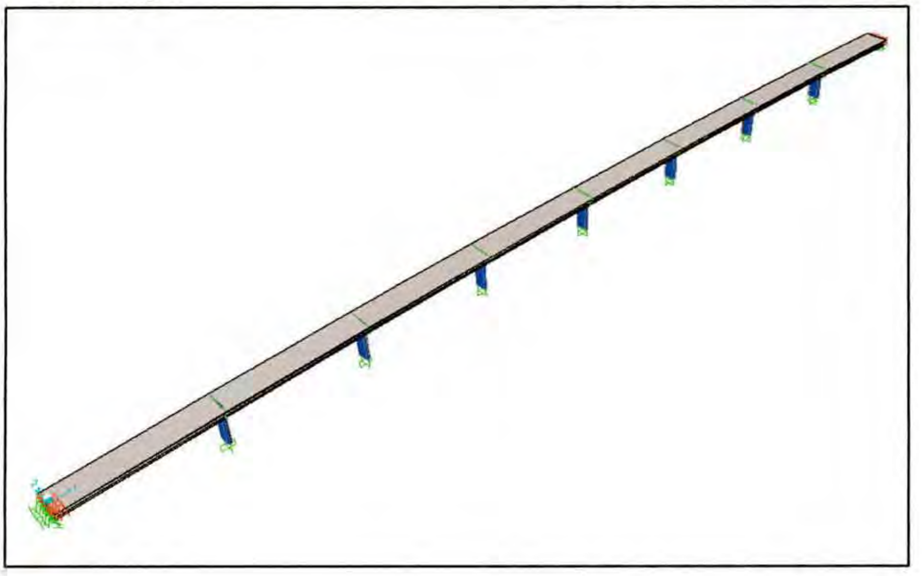


Figure 6.20 3D View of Mathematical Model for Doarika Bridge

6.6.3.1 Capacity Curve of Doarika Bridge

Doarika Bridge Bents consist of a column of oval shape whose strong axis is aligned

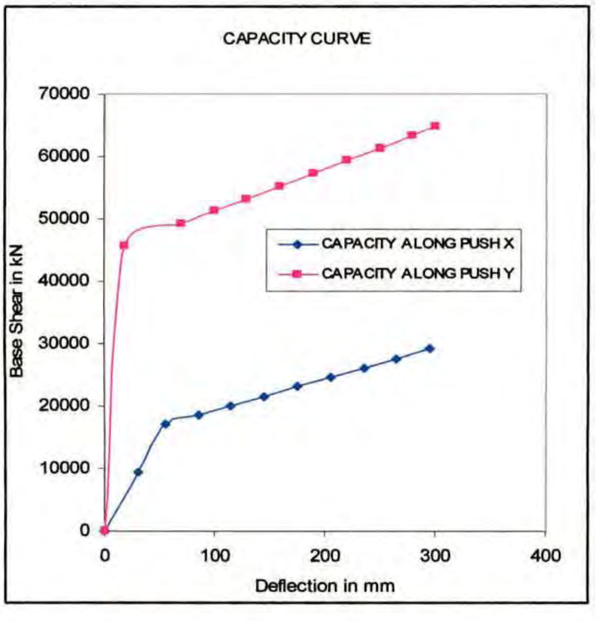


Figure 6.21 Capacity Curve for Doarika Bridge

in the transverse direction. Its bottom end is rigidly connected to the pile cap and top is connected with superstructure through elastomeric bearing at top of pier cap. In both directions, the bent acts as a cantilever column carrying a lumped mass of superstructure. Since the pier column's major axis is aligned in the transverse direction, so in this direction (Y-Direction), load resisting capacity is more than that of longitudinal direction as evident from Capacity Curve (Figure 6-21). The Capacity Curve is a plot of deflection of pier cap (mid span) top vs. base shear or support reactions.

6.6.3.2 Hinge Status of Doarika Bridge near performance points when pushed in the longitudinal direction

Plastic hinges are formed in the piers when stress level exceed yield point. Hinges develop in conformity of hinge properties assigned to the joint in the model.

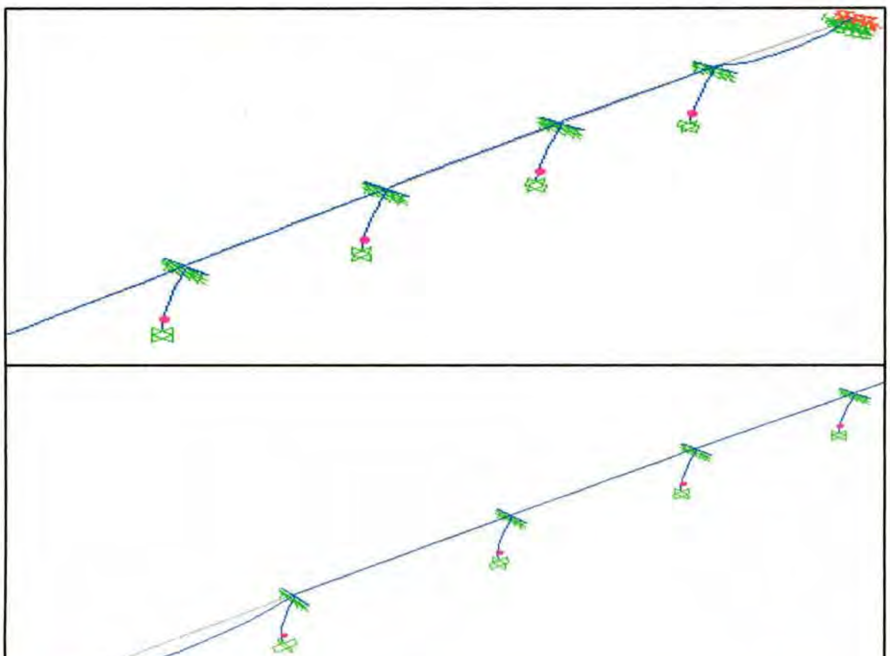


Figure 6.22 Hinges Status of Doarika Bridge near Performance Points along bridge length

Figure 6-22 shows the hinge types formed at pile cap location of pier. Yield stress exceeded at 14 nodes out of 28 nodes when pushed in the longitudinal direction. All these hinges are within Immediate Occupancy (IO) level.

6.6.3.3 Hinge Status of Doarika Bridge near performance points in when pushed in the transverse direction

Plastic hinges are formed in the piers when stress level exceed yield point. Hinges develop in conformity of hinge properties assigned to the joint in the model.

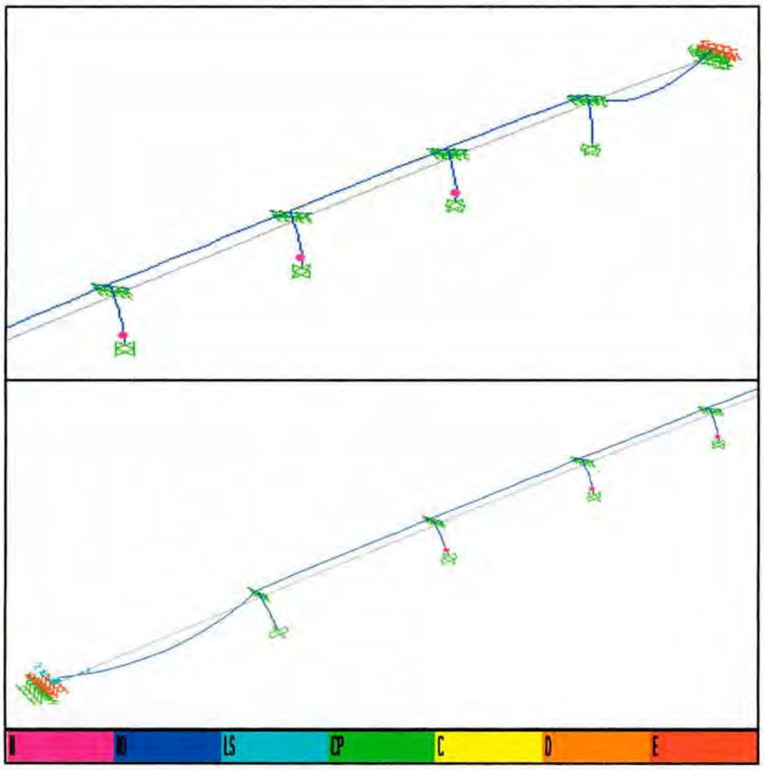


Figure 6.23 Hinges Status of Doarika Bridge near Performance Points in the transverse direction of the bridge

Figure 6-22 shows the hinge types formed at pile cap location of pier. Yield stress exceeded at 8 nodes out of 28 nodes when pushed in the transverse direction. All these hinges are within Immediate Occupancy(IO) level.

6.6.3.4 Graphical Representation of performance level of Doarika Bridge

Figure 6-24 shows the result graphical representation of performance point when the bridge is pushed in the longitudinal direction. At the performance point the spectral demand is 66mm. At this demand, the base shear value is about 16820 kN. The effective damping is 6.7% and effective period is 1.074 second.

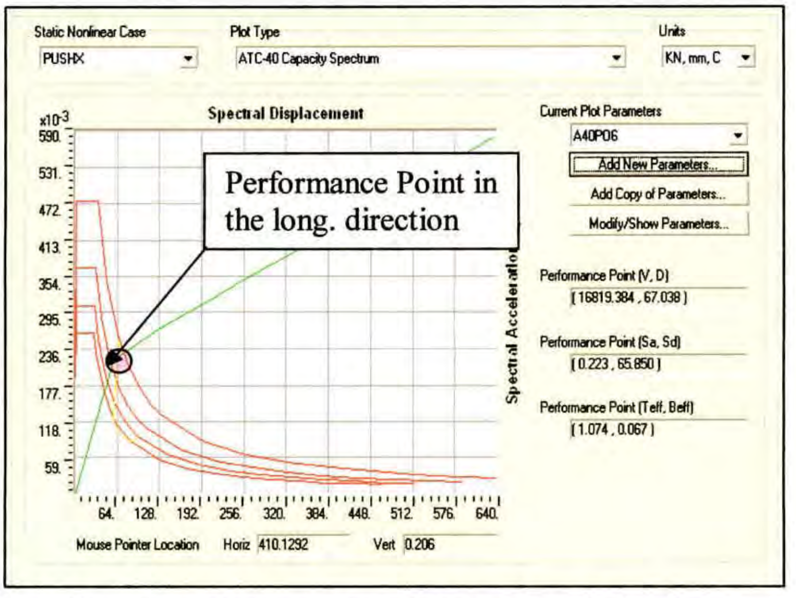


Figure 6.24 Capacity and Demand Spectrum of Doarika Bridge in the direction of the bridge span

At the performance point in the Y-direction (Figure 6-25), the spectral demand is 14mm. At this demand, the base shear value is about 37666 kN. The effective damping is 5.0% and effective is 0.342 second.

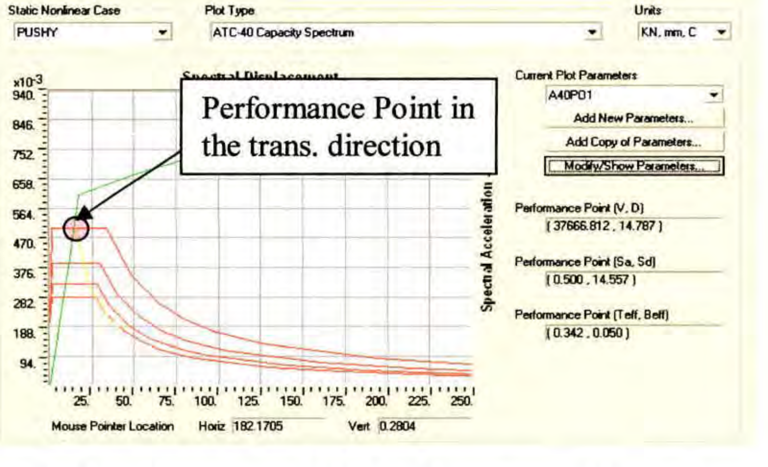


Figure 6.25 Capacity and Demand Spectrum of Doarika Bridge in the direction of the bridge span

6.6.3.5 Graphical Representation of performance level along both axes of Doarika Bridge

Figure 6-26 shows performance points in both the directions are simultaneously plotted and shown in Figure 6.26. .

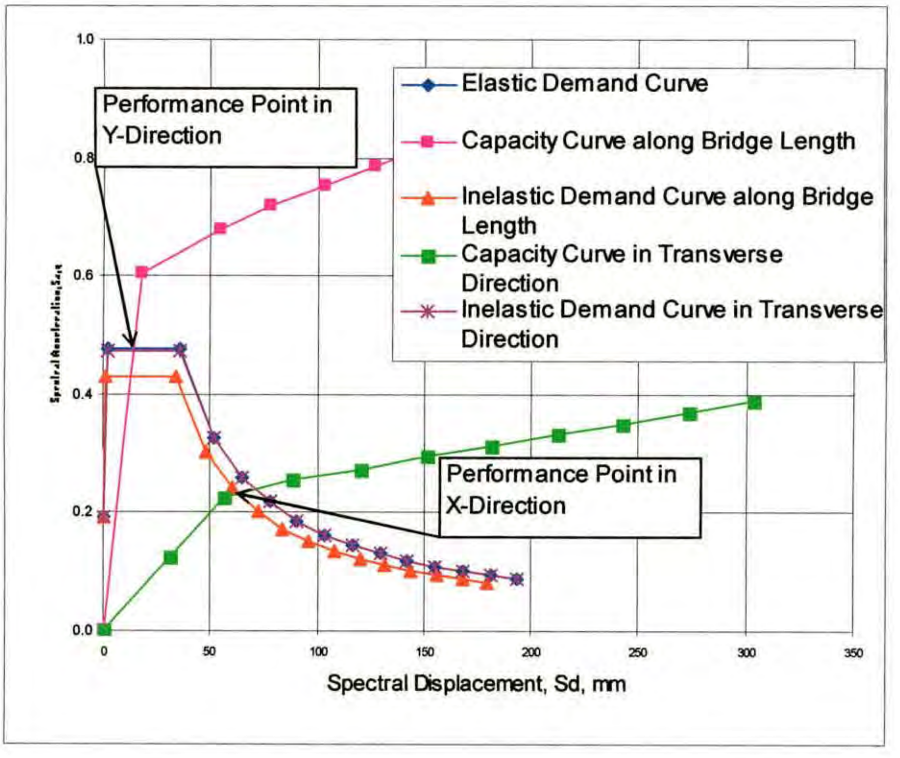


Figure 6.26 Performance Points under expected Push Load

Here X-Direction indicates longitudinal direction and Y-Direction represent transverse direction. It is evident from the plot that performance point in the transverse direction does not exceed elastic limit. Thus the bridge is expected to behave well during earthquake i.e. no damage is expected under a Design Earthquake.

Chapter 7

DISCUSSION ON ANALYSIS AND RESULT

7.1 Introduction

Three bridges are modeled and analysed to evaluate the performance level under seismic loads. Performance level indicates the damage strategy of the bridge or more clearly its serviceability after it has been shaken by an expected tremor. Analysis has been performed for the seismic load corresponding to Design Earthquake. Magnitude of this earthquake has been discussed in chapter 3.

7.2 Seismic Performance of Bridges

Seismic performance of the bridges under study varies because of mainly two reasons- i) the bridges are located in different seismic zones and ii) design geometries are different. The analysis results are discussed below.

7.2.1 Shah Paran Bridge

The seismic force for the type of this bridge is resisted by the supporting element of the deck system. It is expected that the abutment is much more stiff than a pier system so full scale model of abutment is not done. Only piers are modeled full scale. Since pier system consists of a single row of three RC columns along the transverse direction so it is more stiff in this direction compared to the longitudinal direction. Capacity curve indicates the same behavior. The performance quantities in the longitudinal direction 0.141g(S_a) and 227 mm(S_d) while these quantities are 0.574g and 78mm in the transverse direction. The pushover analysis reveals that the bridge has very high seismic demand in the long direction and hence significant damage is expected but it will not collapse. It meets Life Safety Performance Level in the longitudinal direction and Damage Control Performance Level in the transverse direction. It is expected that the elastomeric bearings as provided in the bridge will deform more in the longitudinal direction than its allowable limit (40mm) so probability of bearing damage is expected. The seismic performance quantities are also tabulated in Table 7-1 and 7-2.

Table 7-1 Performance Quantities along Bridge Length (Shah Paran Bridge)

At Performance Point								
Along Length of Bridge								
Teff	V, kN	V/W	$\Delta_L mm$	Maximum total drift ratio		Maximum inelastic drift ratio		Deformation of Bearing, mm
2.546	17957	0.21	227	0.021	Life Safety (LS)	0.017	Life Safety (LS)	45

Table 7-2 Performance Quantities along Transverse Direction (Shah Paran Bridge)

At Performance Point								
Along Transverse Direction of Bridge								
Teff	V, kN	V/W	$\Delta_T mm$	Maximum total drift ratio		Maximum inelastic drift ratio		Deformation of Bearing, mm
0.742	54227	0.65	78	0.007091	Damage Control(DC)	0.0056	Damage Control(DC)	40

7.2.2 Meghna Bridge

The spans of progressive cantilevers are separated by hinges, also same is the case in two simply supported spans. But in cantilever span pier is fixed with the cantilever box girder while piers are connected with the girder by elastomeric bearings in the simply supported spans. Hinges are free to move in three principal directions. As a whole the seismic force is primarily resisted by the supporting element of the deck system i.e. by the piers. Pier system consists of a single unequal hexagonal shape RC columns which is stiffer along the transverse direction than that of the longitudinal direction. Capacity curve does indicate consistent result. The performance quantities in the longitudinal direction 0.246g(S_a) and 100 mm(S_d) while these quantities are 0.392g and 96mm in the transverse direction. The pushover analysis reveals that the bridge has more seismic demand in the long direction. However, its performances in both the direction remain within Immediate Occupancy(IO) Level. This ensures that no significant damage will occur during tremor of intensity as stipulated in Design level earthquake. Push Load are applied simultaneously at all span, no phase lag is considered. The seismic performance quantities obtained are given in Table 7-3 and 7-4

Table 7-3 Performance Quantities along Bridge Length (Meghna Bridge)

At Performance Point								
Along Length of Bridge								
Teff	*V*, kN	*V/W*	$\Delta_L mm$	Maximum total drift ratio		Maximum inelastic drift ratio		Deformation of Bearing, mm
1.274	66025	0.20	109	0.0031	Immediate Occupancy (IO)	0.0028	Immediate Occupancy (IO)	25

Table 7-4 Performance Quantities along Transverse Direction (Meghna Bridge)

At Performance Point								
Along Transverse Direction of Bridge								
Teff	*V*, kN	*V/W*	$\Delta_T mm$	Maximum total drift ratio		Maximum inelastic drift ratio		Deformation of Bearing, mm
0.922	73129	0.22	41	0.0011	Immediate Occupancy (IO)	0.0009	Immediate Occupancy (IO)	12

7.2.3 Doarika Bridge

Piers are modeled full scale excluding foundation. Here the piers are oval shaped single column having stiffness in the transverse direction more compared to the longitudinal direction. Capacity curve indicates the same thing. The performance quantities in the longitudinal direction 0.223g(S_a) and 66 mm(S_d) while these quantities are 0.475g and 17mm in the transverse direction. The pushover analysis reveals that the bridge has more seismic demand in the long direction and its performance meet the Immediate Occupancy Performance Level. It is expected that the elastomeric bearings provided in the bridge will deform more than its allowable limit (40mm). It is indicated by differential movement of piers which extends upto 50mm in the longitudinal direction. The seismic performance quantities obtained are given in Table 7-5 and 7-6 in the next page.

Table 7-5 Performance Quantities along Bridge Length (Doarika Bridge)

At Performance Point								
Along Length of Bridge								
Teff	*V*, kN	*V/W*	$\Delta_L mm$	Maximum total drift ratio		Maximum inelastic drift ratio		Deformation of Bearing, mm
1.074	16819	0.23	65	0.0036	Immediate Occupancy (IO)	0.0026	Immediate Occupancy (IO)	15

Table 7-6 Performance Quantities along Transverse Direction (Doarika Bridge)

At Performance Point								
Along Transverse Direction of Bridge								
Teff	*V*, kN	*V/W*	$\Delta_T mm$	Maximum total drift ratio		Maximum inelastic drift ratio		Deformation of Bearing, mm
0.379	30517	0.41	17	0.0009	Immediate Occupancy (IO)	0.0	Response does not exceed elastic range	8

Chapter 8

CONCLUSION AND RECOMMENDATION FOR FUTURE RESEARCH

8.1 Conclusion

Loss of life or disaster management due to the damage of a bridge by earthquake is less prominent compared to those of buildings. This ideology make scope for the design offices to pay less attention to perform detail analysis for earthquake force even in the recent past. But situation has been changing now because of i) having more probability of occurrence of earthquake strong enough to cause significant damage of bridges and ii) great impact on national economy due to damage of bridges. Again the damages observed during several earthquakes in the past should be a lesson to be learnt for selecting a suitable design earthquake performance level and also in adopting a suitable structural system incorporating necessary detailing. Almost all the bridges in Bangladesh were designed following AASHTO code. AASHTO code developed a separate section of guidelines to design a bridge for earthquake force but does not include inelastic properties of a bridge components resisting earthquake force. Recently developed elasto-plastic design called performance based design solve this shortcomings of the code stipulated design method.

The aim of this study is to evaluate performance of a bridge under a given seismic force incorporating inelastic properties of the bridges. Three existing bridges from three seismic zones of Bangladesh are studied. No design report is found for any bridges, however as-built drawing were made available. Bridges are modeled using the design sections as found in the drawing. All the bridges are modeled for earthquake excitation in two major axis. Analysis for excitation in the vertical axis is not performed. With the limitation of three bridges as detailed in the preceding chapters, the following conclusions can be drawn:

i) Shah Paran Bridge is located in seismic zone-3. The analysis result shows different responses in two directions. In the longitudinal direction, its performance conform to Life Safety Performance level while it meets Damage Control Performance level in the transverse direction. Such performances indicate that the bridge is expected to be significantly damaged without collapse if it is shaken by a design earthquake as considered in the analysis. But the design objective should not allow the structure to go beyond the Immediate Occupancy (IO) Level so that the structure can restore its serviceability condition soon after earthquake. A retrofit scheme like column jacketing of pier will improve performance of the bridge. But retrofitting scheme is yet to be introduced in Bangldesh. On the other hand if design for seismic force could be done following performance based analysis with the performance objective of Immediate Occupancy (IO) Level, the construction cost would increase a bit but difficulties in post earthquake maintenance like retrofitting and/or upgradation could be avoided.

ii) The Meghna and Doarika Bridges meet Immediate Occupancy Performance Level. So structural damages for these bridges is expected to be minor if these are shaken by a Design Level Earthquake.

iii) In Shah Paran Bridge, the differential movement of piers in the longitudinal direction cause strain level of elastomeric bearings to exceed allowable limit indicating potentiality of bearing damage.

iv) Expansion gap between abutment and deck is not enough to accommodate large displacement of deck leading to probability of back wall damage for all the bridges.

v) Performance based design of the existing important bridges should be undertaken and appropriate retrofitting measures to be carried out in order to save the significant damage of these bridges under probable design earthquake forces.

vi) Performance based design for seismic load should be adopted in detail design of bridges in Bangladesh.

8.2 Recommendations for Future Study

The following recommendations are put forward for future study:

i) Research based software should be used to perform analysis instead of general purpose software for better result.

ii) Soil-structure interaction is not incorporated by full scale modeling of abutment and foundation system which is left for future study;

iii) Performance of elastomeric bearing as a link element may be examined at micro level to evaluate its actual behavior.

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