COMPARATIVE STUDY OF FORCE BASED AND DISPLACEMENT BASED DESIGN OF BUILDINGS

A DISSERTATION

Submitted in partial fulfilment of the requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

EARTHQUAKE ENGINEERING

(With Specialization in Structural Dynamics)

By

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CANDIDATE'S DECLARATION

I hereby declare that the work presented in this dissertation "**COMPARATIVE STUDY OF FORCE BASED AND DISPLACEMENT BASED DESIGN OF BUILDINGS**" in partial fulfillment of the requirements for the award of the degree of "**MASTER OF TECHNOLOGY**" in Earthquake Engineering with specialization in ***Structural Dynamics*** of the Indian Institute of Technology Roorkee, Roorkee, is an authentic record of my own work carried out during the period from July 2003 to June 2004 under the guidance of **Dr. Yogendra Singh,** ***Asst. Professor,*** and **Dr. D.K. Paul,** ***Professor,*** Department of Earthquake Engineering, Indian Institute of Technology Roorkee, Roorkee.

The matter embodied in this dissertation has not been submitted by me for the award of any other degree or diploma.

Dated: 28 June, 2004

Place: Roorkee

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CERTIFICATE

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

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ABSTRACT

Earthquakes, unlike other natural disasters, leave no time for mass evacuation of people thus result in a huge loss of lives and property. The developments that took place over the century in the fields of seismology and Earthquake resistant design envisage the understanding and effort put in by the yesteryear researchers. Development of Probabilistic methods to estimate the ground motions have been underway in many countries. In India too, much efforts are made to develop a probabilistic estimation of ground motions. However, a complete Probability based development of Seismic zoning map for Indian region is yet to become a reality.

Capacity Based Design, the first design philosophy to rationally consider the ductility of the structure, provides a detailed sequence of designing the members for effective energy dissipation and guides in assigning a predefined failure mechanism. However, the uncertainties in estimating the exact seismic capacity of the structure, particularly its ductility have forced the research community to develop a new design philosophy called Performance Based or Displacement Based design in which the displacement response of a structure is related with strain-based limit states. Since, the strain and deformation give a better indication of the level of damage in the structure, different levels of performance objectives are fixed based on the strain and displacement limits of the structure and its elements.

Four different models of a building have been used for the present study. The bare frame of the building has been analysed using linear analysis by STAAD Pro software for Importance Factors of one and 1.5, designed using force based methods and the same bare frame has been analyzed using nonlinear pushover curve by RAM Perform3D software to estimate its performance. Later the frame has been modeled with infills and the nonlinear analysis is performed to assess the behavior of building with infills. From the study it was found that the performance of the building designed for unit Importance Factor has matched with the performance targets assigned by Indian standards (IS). However for Importance Factor of 1.5, the structure performed for Life safety under Design Basis Earthquake which is contradictory to Immediate Occupancy level assigned by IS code. The performance of the frame modeled with infills has improved compared to bare frame.

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INTRODUCTION

1.1 GENERAL

An Earthquake, a natural disaster that unlike the other disasters like floods etc leaves no time for evacuation of people to safer places thus causing a huge loss of lives as well as property. Hence designing our buildings to resist these seismic loads is the only feasible alternative. Earthquake resistant design concepts have been under development over the years with the inputs taken from the damages of buildings in each earthquake. Each damage case has provided important information for improving the design and construction practices thus trying to protect the occupants of the buildings. The seismological and design based developments that took place over the last century explains a real perception of the subject by yester year researchers. This chapter gives an insight in to the systematic developments that took place in the field of earthquake engineering and the roles played by different people and organizations.

1.2 HISTORICAL DEVELOPMENT OF SEISMOLOGY

Robert Mallet (1862) is the person who is believed to have done some first scientific investigation about earthquake phenomena. He himself coined the terms such as "seismology", "hypocenter", "Isoseismal" and "wave path" etc. Later, attempts were made to develop seismographs to record the earthquake ground vibrations.

Milne, together with J.A. Ewing and T. Gray developed a modern three directional seismometer in 1881. In 1872, K. Gilbert, a U.S. geologist, tried to relate fault movements with earthquakes in his report. This was evident in 1906 San Francisco earthquake where a clear movement was observed along San Andreas Fault. Harry F. Reid in 1908 presented the "elastic rebound theory". In 1915, Alfred Wegener came out with the theory of continental drift in which he claimed that a single mass, called Pangaea, drifted and split to form the current continents [1].

Major earthquakes tend to occur along the moving tectonic plates when the strain energy, accumulated by the resistance against inter-plate movement is suddenly released. Though there is no consensus over the accurate prediction of an earthquake, the seismically calm regions along the tectonic plates over the years are highly potential sources of an earthquake. With in each tectonic plate there are again intra plates which may cause earthquakes with a shallow focus (30km below earth surface) but with a long return interval (1000-3000 years).

The structural engineering people, however, insisted upon knowing the ground accelerations to estimate the fictitious inertia forces acting on the structures during an earthquake. The seismograph was not capable of measuring these ground accelerations. Efforts were made by Milne et.al to estimate the maximum ground acceleration from the measured seismograph records. But this resulted in an underestimate of maximum ground acceleration as the dominant frequencies in displacement and acceleration signals were different. At the U.S. seismological field survey, established in 1932, F. Wenner et.al worked on the development of first strong motion Accelorograph. The well-known El

Centro records obtained during the 1940 Imperial Valley earthquake are considered as standard acceleration records for a long time.

Later a comprehensive development has taken place in bringing out the present day electronic Accelerographs to pick up the accelerations and record them in the event of an earthquake.

1.3 HISTORICAL DEVELOPMENT OF SEISMIC DESIGN

The first quantitative seismic design recommendations were made in Italy in 1908 after the Messina earthquake which killed more than 83000 people [1]. Professor M. Panetti recommended that the first story be designed for a horizontal force of 1/12 times the weight above and the second and third stories be designed for 1/8 of the building weight above. The height of the building was limited to three stories. Riki Sano in 1916 proposed the use of seismic coefficient that is equal to the maximum ground acceleration normalized to gravity acceleration, *G*. He, however, ignored the amplification of lateral acceleration response of the structure.

The first Japanese building code came into existence in 1919. However, seismic design concepts were included in the code (1924) only after 1923 Kanto earthquake (M 7.9) which killed more than 140,000 people. It considered a seismic design coefficient of 0.1 for the first time in the world. The first edition of Uniform Building code (UBC) in 1927, a model code in the United States, adopted the seismic coefficient method for structural design of buildings. The seismic zone criteria depending upon the seismic risk from one region to other, was accommodated in the future developments of the UBC code. The Building Standard Law was brought out in Japan in 1950 which gives the calculation of lateral force using all related factors such as seismic zone factor (Z), soil structure factor (G), seismic coefficient (K; depends on structure height) and weight of storey (W).

1.3.1 Response spectrum concept

M.A. Biot, in 1933, introduced the concept of response spectrum where he plotted the maximum response amplitude of simple systems with varying time periods. The first earthquake response spectra were developed from the records of 1935 Helena, Montana, earthquake and 1938 Ferndale, California earthquake. He found out that the response amplitude is decreasing with the increase in time period. This finding of Biot had been incorporated by Los Angeles Building code in 1943 and UBC in 1949. Surprisingly the time period effect was not considered in Japan until 1981.

N.M. Newmark made a significant contribution to the earthquake engineering and structural mechanics by developing a numerical procedure to solve the equation of motion on digital computers. Newmark et.al [2] reported the relation between maximum response of linearly elastic and elasto-plastic single degree of freedom systems under ground motions. For the linear and elasto-plastic systems having same initial period, the strain energy stored at the maximum response was comparable in short period range and maximum displacement response amplitudes were comparable in a long period range. Newmark proposed that the elasto-plastic SDOF system having a ductility μ (ultimate deformation divided by yield deformation) has to be provided with a minimum base shear coefficient C_y to resist ground motion that produced elastic response base shear coefficient C_e.

1.4 NEW SEISMIC DESIGN CONCEPTS

Consideration of ductility available in the structure led to the development of very idealized design methodology in 1970s called "Capacity Design Method". This method of design primarily concentrates on assigning a predefined failure mechanism to the structure by proper energy dissipation. The serious concern to fix a performance level to the structure depending upon its importance has given birth to the idea of "Performance Based" or "Displacement Based Design". These two important design methodologies have been discussed in detail in the subsequent Chapters.

Energy dissipation devices like dampers and base isolation devices used to reduce the demand on the structure are also becoming increasingly popular.

1.5 INDIAN PERSPECTIVE

Indian tectonic plate being one of the most active tectonic plates, India has faced a number of deadly earthquakes that left thousands of people dying each time. The Bureau of Indian standards (BIS) has been doing a considerable effort to mitigate the hazards due to these earthquakes. Scientists in India have concentrated on bringing up a code of practice for seismic resistant design (IS 1893), which gives guidelines to Engineers on the amount of forces to be accounted in the seismic regions. Development of Seismic Zoning map has been a subject of research in India for the past 40 years. Seismic zoning map is a map that divides entire country into different regions according to the earthquake potential in those regions.

1.5.1 Development of Seismic Zoning map

BIS constituted a multi-disciplinary committee in 1960 to bring out a code for earthquake resistant design. The first seismic zoning map was developed by this committee using a statistical approach. The isoseismals of 23 major earthquakes, the trend of principal tectonic features are used to develop a seven zone seismic zoning map varying from Zone '0' to Zone 'VI'. This code was later found deficient as the boundaries between seismic zones I and II were not clearly visible in some regions. Also, the Indian Meteorological Department (IMD) has assigned magnitudes to many historical earthquakes using correlation relations. Therefore, the BIS committee revised the seismic zoning map in 1966 to account this available information and to provide additional emphasis on geology and tectonics. The number of zones remained unchanged [3].

The 1967 Koyna earthquake (M 6.5) that occurred in peninsular shield of India has forced the second revision of the code in 1970 to review the given low seismic status to peninsular region. It was also decided to reduce the number of zones to five instead of seven. In the latest revision of seismic zoning map that has been adopted in IS 1893 – 2002, the zone I is enhanced to zone II to make the total number of zones to four (Fig. *1.1*). It was also decided to have an interim revision to review the seismic status of peninsular India based on a probabilistic hazard analysis. IS 1893: 2002 recommended various zone factors for Maximum Considered Earthquake (MCE) for the service life of 100 years. For Design Basis Earthquake (DBE), which is expected once during the lifetime of the structure, half of the MCE zone factor is to be considered.

1.5.2 Design Methodology

IS 1893 has adopted a design philosophy to ensure that structures possess minimum strength to

1) Resist minor earthquakes (< DBE) without damage,
2) Resist moderate earthquakes (DBE) without significant structural damage, and
3) Resist major earthquakes (MCE) without collapse.

The revised code in 2002, considers the ductility in the form of a Response reduction factor (R). It recommends different Importance factors (I) to consider the usage of the building. The code recommends two methods of analysis namely Equivalent static load Method and Dynamic Analysis. For calculating the Design Base Shear of the building using Equivalent static load method, design horizontal coefficient (A_h) has to be found out using the seismic zone factor (Z), Importance factor (I), Response reduction factor (R) and spectral acceleration coefficient (S_a/g) obtained from the Response spectrum curve for the specified soil type and the structure's fundamental time period.

The dynamic analysis is recommended for buildings of 40m in height situated in zones IV and V, and for irregular buildings of 12m or more in height situated in zones IV and V. Code recommends response spectrum method of dynamic analysis with Complete Quadratic Combination (CQC) method used for modal combination [4].

FORCE BASED DEISGN METHODOLOGY

2.1 INTRODUCTION

The development of various design philosophies in the past was always based on the elastic design of members only. These philosophies include Working Stress design followed by Ultimate Strength design and Limit State design. In all these designs, the forces developed in the members are calculated for worst combination of loads using linear analysis methods and the members are designed. Many countries adopted a conservative design philosophy which results in a over stiff structure. In reinforced concrete situations that could actually lead to a reduction in ductility and safety [5]. Inelastic response of members was considered in the design only in the mid-1970s. An integrated design procedure called Capacity Design was developed in New Zealand under the leadership of T. Paulay.

2.2 CAPACITY DESIGN

2.2.1 Background

The experiences from the past Earthquakes have shown that the structural members inevitably undergo nonlinear deformation under the seismic loading. As these nonlinear deformations incorporate the energy dissipation within the structure, these enable us to design the structure for only 15 to 25% of design forces corresponding to elastic response of the structure and expect the structure to undergo large inelastic deformations without collapsing. However, not all inelastic modes of deformation are equally acceptable in the design. An inelastic mode like shear deformation reduces the member strength drastically and leads to severe damage.

Only those inelastic modes, which induce ductility in the structure, are encouraged as the ductility is the main source of energy dissipation. At the same time, it is not advisable to allow all the members of the structure to undergo inelastic deformation as the strength degradation in many members will make the structure unsafe for gravity loads. Keeping in view all these aspects, the Capacity Design method has evolved which is rational, deterministic and simpler in its approach.

Paulay stated that, "In the Capacity Design of structures for earthquake resistance, distinct elements of the primary lateral force resisting system are chosen and suitably designed and detailed for energy dissipation under severe imposed deformations [6]". Three strengths are commonly used in the Capacity Design method, namely Required Strength, Ideal Strength and Over Strength.

(a) Required Strength (S_u)

The strength demand arising from the application of loads & forces is the Required Strength. It is also called the Design or Dependable Strength.

(b) Ideal Strength (S_i)

The Ideal or nominal strength of a section of a member, S_i, is based on established theory predicting a prescribed limit state with respect to failure of that section. It is derived from the dimensions, reinforcing content, details of the section designed and code specified nominal strengths. In India, the Characteristic Strength corresponding to lower 5percentile limit of measured strength is adopted as Ideal Strength.

(c) Over strength (S_o)

The Over strength of a section, S_o, takes into account all possible factors that may contribute to strength exceeding the nominal or ideal value. These include steel strength greater than the specified yield strength, strength enhancement of steel due to strain hardening at large deformations, concrete strength at a given age of the structure being higher than specified, unaccounted-for compression strength enhancement of the concrete due to its confinement, and strain rate effects.

2.2.2 Salient features

Salient features [6] in the capacity design are as followed

1. The regions where Potential plastic hinges can be accommodated are identified and designed for a flexural strength closer to required strength S_u. These regions are carefully detailed by closely spacing the transverse reinforcement in order to accommodate the expected ductility demands.
2. Undesirable modes of inelastic deformation like shear or anchorage failures are avoided by ensuring the strengths of these modes exceed the capacity of plastic hinges at over strength S_o.
3. Components which are not part of the seismic resisting system are designed to remain elastic by ensuring that their strength exceeds the demands originating from the over strength of plastic hinges.

2.2.3 Weak beam - strong column strategy

A well-defined plastic mechanism is required for the effective implementation of capacity-based design. Many structural engineers prefer to adopt a weak beam–strong column strategy in which the moment resisting frames develop hinges, first at the end of girders and finally at the base of the first story columns. As there is no axial force acting on the girders, the deformation capacity of the girders will be high and they develop stable hysteresis loops.

For a given displacement of the structure, the ductility demand of the hinges in beams is less since the plastic deformations are uniformly distributed through out the structure (Fig. *2.1*). Where as for a simple column mechanism, the hinges at base of the column have to accommodate large rotations for the same displacement of the structure, which is very difficult. In addition, the existence of high axial load limits the rotational capacity of columns.

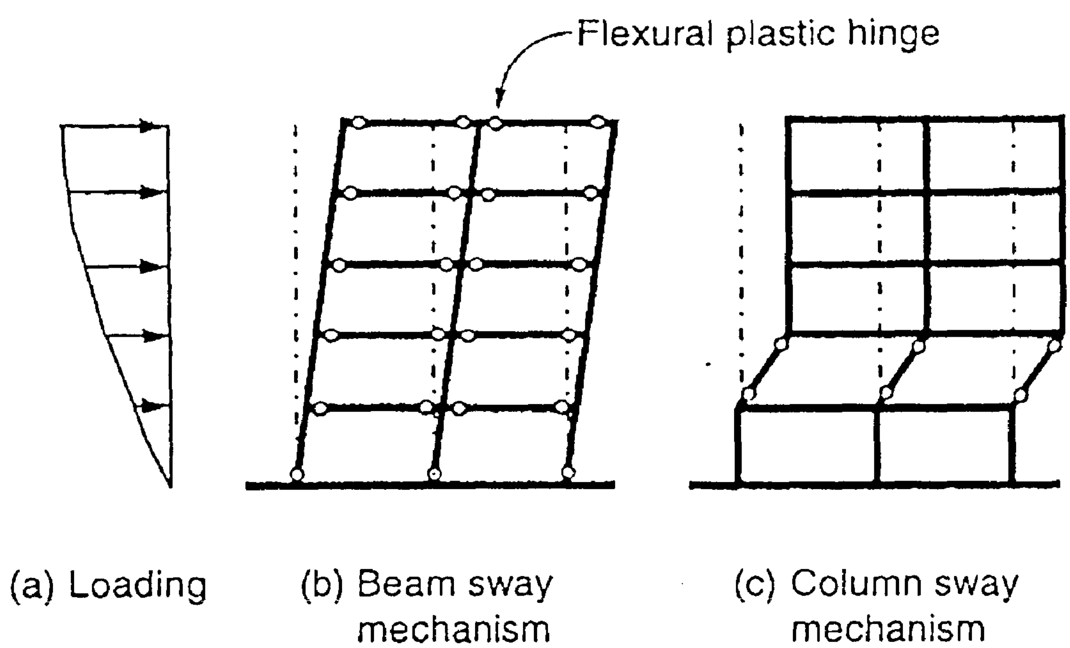


Fig. 2.1 Flexural mechanism of multistoried frames [13]

2.2.4 Uncertainties

The area of greatest uncertainty in Capacity Design of structures is the level of inelastic deformations that might occur under strong ground motions. However, by careful design of plastic hinges using quality detailing, the variation in the ductility demands from the expected value can be accommodated without loss of resistance to lateral forces.

2.2.5 Capacity Design procedure

A well-defined sequence of member design is required in order to achieve the prescribed weak beam - strong column mechanism. The steps given below explain the general procedure followed in the capacity design.

The India standard code IS 13920:1993 follows almost the same lines of Capacity Design methodology [7] except some aspects. The specifications used are also elaborated below.

2.2.5.1 Beam flexural design

Beams are designed in such a way that their dependable strength at selected plastic hinge locations is as close as possible to the moment requirements at those locations. Plastic hinges, in general, are located to be at column faces.

As per the IS code, the Beams are to be designed to resist the Earthquake load in flexure. This is made sure by providing the following specifications.

(a) The top and bottom consists of at least two bars continued through out the member length.
(b) The maximum steel ratio on any face at any section is limited to 0.025.
(c) The positive steel at any joint face has to be at least half of the negative steel at that face.

(d) Not more than 50% of the longitudinal bars can be spliced at one section. In addition, lap splices are not allowed within a joint or within the quarter length of the section where flexural yielding generally occurs. Lap length has to be equal to development length of tensile bar and hoops are provided over the entire lap length at spacing of 150mm.

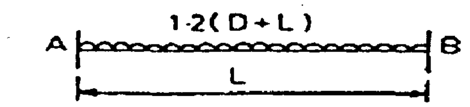
2.2.5.2 Beam Shear design

As the shear modes of inelastic deformations are ineffective in energy dissipation, shear strength at all sections in the beam is designed to be higher than the shear related to maximum flexural strength at plastic hinges. At the plastic hinge regions, special transverse reinforcement designed for conservative estimates of shear strength is adopted. IS 13920 gives the shear force to be resisted by vertical hoops as the shear force due to formation of plastic hinges at both the ends of the beam plus the factored gravity load on the span (Fig. *2.2*).

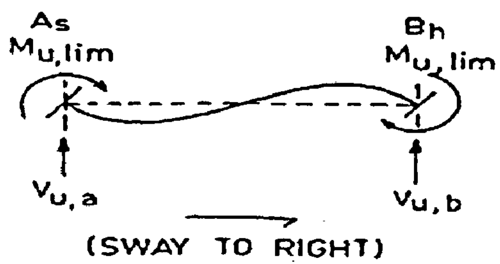
Hoops are arranged at a minimum spacing of 100mm over a length equal to 2d on either side of the flexural yielding section. In the other parts of the beam, a spacing of d/2 is provided.

2.2.5.3 Column flexural strength

Columns have to be designed for a moment capacity greater than the beam flexural over strength in order to ensure a weak beam-strong column hierarchy. While there is no mention in the IS code regarding the column flexural strength, a draft code proposed by S.K. Jain and C.V.R. Murthy [8] recommended the moment of resistance of columns to be at least 1.1 times to that of beams at a joint.

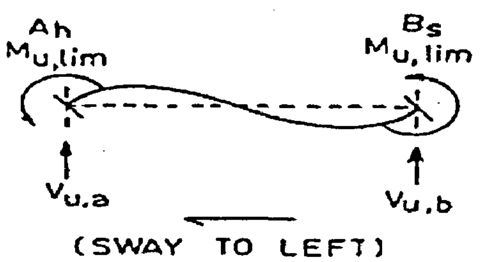


$$V_a^{D+L} = V_b^{D+L} = \frac{1{\cdot}2(D+L)}{2}$$



$$V_{u,a} = V_a^{D+L} - 1{\cdot}4\left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}}\right]$$

$$V_{u,b} = V_b^{D+L} + 1{\cdot}4\left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}}\right]$$



$$V_{u,a} = V_a^{D+L} + 1{\cdot}4\left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}}\right]$$

$$V_{u,b} = V_b^{D+L} - 1{\cdot}4\left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}}\right]$$

Fig 2.2 Calculation of Design shear force for beam [7]

2.2.5.4 Transverse reinforcement for columns

Depending upon the flexural over strengths of adjacent beams, an estimate of shear force is made from which the transverse reinforcement details are obtained.

IS 13920 takes the design shear force for columns as a factored shear force given by

$$V_u = 1.4\left[\frac{M_{u,lim}^{bL} + M_{u,lim}^{bR}}{h_{st}}\right] \tag{2.1}$$

Where $M_{u,lim}{}^{bL}$ and $M_{u,lim}{}^{bR}$ are moment of resistance, of opposite sign, of beams framing into the column from opposite faces and h_{st} is the storey height.

2.2.5.5 Beam-column joint design

As the beam-column joints are poor sources of energy dissipation, the inelastic deformations of these components have to be minimized by taking the ideal strengths of these joints equal to over strengths of plastic hinge regions in the adjacent beams. IS 13920 recommends the special confinement steel provided in column to be continued through the joint as well with a minimum spacing of 75mm.

DISPLACEMENT BASED DESIGN METHODOLOGY

3.1 INTRODUCTION

Conventional methods of seismic design including Capacity Design followed in several countries have the objectives of providing Life Safety and Damage Control depending upon the importance of the building. The design criteria are defined by limits on stresses and member forces calculated from prescribed levels of lateral shear force. Although the buildings designed by using these current codes performed well in the recent earthquakes for life safety point of view, the damages incurred were so high that either the building has lost its usage or the repair costs were very high [9]. This is mainly because of the uncertainties in estimating the exact seismic capacity of the structure particularly its ductility.

There is strong concern in research community that a design procedure is needed in which the displacement response of a structure is related with strain-based limit state. The strain and deformation give a better indication of the level of damage in the structure. So, by defining different levels of performance objectives based on the strain and displacement limits of the structure and its elements, the damage in any structure can be monitored. Displacement based or Performance based design has emerged as a powerful approach to cater the above needs.

3.2 METHODOLOGY

Displacement based design is defined as "a methodology in which structural design criteria are expressed in terms of achieving a set of performance objectives". A general methodology of applying Displacement based design to structures is, a traditional force based design is conducted and its design results are obtained. These results are used for nonlinear modeling of the same structure with all the displacements and strain limits defined and analyzing it using pushover analysis.

3.3 STAGES OF DEVELOPMENT

Many researchers are trying innovative ways of employing this method by inducing different aspects of design [10]. Also, the researches tried to design structures using force- based methods and compared its performance by displacement based approach [11]. Three organizations are mainly involved in the development of concepts and procedures for the Displacement based design namely: Structural Engineering Association of California (SEAOC vision 2000), Federal Emergency Management Agency (FEMA 273) and Applied Technology Council (ATC 40).

3.3.1 SEAOC vision 2000

This document has developed the framework for procedures that lead to design of structures of predictable seismic performance and is able to accommodate multiple performance objectives. The document presents the concepts and addresses the performance levels for structural and non-structural systems. Five performance levels are defined with specified limits of transient and permanent drift. It is suggested that capacity design principles should be applied to guide the inelastic response of the analysis of the structure and to designate the ductile links or forces in the lateral-force-resisting system. Possible design approaches suggested are (1) conventional force and strength methods; (2) Displacement based design; (3) energy approaches; and (4) prescriptive design approaches [9].

3.3.2 FEMA 273 document

This document [12] presents a variety of performance objectives with associated probabilistic ground motions. It discusses a number of analysis and design methods ranging from linear static to non-linear time history analysis. The performance levels defined (Table3.1) are discrete points on a continuous scale describing the building's expected performance, or alternatively, how much damage, economic loss, and disruption may occur. Each Building Performance Level is made up of a Structural Performance Level that describes the limiting damage state of the structural systems and a Nonstructural Performance Level that describes the limiting damage state of the nonstructural systems. The entire concept is based on the assumption that performance can be measured using analytical results such as story drift ratios or strength and ductility demands on individual components or elements. To enable structural verification at the selected Performance Level, stiffness, strength, and ductility characteristics of many common elements and components have been derived from laboratory tests and analytical studies and put in this document.

Table 3.1 Performance levels

Structural performance Levels	**Sign**	**Description (Post Earthquake damage state)**
Immediate occupancy	S-1	Limited structural damage, risk of Life-threatening is negligible and the building should be safe for occupancy
Damage control	S-2	An intermediate damage state that ranges from SP-1 to SP-3. This is required when damage state has to be limited beyond SP-2 but no need to achieve SP-1
Life Safety	S-3	Significant damage to structure, but risk of life threatening is very low. Extensive structural repairs are required.
Limited Safety	S-4	An intermediate damage state that ranges from SP-3 to SP-5. This is required when damage state has to be limited beyond SP-5 but no need to achieve SP-3
Structural stability	S-5	Structural system on the verge of experiencing total or partial collapse, gravity load resisting system is intact but significant risk of injuries. Significant aftershocks may lead to collapse
Not considered	S-6	It is useful for situations where non structural evaluation or retrofit is performed.
Nonstructural Performance Levels		
Operational	N-A	Non structural elements and systems are generally in place and functional. All machinery and equipment should be working.
Immediate occupancy	N-B	Minor disruption and cleanup should be expected, but functionality may exist.
Life Safety	N-C	Damage expected to non structural elements, but should not collapse or fall to cause any life injury.
Reduced Hazard	N-D	Extensive damage to non structural components, but should not cause significant injuries to groups of people.
Not considered	N-E	Non structural; elements, other than those that have an effect of structural response, are no evaluated.
Building Performance Levels		
Operational	1-A	Limited structural damage, minor non structural damage, and safe occupancy.
Immediate occupancy	1-B	Most widely used criteria for essential systems, all systems are reasonably usable. Contents may be damaged.
Life Safety	3-C	Low probability of threats to life safety either from structural or from non-structural elements.
Structural stability	5-E	Stability of structure only under vertical loads, falling of non structural elements.

3.3.3 ATC40 document

This document mainly deals with concrete buildings and it uses the Capacity spectrum method (CSM) of analysis extensively in the Evaluation procedures [13]. This nonlinear Static 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 representations of the earthquake demands. Two key stages of this method that outlines the entire procedure are developing a Capacity curve and Demand curve in order to obtain a Performance point, which is essentially useful in knowing the performance level of the structure.

(a) Development of capacity curve

Development of capacity curve gives a very good insight into the building's performance and the failure mechanism under yielding conditions. The capacity curve is generally constructed to represent first mode of response of the structure since the fundamental mode is expected to contain the predominant response. The structure's displacement response beyond its elastic limit is tracked and plotted here. This procedure uses a series of sequential elastic analyses, super imposed to approximate the force-displacement capacity diagram of the overall structure. Once after a set of elements yielded, the base shear and Roof displacements are to be noted and the model has to be modified by keeping zero or negligible stiffness to the yielded elements. A modified lateral force distribution is again applied until additional components yield. This process is continued until the structure becomes unstable or until a predetermined limit is reached. A typical capacity curve is shown in Fig. *3.1*.

This capacity curve has to be transformed into Acceleration-Displacement Response Spectra (ADRS) plot in order to be used in capacity spectrum method. The Transformation has to be done using the following equations.

$$S_{ai} = (V_i/W)/\alpha_I \tag{3.1}$$

$$S_{di} = \Delta_{roof}/(PF_I * \varphi_{I,roof}) \tag{3.2}$$

Where

S_{ai} = Spectral acceleration,

S_{di} = Spectral displacement

α_I = modal mass coefficient for first mode

PF_I = Mode participation factors for first mode

$\varphi_{I,roof}$ = roof level amplitude of first mode

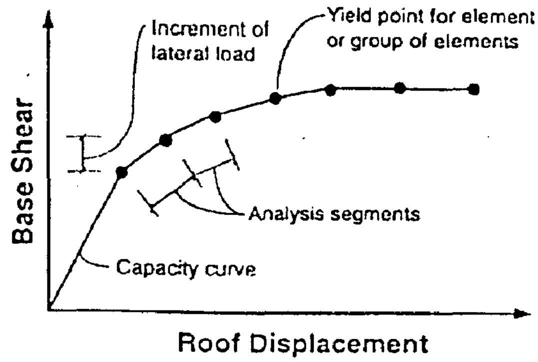


Fig 3.1 capacity curve [13]

(b) Development of Demand curve

A plot to represent the given ground acceleration has to be plotted against the Time period with 5% damping. In addition to this viscous damping, the structure pocess hysteretic damping also since the ground motion is of cyclic nature (Fig. *3.2*). Hence the given response spectrum has to be modified

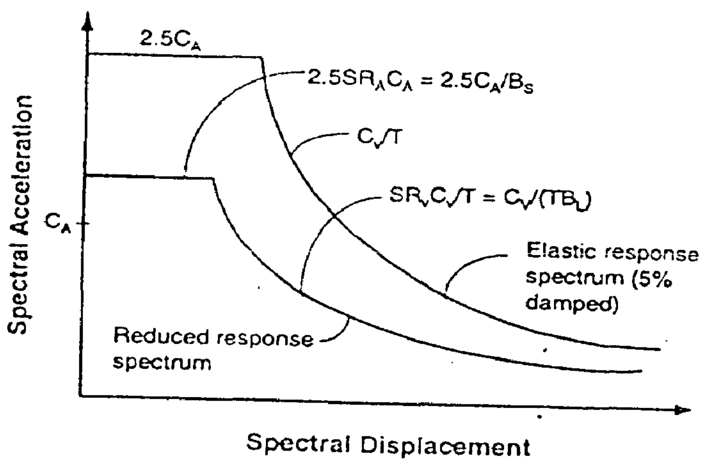


Fig 3.2 Reduced Response spectrum [13]

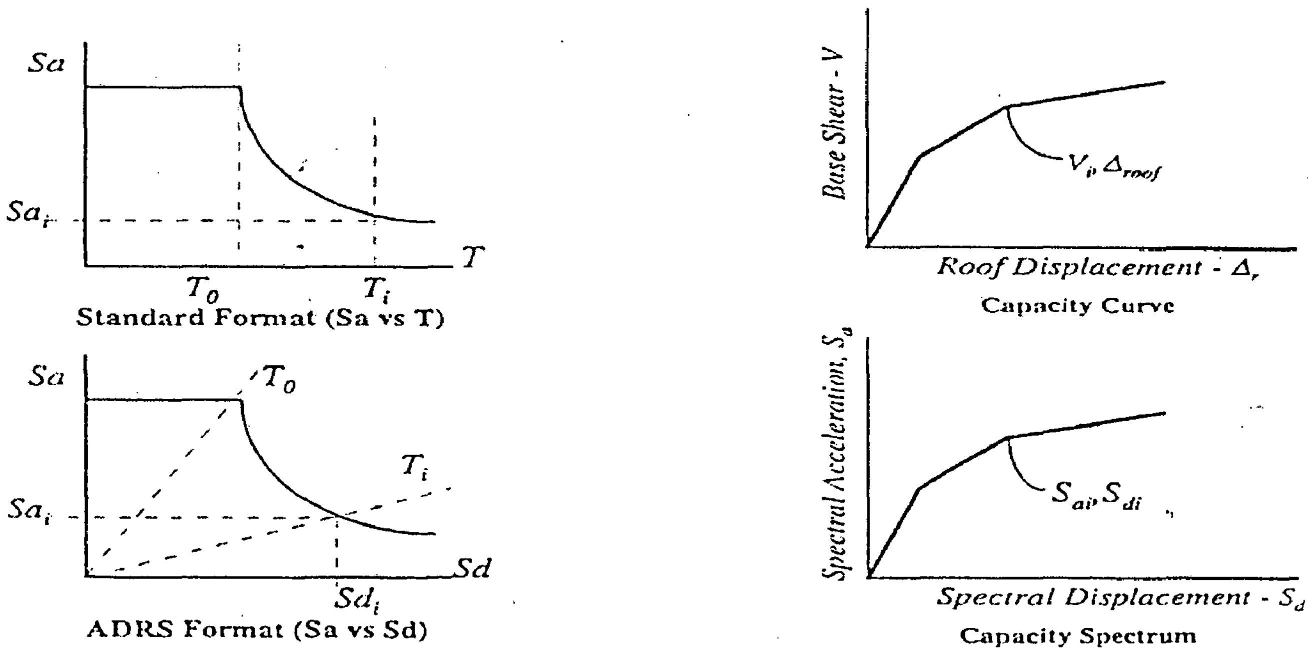
considering this hysteretic damping to get a reduced response spectrum. Later, this reduced response spectrum is converted into ADRS format (Fig. *3.3*) using the following equations.

$$S_{di} = \left(T_i^2 / 4\pi^2\right) * S_{ai} g \tag{3.3}$$

$$S_{ai} = \left(2\pi / T_i\right) * S_v \tag{3.4}$$

$$S_{di} = \left(T_i / 2\pi\right) * S_v \tag{3.5}$$

Where S_V = spectral velocity



(a) Response spectrum conversion *(b) Capacity curve conversion*

Fig 3.3 Conversion to ADRS format [13]

(c) Identifying the performance point

A performance point is the intersection of the capacity spectrum with the appropriate demand spectrum (Fig. *3.4*). This performance point represents the condition for which the seismic capacity of the structure is equal to the seismic demand imposed on the structure by the specified ground motion.

Determination of this performance point requires some trial and error procedures. ATC40 explains three such iteration procedures (analytical as well as graphical) using which the performance point can be identified. After identifying the performance point, the critical components are identified and their actions are checked for the specified performance level.

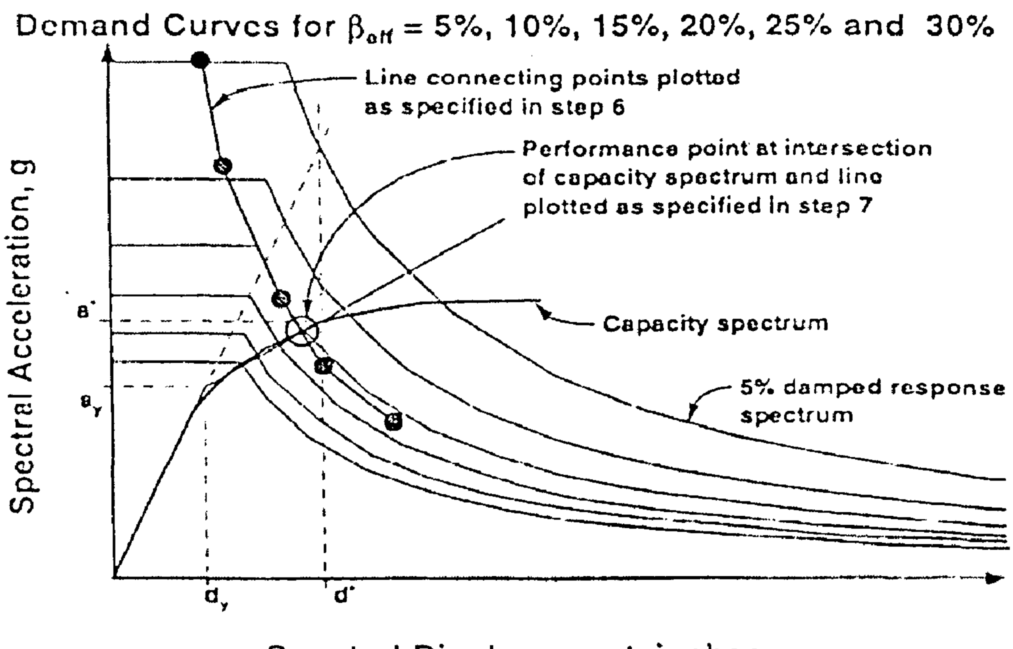


Fig 3.4 performance point evaluation [13]

MODELING, ANALYSIS AND DESIGN

4.1 BUILDING CONFIGURATION

The basic structural configuration used for this dissertation work is a Multistoried (G+5) RC building with symmetricity in its plan (Fig. *4.1*). It has 10 bays in its longitudinal direction and 4 bays in its transverse direction. The corridors possess no transverse beams (only the slab runs over the corridor). Longitudinal direction is mentioned as '*X*' or '*H1*' while the transverse direction is mentioned as '*Z*' or '*H2*'. The building has been assumed to have constructed in seismic zone – IV. Wind loads have not been considered in the study as their effect will be very little over a six-storied building. The design has been conducted with an objective to have optimum sizes of beams and columns. The preliminary data used is given below:

1) Type of structure ------------------ Hospital Building
2) No of stories----------------------- G + 5
3) Floor to Floor height-------------- 3.3m with ground floor height being 3.75m
4) External walls---------------------- 230 mm including plaster
5) Internal walls----------------------- 150 mm including plaster
6) Seismic zone----------------------- IV
7) Basic wind speed------------------ not considered
8) Depth of slab---------------------- 100mm
9) Materials: Steel----------------- Fe-415

   Concrete------------ M25

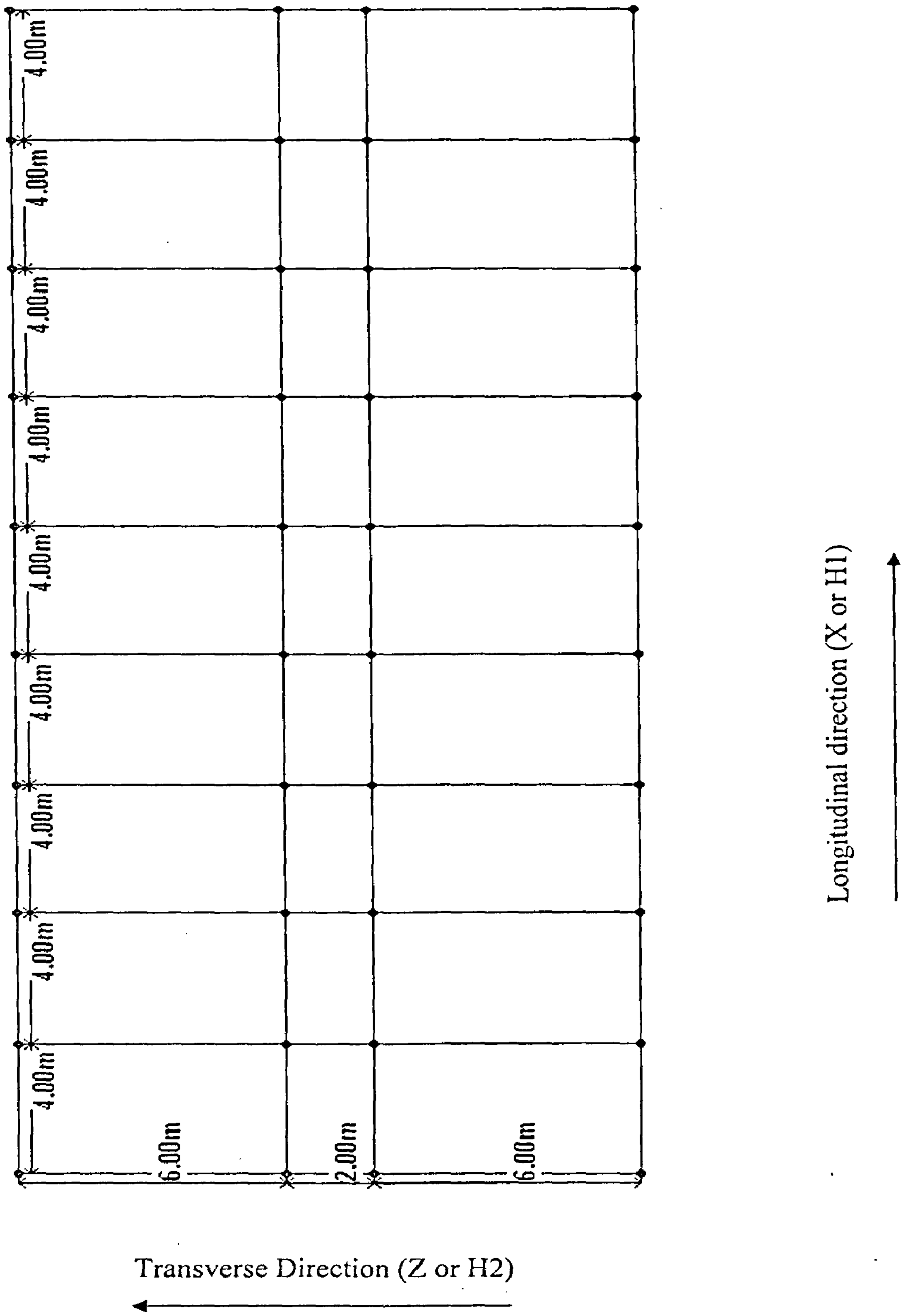


Fig 4.1 Plan details of building

4.1.1 Loads and Load combinations

The unit weights of materials used for the Dead Load calculations (Table *4.1*) are according to IS 875 (part 1): 1987 [14]. The Live loads (Table *4.2*) used for the analysis are obtained from IS 875 (part 2): 1987 [15]. Yield line theory has been used to assign these loads over the structure as trapezoidal, triangular or udl.

For seismic loading, Design Basis Earthquake (DBE) as per IS 1893 (part1):2002 [4] has been used. The specifications used for the analysis are given in Table *4.3*. A response spectrum for medium type of soil has been adopted for the dynamic analysis of the building. The seismic weight considered for the analysis is the sum of Dead load plus 0.25 times the Live load. No live load has been considered over roof. Increase in forces due to accidental torsion has not been considered for the analysis.

Load combinations used for the analysis to calculate the combined effect of dead load (DL), live load (LL) and seismic load (ELX and ELZ) in both directions have been obtained from IS 1893 (part 1):2002. A total number of 17 load combinations as listed below have been used for the analysis and the worst load combinations have been used for the subsequent design.

1)	DL	7)	1.2*(DL+LL-ELX)	13)	1.2*(DL+LL-ELZ)
2)	LL	8)	1.5*(DL+ELX)	14)	1.5*(DL+ELZ)
3)	ELX	9)	1.5*(DL-ELX)	15)	1.5*(DL-ELZ)
4)	ELZ	10)	(0.9DL+1.5ELX)	16)	(0.9DL+1.5ELZ)
5)	1.5(DL+0.6LL)	11)	(0.9DL-1.5ELX)	17)	(0.9DL-1.5ELZ)
6)	1.2*(DL+LL+ELX)	12)	1.2*(DL+LL+ELZ)		

Table 4.1 Dead Load Calculations

Sl. No.	Description	L	B (m)	H (m)	Density (kN/m^3)	Load	Units	Load
1	Panel (4.0x6.0)							
	Longitudinal Beam	Per m	2	0.1	25	5.00	kN/m	Triangular
	Transverse Beam	Per m	2	0.1	25	5.00	kN/m	Trapezoidal
	Panel (2.0 x 4.0)							
	Longitudinal Beam	Per m	1	0.1	25	2.50	kN/m	UDL
2	Exterior Wall Load	Per m	0.2	3.3	18	13.7	kN/m	UDL
	Interior Wall Load	Per m	0.2	3.3	18	8.91	kN/m	UDL
3	Parapet Wall Load	Per m	0.2	1.0	18	2.7	kN/m	UDL
4	Floor Finishes Load	(1.1 kN/m^2)						
	Panel (4.0x6.0)							
	Longitudinal Beam	Per m	2	-	-	2.20	kN/m	Triangular
	Transverse Beam	Per m	2	-	-	2.20	kN/m	Trapezoidal
	Panel (2.0x4.0)							
	Longitudinal Beam	Per m	1	0.055	20	1.10	kN/m	UDL
5	Roof Finishes Load	(1.95 kN/m^2)						
	Panel (4.0x6.0)							
	Longitudinal Beam	Per m	2	-	-	3.90	kN/m	Triangular
	Transverse Beam	Per m	2	-	-	3.90	kN/m	Trapezoidal
	Panel (2.0x4.0)							
	Longitudinal Beam	Per m	1	-	-	1.95	kN/m	UDL

Table 4.2 Live load Calculations

Sl.No.	Description	L	B	Intensity	Load	Units	Load Type
1	**Live Load on Floors**						
	Panel (4.0x6.0)						
	Longitudinal Beam	Per m	2	2 kN/m^2	4.00	kN/m	Triangular
	Transverse Beam	Per m	2	2 kN/m^2	4.00	kN/m	Trapezoidal
	Panel (2.0x4.0)						
	Longitudinal Beam	Per m	1	4 kN/m^2	4.00	kN/m	UDL
2	**Live Load on Roof**						
	Panel (4.0x6.0)						
	Longitudinal Beam	Per m	2	1.5 kN/m^2	3.00	kN/m	Triangular
	Transverse Beam	Per m	2	1.5 kN/m^2	3.00	kN/m	Trapezoidal
	Panel (2.0x4.0)						
	Longitudinal Beam	Per m	1	1.5 kN/m^2	1.50	kN/m	UDL

Table 4.3 Seismic load specifications

Seismic Zone	IV, $Z = 0.24$
Type of soil	Medium type soil
Importance factor, I	1, 1.5
Response reduction factor, R	5

4.2 BUILDING MODELS

Two models of the building described above have been used in the present study namely, Bare frame model and frame with Infills.

4.2.1 Bare frame model

A three dimensional bare frame has been modeled with only beams and columns as the structural components. The supports have been assigned rigid base conditions. The diaphragm has been modeled as a rigid diaphragm.

4.2.2 Frame with infills

A three dimensional model has been made with all the infills of the structure being modeled as compression struts. The thicknesses of the infill struts are, 0.23*m* for outer walls and 0.15*m* for the interior infills. The equivalent width, *a* of these struts have been obtained from FEMA 273 document [12] using the formula:

$$a = 0.175\ (\lambda_l h_{col})^{-0.4}\ r_{inf} \tag{4.1}$$

Where,

$$\lambda_l = \left[\frac{E_{me}\, t_{inf} \sin 2\theta}{4E_{fe}\, I_{col}\, h_{inf}} \right]^{\frac{1}{4}} \tag{4.2}$$

h_{col} = Column height between centerlines of beams, *in.*

h_{inf} = Height of infill panel, *in.*

E_{fe} = Expected modulus of elasticity of frame material, *psi*

E_{me} = Expected modulus of elasticity of infill material, *psi*

I_{col} = Moment of inertia of column, in^4.

L_{inf} = Length of infill panel, *in.*

r_{inf} = Diagonal length of infill panel, *in.*

t_{inf} = Thickness of infill panel and equivalent strut, *in.*

θ = Angle whose tangent is the infill height-to-length aspect ratio, *radians*

λ_l = Coefficient used to determine equivalent width of infill strut

4.3 LINEAR ANALYSIS

The building has been modeled using STAAD Pro software for its Linear Dynamic Analysis and design. Following are the various steps followed in linear analysis.

4.3.1 Modeling for Linear Analysis

A three dimensional bare frame has been modeled (Fig. *4.2*) with beam and column elements. The slab has been modeled as a rigid diaphragm using fictitious rigid truss elements. Fictitious rigid truss elements have also been used in the transverse direction of corridor (Note: the corridor does not possess any beams in its transverse direction and thus rigid truss elements have been used for maintaining the rigid diaphragm action). For infill modeling, equivalent compression struts have been used (Fig. *4.3*).

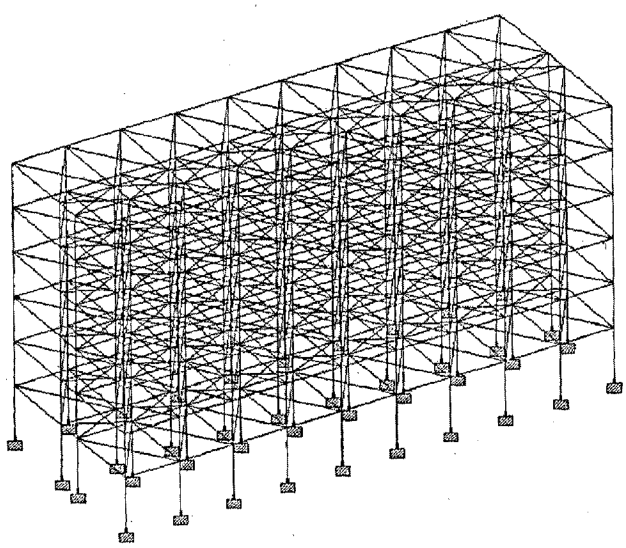


Fig. 4.2 Bare frame model with rigid diaphragm

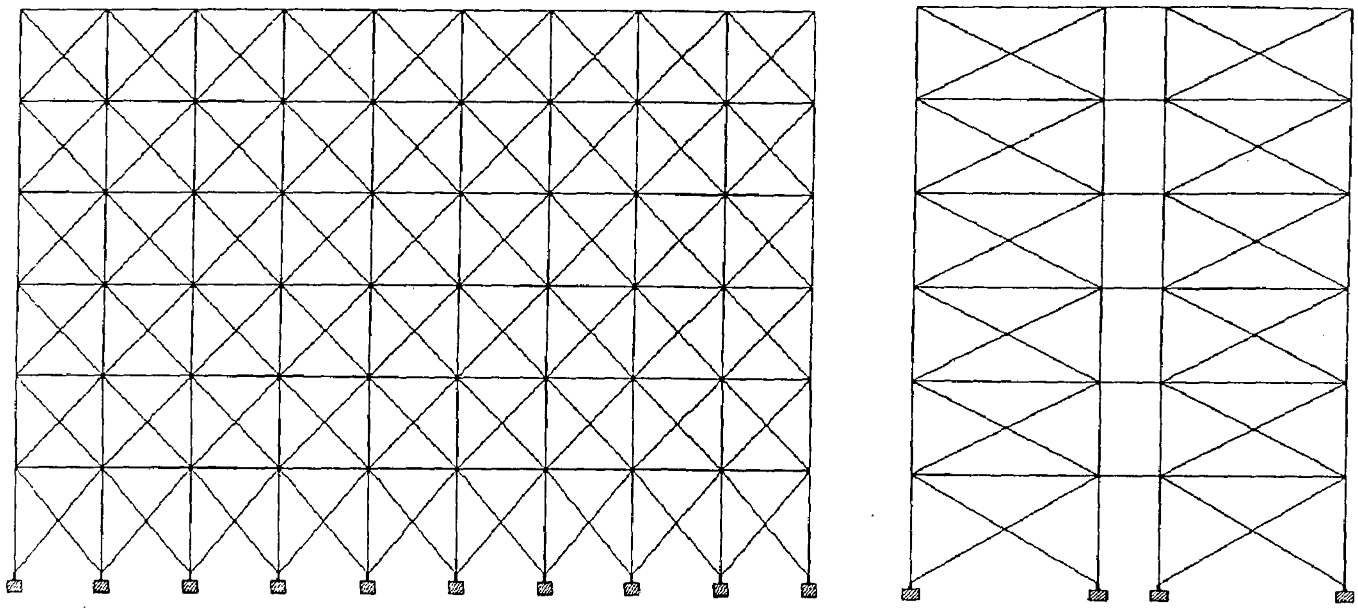


Fig. 4.3 Infill Modeling with Compression Struts

4.3.2 Analysis

The Dead load and Live loads have been applied as per the relevant codes and all load combinations described in 4.1.1 have been used. For the bare frame analysis, the infill loads are calculated and applied as uniformly distributed load over the beams. For seismic loading, two values of Importance Factors, I have been considered as already explained. The linear dynamic analysis has been carried out using response spectrum analysis. Complete Quadratic Combination (CQC) method has been used for the modal combination.

4.3.3 Design

The Limit State design has been performed using the same software and the member sizes and reinforcement ratio are obtained. IS 456:2002 [16] has been followed for the design.

4.4 NONLINEAR ANALYSIS

RAM Perform-3D software has been used for the Nonlinear Analysis in this study. The nonlinear modeling of the RC structure using this software is the most complex part of the analysis as it involves many important issues.

4.4.1 Nonlinear Modeling

The modeling of structure is done using nonlinear elements of beams and columns as described. The slab has been modeled as a horizontal rigid diaphragm.

(a) Beam element

Beam element is modeled as a frame compound component with small axial forces and zero bending moment about vertical axis. A frame compound component for beam essentially consists of one or more basic components. Among the various basic components available from RAM, stiff end zones and FEMA concrete beam have been used for the modeling purpose in this dissertation.

"FEMA 273 concrete beam" is the basic component used to implement the chord rotation model of the beams. Chord rotation model (Fig. *4.4*) is the simplest model with most limitations, which can be used to model the beams and columns. This is a symmetrical beam with equal and opposite end moments. This model requires the nonlinear relationship between the end moment and end rotation be specified. A major advantage using this model is that FEMA-273 gives specific properties, including end rotation capacities.

Fig. 4.4 Chord rotation model [17]

End zones used in RAM Perform-3D are elastic components that are stiff but not rigid. They are used at the ends of frame compound components. The default end zone set in RAM, which can be used for beam and column elements, has a stiffness that is 10 times larger than the body of the component, and an "auto" length that is obtained from the dimensions of the adjacent beams and columns [17].

Though the software contains the option to model beam-to-column connections using linear/nonlinear panel zones, it has not been considered for this study. The beam-to-column connections have been considered to be rigid.

(b) Column element

Column element has been modeled as a frame compound component, which can have substantial axial forces and biaxial bending moment. Frame compound component for column element contains default end zones and "FEMA 273 column component" as basic components. This FEMA 273 column component also uses the chord rotation model for modeling the column elements. In addition to the properties it has got for the case of beam element, it has been modeled for P-M-M interaction also. The component properties for this component can be obtained form FEMA 273 document.

(c) Infill panel element

"Inelastic infill, shear model" (Fig. *4.5*) has been used to model the infill panel components. This component has shear stiffness and strength only. The action for this model is the horizontal shear force and deformation is the shear displacement over the height of the panel.

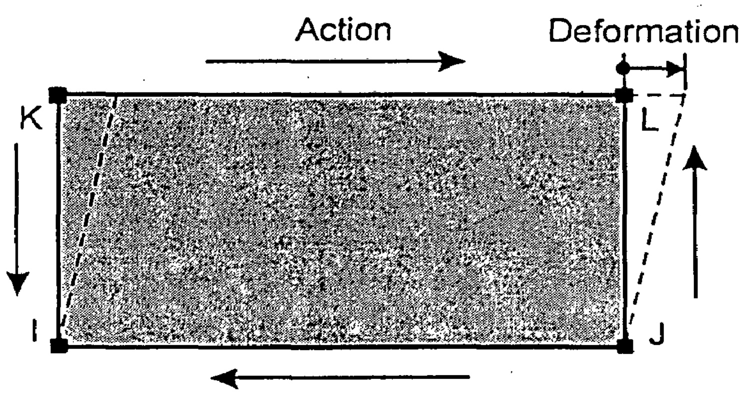


Fig. 4.5 Infill Panel, Shear model [17]

4.4.2 Component properties

The elements modeled in RAM are to be assigned their nonlinear component properties before the actual nonlinear analysis. Some of these component properties depend upon the design results obtained from linear analysis. The required reinforcement area obtained has been used to find out the performance levels of the beam and column elements from FEMA 273 document. Three performance levels have been used in the present analysis, viz. Immediate Occupancy level (IO), Life Safety level (LS) and Collapse Prevention level (CP). The transverse reinforcement in beams and columns has been assumed to be conforming (C). A component is said to be conforming if, within the flexural plastic region, closed stirrups are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (Vs) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming (NC) [9]. The force deformation relationship is assumed Elastic Perfectly Plastic (E-P-P).

The capacities and deformation properties of infill have been obtained from FEMA 273. All the infills in the structure have been modeled with nonlinear diagonal struts and the relevant material properties have been used from the literature [18].

4.4.3 Analysis

Nonlinear Static Pushover analysis has been employed in order to find out the performance levels of the structure for different importance factors and ground shaking levels. For Dead load (DL) and Live load (LL), a load combination of 1.2 times DL plus LL has been used. To apply the pushover load over the structure, nodes have been defined at the geometric centre of each floor. Two load patterns are used for the analyses that are applied one at a time in each direction (*H1* and *H2*). The first load pattern is a linearly varying load pattern (Fig. *4.6*) in which the weights of each floor have been multiplied with their heights from the base and used as a nodal load at the floor levels. The second load pattern is a parabolic load pattern in which the nodal loads have been varied in parabolic fashion over the height of the structure. The decrease in loading at the roof level is because the mass lumped at the roof level is less than that at a typical floor level. All the three limit states have been assigned for beams, columns and drift limit states have been set for the structure to identify where the required performance levels will be reached.

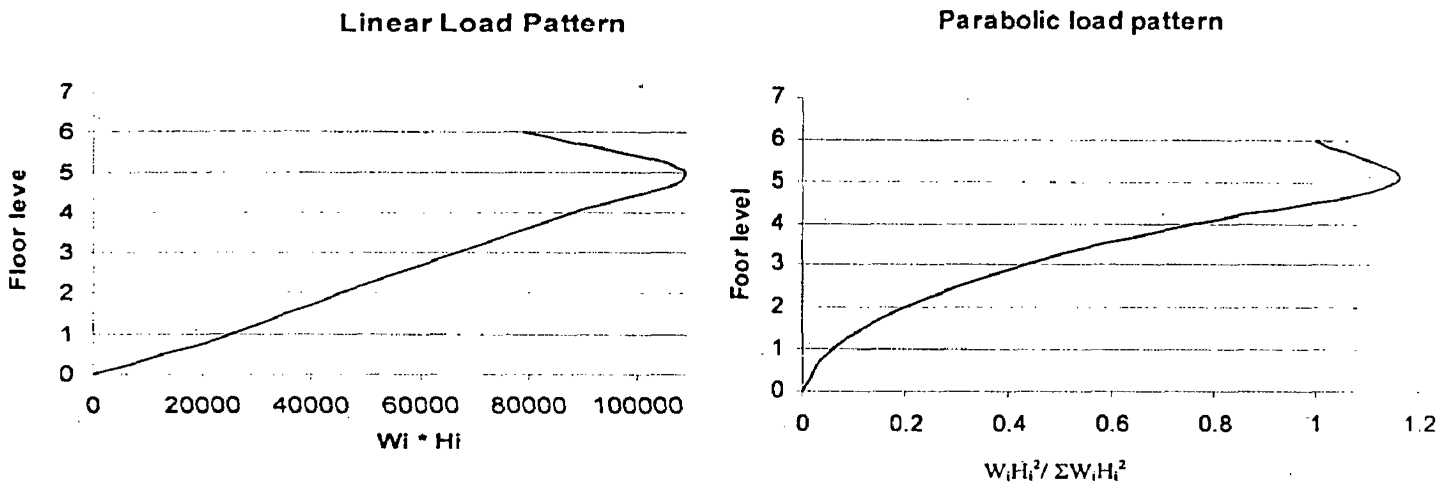
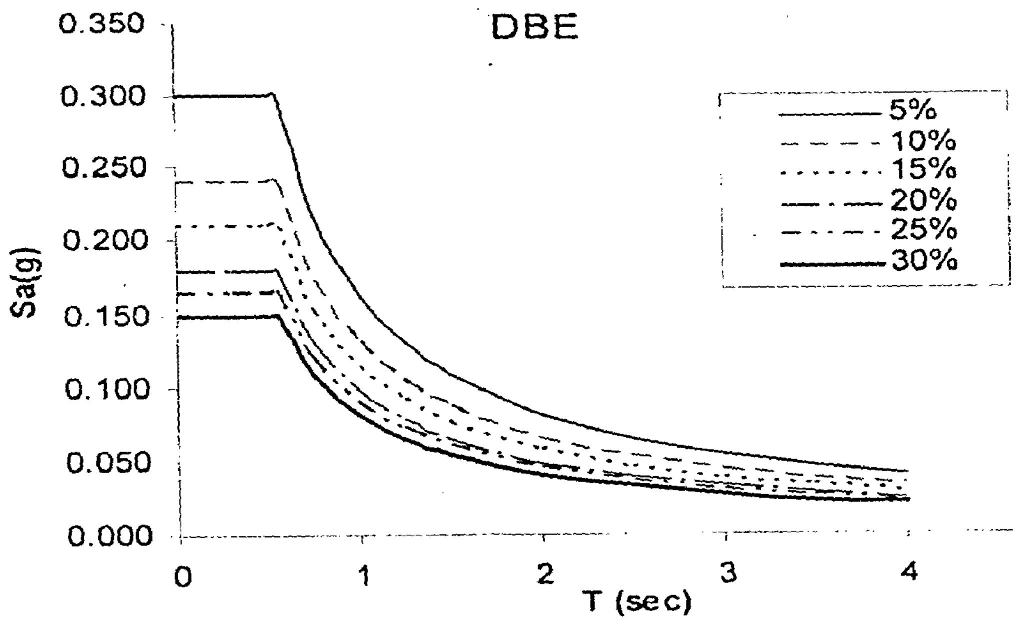
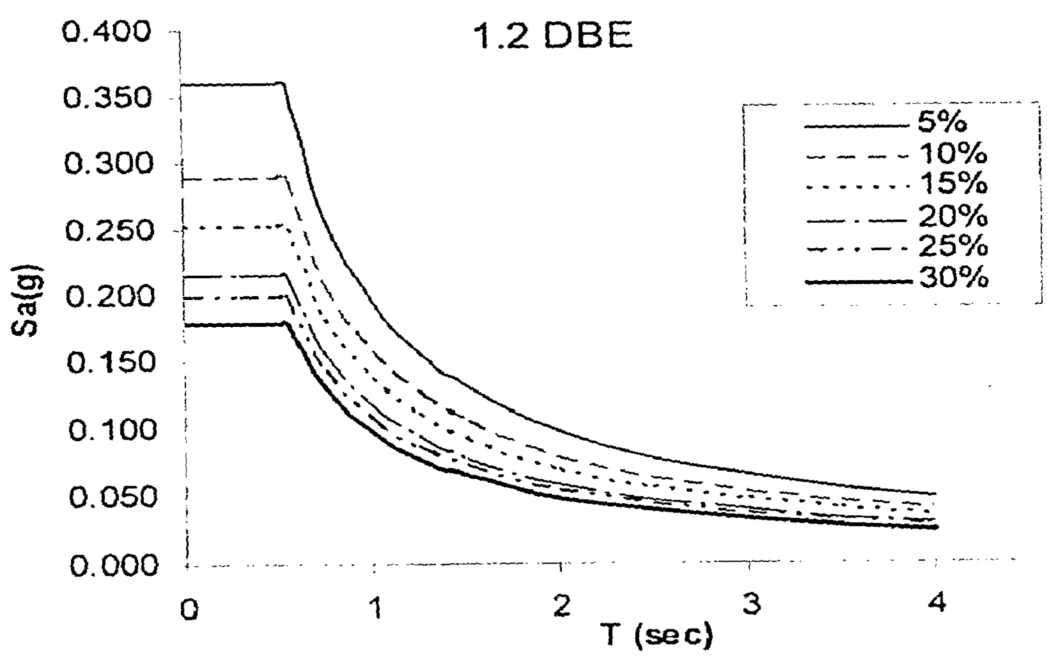


Fig. 4.6 Linear and Parabolic load patterns

The coordinates for demand diagrams of different damping percentages (viz.5%, 10%, 15%, 20%, 25% and 30%) have been obtained using Seismic Zone Factor and Response Spectra given by IS 1893 (Part1):2002. The *Sa (g)* Vs. Time Period plots for Design Basis Earthquake (DBE), 1.2DBE, Maximum Considered Earthquake (MCE) and 1.2MCE have been shown in Fig. *4.7* and *4.8.* The rising limb of the response spectra given by IS code has not been considered for these plots.

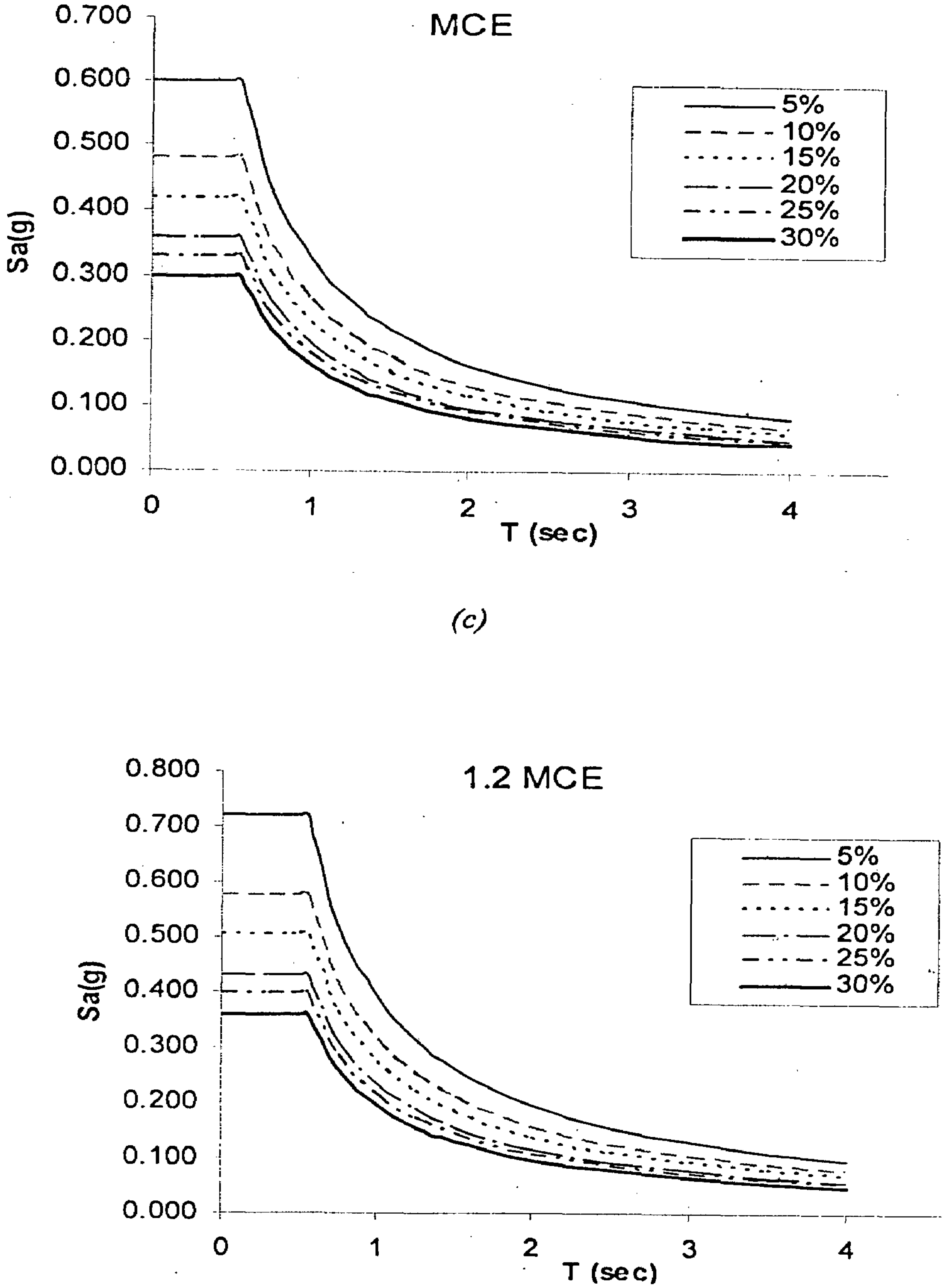


(a)



(b)

Fig. 4.7 Demand diagrams for (a) DBE (b) 1.2 DBE



(c)

(d)

Fig. 4.8 Demand diagrams for (c) MCE (d) 1.2 MCE

PARAMETRIC STUDIES

5.1 METHODOLOGY USED FOR STUDY

The methodology followed for the present study is that the structure ha been designed for the forces obtained in the linear analysis and these design results have been used to conduct the nonlinear pushover analysis in order to assess the performance of the structure in the event of an earthquake.

5.2 BARE FRAME ANALYSIS

5.2.1. Analysis of frame with Unit Importance factor

The Linear analysis has been carried out for the bare frame with $I = 1$ and the results obtained have been used for Nonlinear modeling of the structure and its pushover analysis.

5.2.1.1 Linear Analysis

The first six time periods obtained from the analysis are 1.79, 1.61, 1.39, 0.59, 0.54 and 0.49 seconds respectively. The corresponding six mode shapes are shown in Fig. *5.1*.

The dynamic analysis ha been conducted using the first 15 mode shapes. The dynamic weight obtained for these 15 mode shapes is 34204 *kN* out of the total seismic weight 34434 *kN* used. The base shear obtained in *X*-direction is 720 *kN* while it was 566 *kN* in *Z*-direction. The mass participation factors obtained are shown in Table *5.1*.

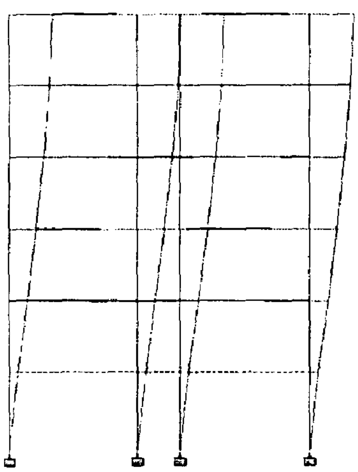
Table 5.1 Mass Participation factors

MASS PARTICIPATION FACTORS IN PERCENT

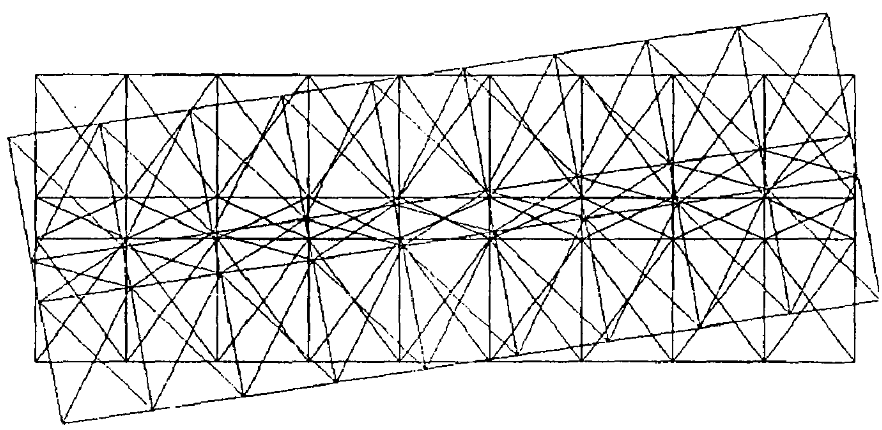
MODE	X	Y	Z	SUMM-X	SUMM-Y	SUMM-Z
1	0.00	0.00	83.62	0.000	0.000	83.615
2	0.00	0.00	0.00	0.000	0.000	83.615
3	86.04	0.00	0.00	86.036	0.000	83.615
4	0.00	0.00	10.18	86.036	0.000	93.797
5	0.00	0.00	0.00	86.036	0.000	93.797
6	9.14	0.00	0.00	95.178	0.000	93.797
7	0.00	0.00	3.73	95.178	0.000	97.527
8	0.00	0.00	0.00	95.178	0.000	97.527
9	3.18	0.00	0.00	98.360	0.000	97.527
10	0.00	0.00	1.32	98.360	0.000	98.844

5.2.1.2 Design

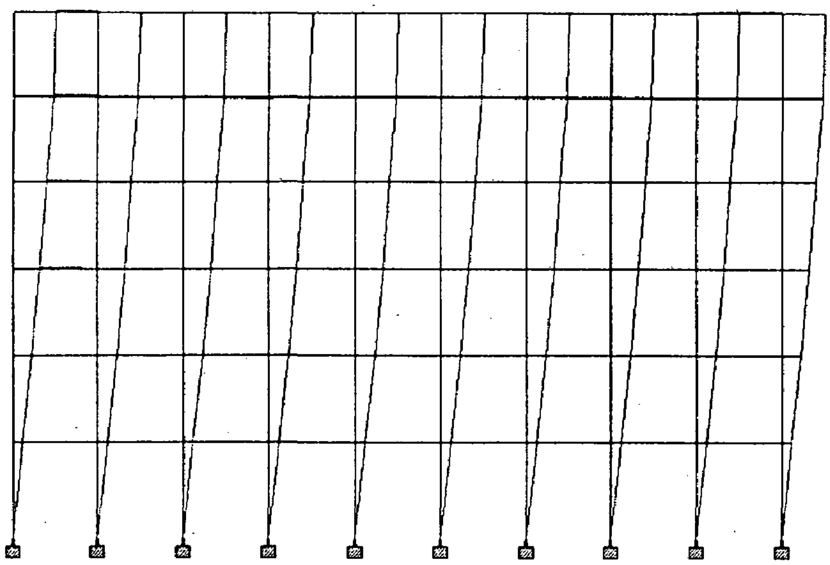
For the forces obtained using linear analysis, design has been performed using the same software and the optimum member sizes have been obtained from the design. The reinforcement percentage provided in beams and columns, and the design axial, shear and moments are tabulated in Table *5.2*.



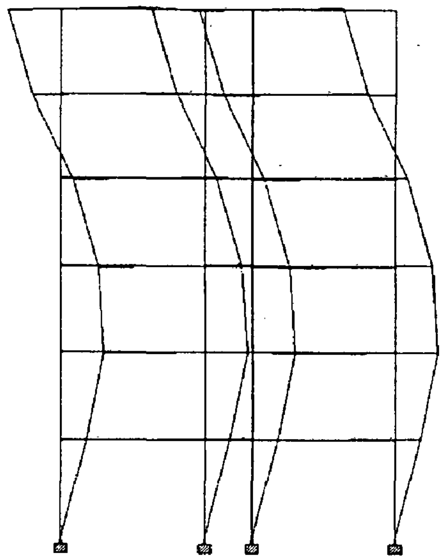
(a) First mode (Transverse)



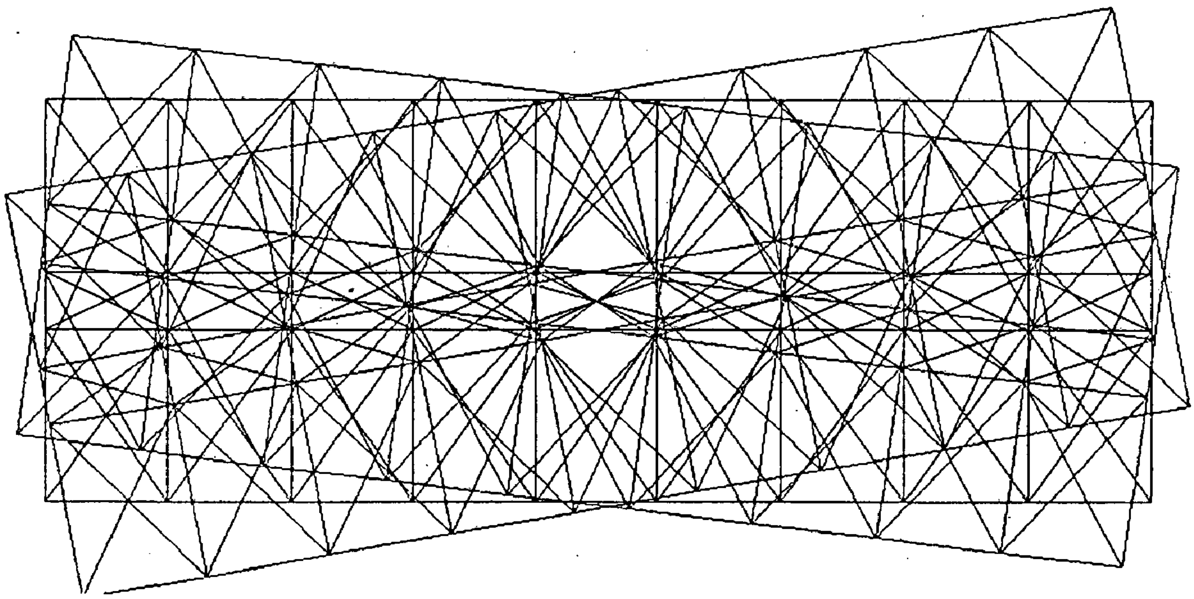
(b) Second mode (Torsional)



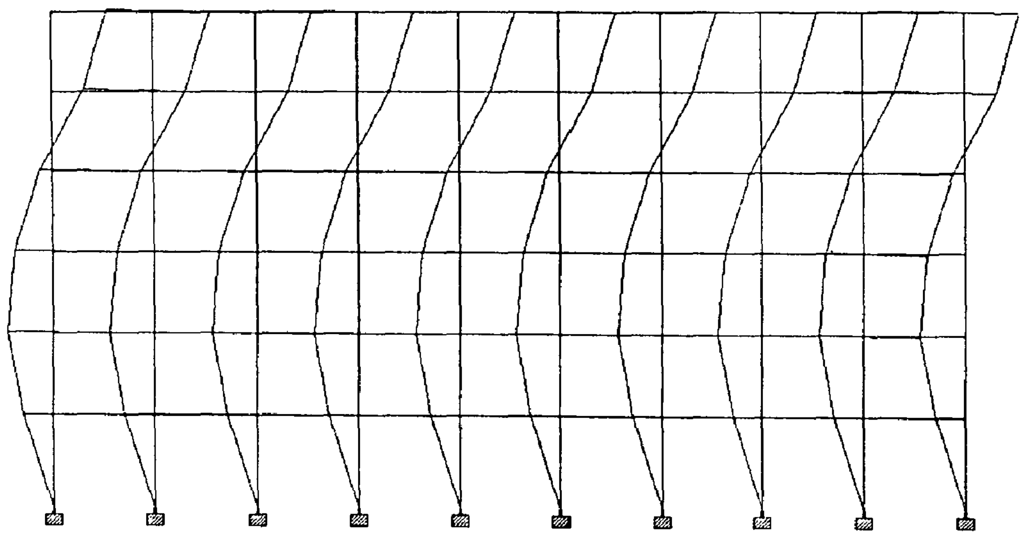
(c) Third mode (Longitudinal)



(d) Fourth mode (Transverse)



(e) Fifth mode (Torsional)



(e) Sixth mode (Longitudinal)

Fig. 5.1 Mode shapes

5.2.1.2 Nonlinear Analysis

In order to conduct the pushover analysis, the component properties for nonlinear modeling have been obtained from the following design results.

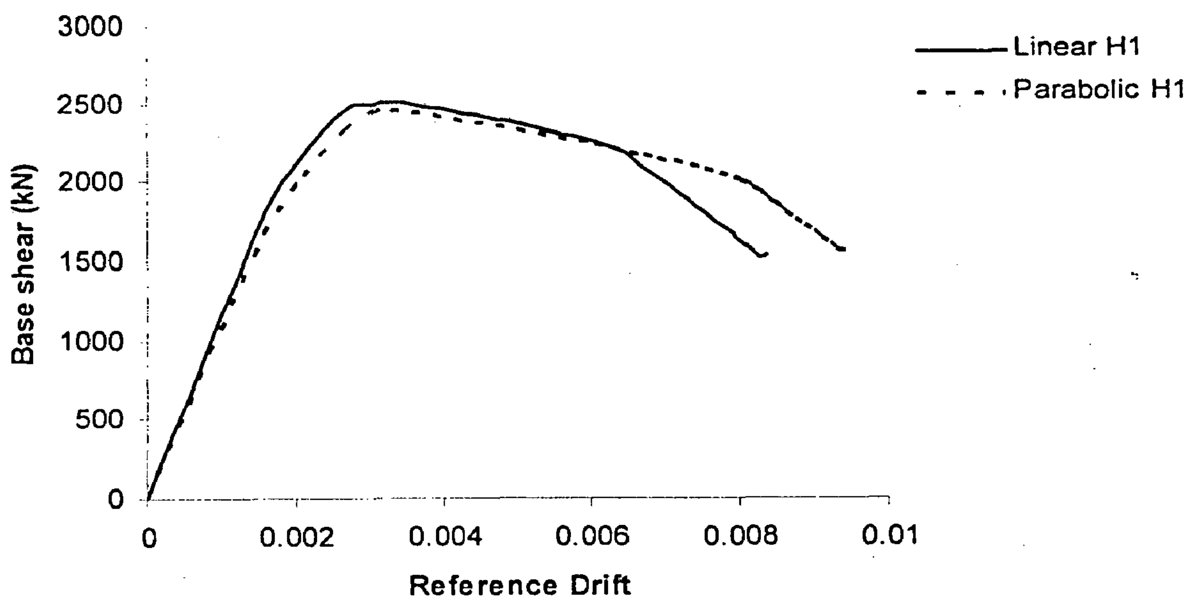
1) The required reinforcement area for beams and columns
2) Design axial load and shear forces for columns, and
3) Design shear forces for beams

The load patterns have been applied separately in both the directions (*H1* and *H2*). Roof drift with respect to base have been taken as reference drifts in both the directions for the analysis. To validate the models, first a linear analysis of the model was performed using Perform 3D also. The time periods and mode shapes were compared with those obtained with STAAD.

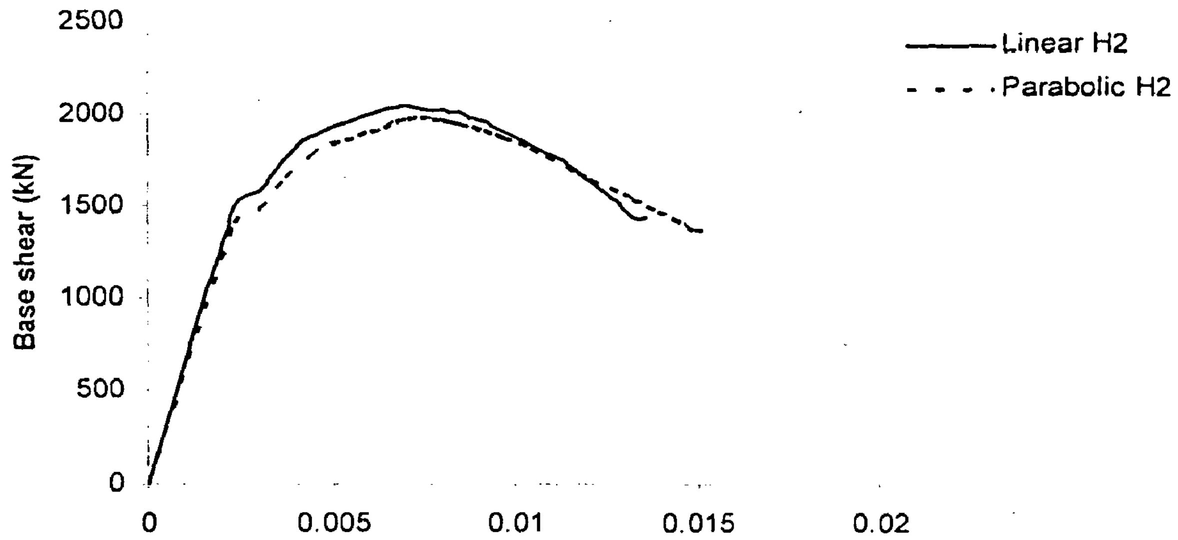
The first six time periods and the corresponding mode shapes have been found to match completely. Hence, it was concluded that the model has been accurately modeled in both the analyses.

(a) Effect of load pattern on pushover curve

The Pushover curve plots showing variation of Maximum base shears with reference drift were obtained using the two predefined load patterns and have been shown in Fig. *5.2*. From the plots, it can be concluded that there is no significant variation of results because of the change in load patterns. Hence, the results have been presented only for linear load pattern in the rest of this study.



(a) Longitudinal direction



(b) Transverse direction

Fig. 5.2 Effect of Load pattern on pushover curve (a) Longitudinal (b) Transverse

(b) Estimation of over-strength in the structure

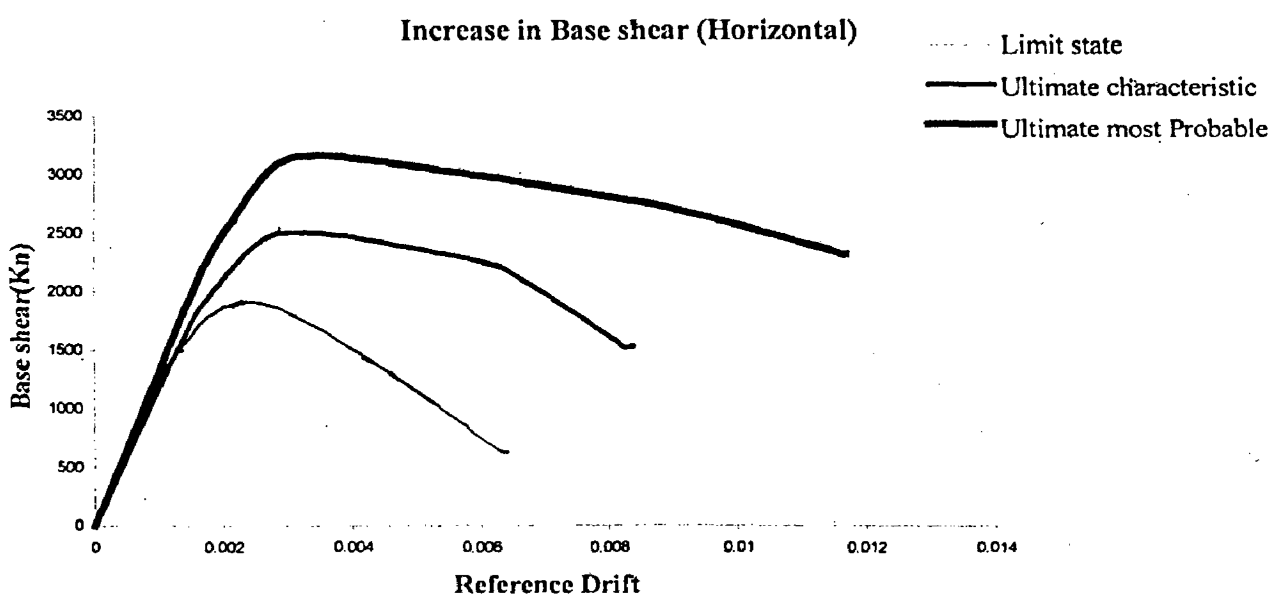
To estimate the over strength present in the structure, three kinds of member capacities have been used for modeling the beams and columns in the nonlinear analysis, namely, Limit State capacities, Ultimate State capacities characteristic and Ultimate Capacities with the most probable strength of materials.

In the first case, the limiting strengths of beams and columns (SP-16 [19]) generally used in the limit state design (Table *5.3*) have been used to find out the performance level of the structure. In the second case, the ultimate strengths of beams and columns, which is the obvious case for nonlinear analysis, are assigned to the members. A FORTRAN programme developed by Ratnesh to find out the ultimate capacities of beams and columns (Table *5.4*) has been used for obtaining ultimate capacity and used in the modeling.

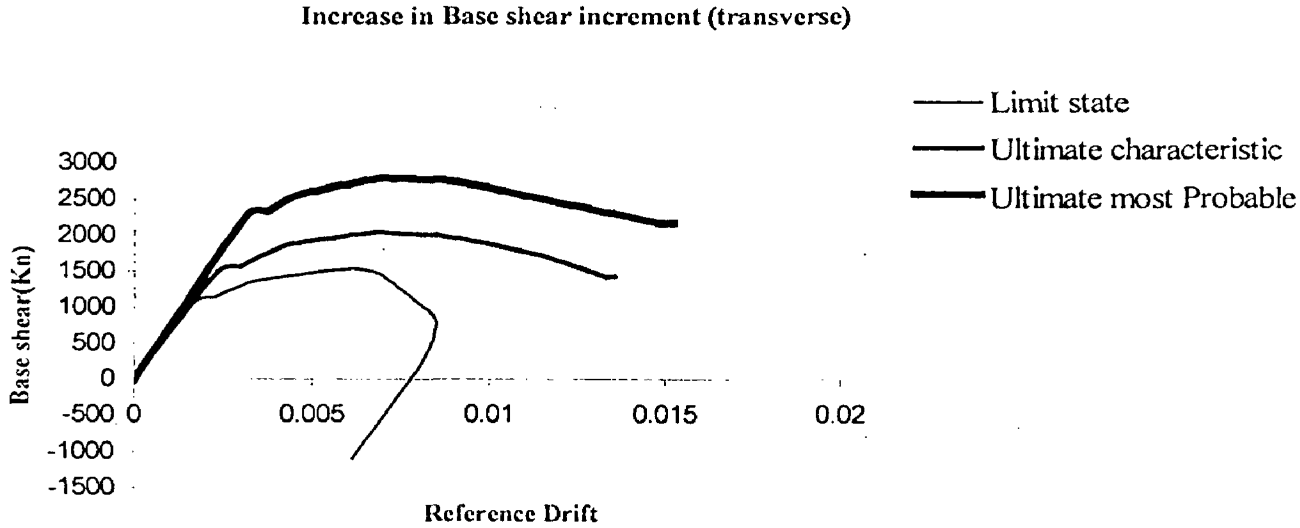
In the two previous cases, the characteristic strengths of concrete and steel with 95% confidence level of the test results (IS 456:2000) have been used for the analysis. However, it is a mere underestimation of the actual strengths of both concrete and steel. In the third case, the mean strengths (50% confidence level) of both concrete [16] and steel [20] have been used as the most probable strengths. The ultimate capacities have been calculated using these most probable strengths for beams and column elements (Table *5.5*) and used in the analysis.

The plots showing the increase in base shear with variations in capacities have been shown in Fig. *5.3*. The maximum base shears obtained from the three cases along *H1* direction (For linearly varying load pattern) are 1890 *kN*, 2505 *kN* and 3168 *kN* respectively. The base shear with ultimate strengths of elements has been found to be 1.325 times to that obtained using limiting strengths.

When the most probable material strengths have been used, the base shear has increased to 1.672 times of that obtained from limit state strengths.



(a)



(b)

Fig. 5.3 Over strength of structure (a) Longitudinal (b) Transverse

(c) Performance points

The performance points have been obtained for different Demands on the structure. Complete plots related to Performance points have been presented in Appendix. Table *5.6* gives the trends of performance levels exhibited by beams, columns and the structure as a whole. For the DBE as a demand diagram, with linear loading pattern, the Performance point in *H1* direction has reached at a base shear of 2420 *kN* with reference drift reaching 0.0026. The columns have reached their Immediate Occupancy (IO) level before the performance point. In *H2* direction, it was at base shear1732 *kN* and drift 0.0036.

For 1.2 DBE also, the columns have reached their IO level in both *H1* and *H2* directions before the performance point is reached. For MCE, in both the directions, columns have reached their Life safety (LS) before the performance point. However, for 1.2 MCE, in *H1* direction, the performance point could not be obtained, which means that the structure will collapse under this earthquake. In *H2* direction also, columns have reached their collapse prevention (CP) level before the performance point is reached.

(d) Effect of imperfect hysteresis loop formation

In the previous analysis, the structure has been assumed to develop a full hysteresis loop under earthquake loading. However, this is an over estimation of the hysteresis damping since the practical experience tells that only degraded hysteresis loop will be formed by structures. ATC 40 considers this degradation of hysteresis loops by applying reduction factors, *k* to equivalent viscous damping. The software used for pushover analysis has an option to apply this degradation effect.

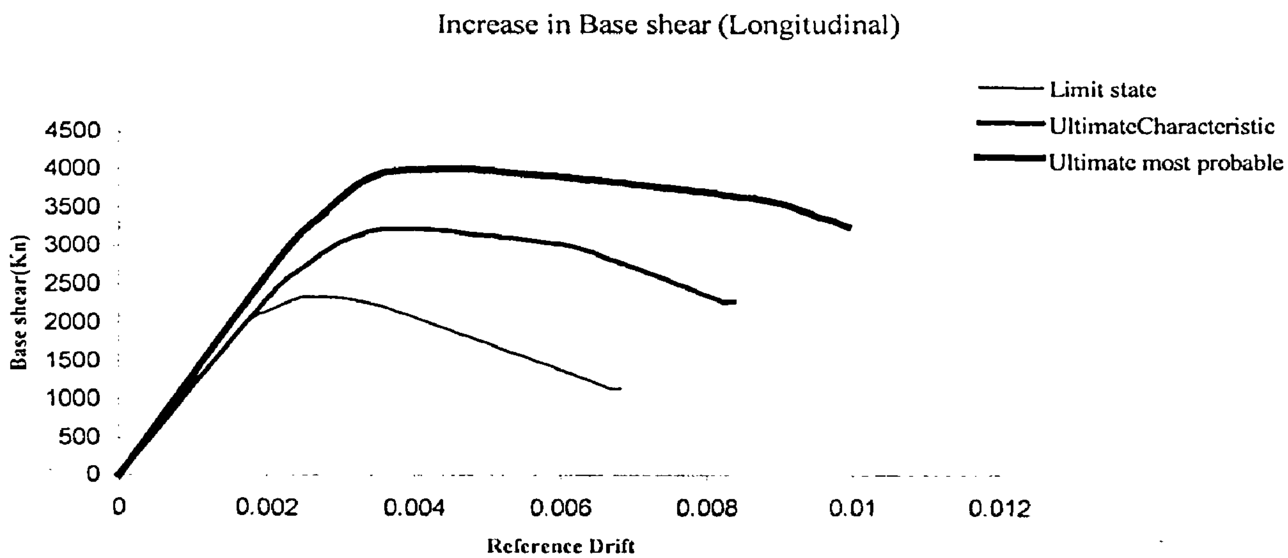
A degradation of 33% has been applied at the member levels for the IS code Demand diagrams and the analysis has been performed. This 33% reduction is equal to reduction factors applied for a 'B' type structure in ATC 40. The drift at performance point for DBE has increased by 11% for the structure with the degraded hysteresis loop.

5.2.2. Analysis of Frame with 1.5 Importance factor

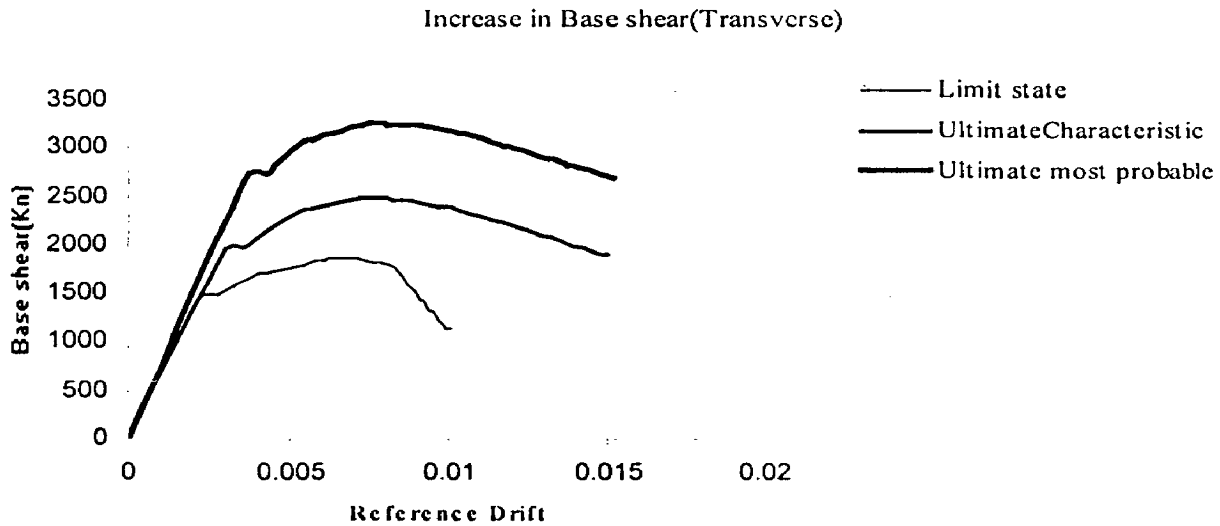
The same loading and model, as in the case of earlier analysis has been used with a modified Importance factor for the seismic loading. While the time period and mode participation factors are same as in the previous case, the base shear obtained in linear dynamic analysis has increased to 1081 *kN* in *X*-direction and to 850 *kN* in Z-direction. These base shears are used for design of the bare frame members (Table *5.7*). The designed bare frame has been used for the following studies:

(a) Estimation of over-strength in the structure

The maximum base shears obtained from the three cases along *H1* direction (For linearly varying load pattern) are 2349 *kN*, 3228 *kN* and 4020 *kN*, respectively. The maximum base shears from nonlinear analysis in all three cases have increased by 1.24, 1.29 and 1.27 times respectively, compared to corresponding base shears for $I = 1$.



(a)



(b)

Fig. 5.4 Over strength of structure (a) Longitudinal (b) Transverse *(b) Performance points*

The summary of performance points for different ground shaking levels has been shown in Table 5.6. For both DBE and 1.2 DBE cases, the trend of Performance point has been the same as in earlier analysis with I = 1. The columns have reached their Immediate Occupancy before the Performance point. The IS 1893 recommends I = 1.5 for important structures like hospital buildings where Immediate Occupancy is needed. However, this objective could not be achieved in the present analysis. For 1.2 MCE, the structure showed an improved performance compared to that designed for I = 1. The performance point has reached before the columns deformed to their collapse prevention level.

5.3 INFILLED FRAME ANALYSIS

The structure has been modeled with compression struts as infills and analyzed for I =1. The infill properties used for model are given in Table 5.8.The first three time periods obtained are 0.38 (transverse mode), 0.26 (torsion mode) and 0.22 (longitudinal mode) respectively. The fundamental time periods calculated manually using the empirical formula given by IS 1893 (Part1): 2002 are 0.30 in longitudinal direction and 0.49 in transverse direction respectively. The time period results obtained from the linear analysis seems to be in fair agreement with the codal values, however the analytical model of the structure is showing a slightly rigid behavior. This rigid behavior may be because all the frames in the structure have been assumed to be infilled, which is rarely the case. Also, the effect of soil on time period of structure has not been considered in the analysis. The base shear obtained has been found as 1788 *kN* in longitudinal direction while it has been 1624 *kN* in transverse direction.

With these values obtained from the analyses, the design of beams and columns have been performed and presented in Table *5.9*.

Inelastic infill shear model has been used to model the infills in the nonlinear analysis. With the infills modeled, the structure has showed an improved performance (Table *5.10*). The columns have reached their IO performance level before the performance point in case of DBE and 1.2DBE. Though the same pattern was also observed in bare frame analysis, the base shears developed in this case are very high with very small reference drifts. For MCE, the structure exhibited Life Safety performance level with the performance point reaching before the column life safety. For 1.2MCE, the columns have reached their life safety before the performance point.

5.3.1 Bare frame with time period modified for infills

From the plan details of the structure, the time period of the structure with infills has been obtained using the empirical formula given by IS 1893(Part1). The Dead load and live loads of the structure have been used to calculate the seismic weight of the structure, from which the base shear has been obtained. This base shear has been used in the linear analysis of a bare frame model of the building and the design of beams and columns has been performed for the modified forces (Table *5.11*).

The designed bare frame has been modeled to conduct the pushover analysis. The performance points obtained from this analysis showed a remarkable improvement (Table *5.10*). The structure showed Immediate Occupancy performance level for DBE and 1.2 DBE. For MCE and 1.2 MCE the structure performed for Life Safety.

Table 5.2 Design specifications for bare frame (for I=1)

COLUMNS						
Floor level	Length (*m*)	Size (*m*)	% steel	Design axial load, (*kN*)	Design moment (*kN-m*)	Design shear force (*kN*)
Ground	3.75	0.35 X 0.35	1.65	1360	69.5	24.7
Second	3.3	0.35 X 0.35	1.65	1090	82.5	33.3
Third	3.3	0.35 X 0.35	0.8	770	74	33.3
Fourth	3.3	0.35 X 0.35	0.8	859	75	34.74
Fifth	3.3	0.3 X 0.3	1.3	350	59	23.83
Roof	3.3	0.3 X 0.3	1.3	350	59	23.83

BEAMS					
Type	Length (*m*)	Size (*m*)	% steel (at both ends)		Design shear force (*kN*)
			Top	Bottom	
Longitudinal	4	0.3 X 0.4	0.64	0.25	125
Transverse	6	0.3 X 0.4	1.16	0.19	155

Table 5.3 Component properties for Bare frame (I=1) (Limit state)

COLUMNS										
FLOOR LEVEL	FEMA 273 PROPERTIES					Column capacities (Ultimate state)				
						Axial (*kN*)			Moment (*kN-m*)	
	a	b	IO	LS	CP	Compression (PC)	Tension (PT)	Balance point (PB)	At Balance point	At P=0
Ground	0.015	0.025	0.000	0.005	0.015	2021	674	551	139	97
Second	0.015	0.025	0.000	0.005	0.015	2021	674	551	139	97
Third	0.017	0.027	0.002	0.007	0.017	1654	306	613	97	48
Fourth	0.017	0.027	0.002	0.007	0.017	1654	306	613	97	48
Fifth	0.018	0.028	0.003	0.008	0.018	1350	371	450	74.25	47.25
Roof	0.018	0.028	0.003	0.008	0.018	1350	371	450	74.25	47.25

BEAMS						
TYPE OF BEAM	FEMA 273 PROPERTIES					Beam Moment capacity (Limit state) (*kN*)
	a	b	IO	LS	CP	
Longitudinal beams	0.025	0.05	0.005	0.020	0.025	93
Transverse beams	0.024	0.049	0.005	0.019	0.024	150

Table 5.4 Component properties for Bare frame (I=1) (Ultimate state)

COLUMNS										
FLOOR LEVEL	FEMA 273 PROPERTIES					Column capacities (Ultimate state)				
						Axial (*kN*)			Moment (*kN-m*)	
	a	b	IO	LS	CP	Compression (PC)	Tension (PT)	Balance point (PB)	At Balance point	At P=0
Ground	0.015	0.025	0.000	0.005	0.015	2735	839	784	152	118
Second	0.015	0.025	0.000	0.005	0.015	2735	839	784	152	118
Third	0.017	0.027	0.002	0.007	0.017	2377	407	893	118	65
Fourth	0.017	0.027	0.002	0.007	0.017	2377	407	893	118	65
Fifth	0.018	0.028	0.003	0.008	0.018	1900	486	622	84	58
Roof	0.018	0.028	0.003	0.008	0.018	1900	486	622	84	58
BEAMS										
TYPE OF BEAM	FEMA 273 PROPERTIES					Beam Moment capacity (Limit state) (*kN*)				
	a	b	IO	LS	CP					
Longitudinal beams	0.025	0.05	0.005	0.020	0.025	108				
Transverse beams	0.024	0.049	0.005	0.019	0.024	179				

Table 5.5 Component properties for Bare frame (I=1) (Ultimate state with most probable strength)

COLUMNS: (Max probable strength for concrete = 32N/mm^2, for steel = 490N/mm^2)										
FLOOR LEVEL	**FEMA 273 PROPERTIES**					**Column capacities (Ultimate state)**				
						Axial (*kN*)			**Moment (*kN-m*)**	
	a	**b**	**IO**	**LS**	**CP**	**Compression (PC)**	**Tension (PT)**	**Balance point (PB)**	**At Balance point**	**At P=0**
Ground	0.015	0.025	0.000	0.005	0.015	3355	990	1060	185	139
Second	0.015	0.025	0.000	0.005	0.015	3355	990	1060	185	139
Third	0.017	0.027	0.002	0.007	0.017	2975	480	1150	146	79
Fourth	0.017	0.027	0.002	0.007	0.017	2975	480	1150	146	79
Fifth	0.018	0.028	0.003	0.008	0.018	2350	573	800	102	71
Roof	0.018	0.028	0.003	0.008	0.018	2350	573	800	102	71
BEAMS: (Max probable strength for concrete = 32N/mm^2, for steel = 490N/mm^2)										
TYPE OF BEAM	**FEMA 273 PROPERTIES**					**Beam Moment capacity (Limit state) (*kN*)**				
	a	**b**	**IO**	**LS**	**CP**					
Longitudinal beams	0.025	0.05	0.005	0.020	0.025	130				
Transverse beams	0.024	0.049	0.005	0.019	0.024	232				

Table 5.6 Performance Levels of Bare frame Building

Importance factor	Level of shaking	Direction	Performance point [Immediate occupancy (IO), Life safety (LS), Collapse Prevention (CP)]
I=1	DBE	Longitudinal (H1)	Columns reached their IO before performance point. Beams and structure are within their IO.
		Transverse (H2)	Columns reached their IO before performance point. Beams and structure are within their IO.
	1.2 DBE	Longitudinal (H1)	Same trend as in case of DBE.
		Transverse (H2)	Same trend as in case of DBE.
	MCE	Longitudinal (H1)	Columns reached their LS. Beams and structure drift reached their IO.
		Transverse (H2)	Both columns and beams reached their LS. Structure reached its IO.
	1.2 MCE	Longitudinal (H1)	No performance point. Structure Unstable.
		Transverse (H2)	Columns reached their CP. Beams reached LS. Structure drift is in its LS.
I=1.5	DBE	Longitudinal (H1)	Columns reached their IO. Beams and structure are within their IO.
		Transverse (H2)	Columns reached their IO before performance point. Beams and structure are within their IO.
	1.2 DBE	Longitudinal (H1)	Same trend as in case of DBE.
		Transverse (H2)	Same trend as in case of DBE.
	MCE	Longitudinal (H1)	Columns reached their LS. Beams reached their IO. Structural drift is within its IO
		Transverse (H2)	Columns reached their LS. Beams reached their IO at the performance point. Structural drift is within its IO
	1.2 MCE	Longitudinal (H1)	Both columns and beams have reached their LS. Structure drift has reached its IO.
		Transverse (H2)	Both columns and beams have reached their LS. Structure drift has reached its IO.

Table 5.7 Design specifications for bare frame (for I=1.5)

COLUMNS						
Floor level	**Length** (*m*)	**Size** (*m*)	**% steel**	**Design axial load** (*kN*)	**Design moment** (*kN-m*)	**Design shear force** (*kN*)
Ground	3.75	0.35 X 0.35	2.48	1290	88	34
Second	3.3	0.35 X 0.35	2.48	1290	88	34
Third	3.3	0.35 X 0.35	1.3	880	93	36
Fourth	3.3	0.35 X 0.35	1.3	880	93	36
Fifth	3.3	0.3 X 0.3	1.89	380	69	28
Roof	3.3	0.3 X 0.3	1.89	380	69	28
BEAMS						
Type	**Length** (*m*)	**Size** (*m*)	**% steel (at both ends)**		**Design shear force** (*kN*)	
			Top	**Bottom**		
Longitudinal	4	0.3 X 0.4	0.87	0.326	99	
Transverse	6	0.3 X 0.4	1.4	0.33	126	

Table 5.8 Infill properties used for the analysis (for I=1)

INFILLS						
Type	**Length** (*m*)	**Equivalent width of compression strut** (*m*)	**Thickness of Infill** (*m*)	**Modulus of Elasticity** (kN/m^2)	**Shear stiffness (G.A/H)** (*kN/m*)	**Shear strength** (*kN/m2*)
Longitudinal External Infill	5.19	0.41	0.23	1.38 E+07	1.67 E+06	138
Longitudinal Internal infill	5.19	0.43	0.15	1.38 E+07	1.09 E+06	138
Transverse External infill	6.85	0.57	0.23	1.38 E+07	2.51 E+06	138
Transverse internal infill	6.85	0.59	0.15	1.38 E+07	1.64 E+06	138

Table 5.9 Design specifications for frame with infills (for I=1)

COLUMNS						
Floor level	**Length** (*m*)	**Size** (*m*)	**% steel**	**Design axial load** (*kN*)	**Design moment** (*kN-m*)	**Design shear force** (*kN*)
Ground	3.75	0.35 X 0.35	0.7	1160	20	07
Second	3.3	0.35 X 0.35	0.7	1160	20	07
Third	3.3	0.35 X 0.35	0.45	770	65	26
Fourth	3.3	0.35 X 0.35	0.45	770	65	26
Fifth	3.3	0.3 X 0.3	0.75	300	46	19
Roof	3.3	0.3 X 0.3	1.67	118	60	24
BEAMS						
Type	**Length** (*m*)	**Size** (*m*)	**% steel (at both ends)**		**Design shear force** (*kN*)	
			Top	**Bottom**		
Longitudinal	4	0.3 X 0.4	0.36	0	70	
Transverse	6	0.3 X 0.4	0.19	0	114	

Table 5.10 Performance Levels of Frame with Infills (for I=1)

Model type	Level of shaking	Direction	Performance point [Immediate occupancy (IO), Life safety (LS), Collapse Prevention (CP)]
Infills modeled for analysis	DBE	Longitudinal (H1)	Columns reached their IO before performance point. Beams and structure are within their IO.
		Transverse (H2)	Columns reached their IO before performance point. Beams and structure are within their IO.
	1.2 DBE	Longitudinal (H1)	Same trend as in case of DBE.
		Transverse (H2)	Same trend as in case of DBE.
	MCE	Longitudinal (H1)	Columns and beams reached their IO. Structure drift is below its IO.
		Transverse (H2)	Columns and beams reached their IO. Structure drift is below its IO.
	1.2 MCE	Longitudinal (H1)	Columns have reached their LS level. Beams reached their IO level. Structure drift is within its IO level.
		Transverse (H2)	Both beams and columns reached their LS level. Structural drift is within its IO level.
Base shear obtained by IS code formula is used for the design of Bare frame and pushover analysis is carried out.	DBE	Longitudinal (H1)	The performance point is well within the IO level of members as well as structure drift.
		Transverse (H2)	The performance point is well within the IO level of members as well as structure drift
	1.2 DBE	Longitudinal (H1)	Same trend as in case of DBE.
		Transverse (H2)	Same trend as in case of DBE.
	MCE	Longitudinal (H1)	Performance point reached at beam IO. Structure and columns are within their IO.
		Transverse (H2)	Only beams have reached their IO before performance point.
	1.2 MCE	Longitudinal (H1)	Beams have reached their IO. Columns and structure are within their IO.
		Transverse (H2)	Beams have reached their IO. Columns and structure are within their IO.

Table 5.11 Design specifications for Bare frame designed for codal base shear (for I=1)

COLUMNS						
Floor level	**Length (*m*)**	**Size (*m*)**	**% steel**	**Design axial load (*kN*)**	**Design moment (*kN-m*)**	**Design shear force (*kN*)**
Ground	3.75	0.6 X 0.6	0.8	590	340	120
Second	3.3	0.5 X 0.5	0.7	470	330	133
Third	3.3	0.5 X 0.5	0.7	470	330	133
Fourth	3.3	0.5 X 0.5	0.7	470	330	133
Fifth	3.3	0.5 X 0.5	0.7	470	330	133
Roof	3.3	0.5 X 0.5	0.6	160	130	52

BEAMS					
Type	**Length (*m*)**	**Size (*m*)**	**% steel (at both ends)**		**Design shear force (*kN*)**
			Top	**Bottom**	
Longitudinal	4	0.3 X 0.5	0.8	0.5	126
Transverse	6	0.3 X 0.5	1.3	0.5	155

CONCLUSIONS

1) Four buildings with different Importance factors and with and without infills have been analyzed using Linear analysis, designed using IS 456 and the same models have been analyzed using Nonlinear Static Pushover analysis to estimate their performance points.

2) The models have been validated by cross checking the results obtained from two different softwares.

3) There is no significant variation in pushover curve plots with the change in load patterns. So any load pattern- Linear or Parabolic can be used for the analysis.

4) Degradation of the hysteresis loop by 33% moved the performance point by 11% of the lateral drift compared to that of a non-degraded hysteresis loop.

5) It has been observed that buildings have considerable over strength and ductility. The base shear for the bare frame with ultimate strengths of elements has been obtained as 1.325 times that obtained using limiting strengths. This means that the use of limit strength for the design under-estimates the capacities of the members by more than 30%. When the most probable strength of the materials have been used, the base shear has

increased by 67% compared to that obtained from limit strengths corresponding to characteristic strength of materials.

6) For bare frame designed with unit Importance factor (I=1), the structure exhibited the Life Safety performance level for DBE and 1.2DBE. For MCE, the structure performed for collapse prevention level. For 1.2 MCE, the structure has become completely unstable. This trend completely matches with the IS code design philosophy.

7) For bare frame designed for I = 1.5, contrary to code, the structure could perform only Life safety under DBE and 1.2DBE conditions. However, for MCE and 1.2MCE, the structure has exhibited collapse prevention. It suggests that the Importance Factor used by IS code for structures with post earthquake importance is not sufficient.

8) In the linear analysis of frame with infills modeled as compression struts, the time period results obtained from the linear analysis seems to be in fair agreement with the codal values.

9) From the pushover analysis of frame with infills, it could be seen that the performance levels have been significantly improved from that of a bare frame. For MCE, the infilled frame structure exhibited Life Safety performance level and for 1.2MCE, the infilled model exhibited Collapse Prevention level.

10) The bare frame model designed for codal base shear exhibited a remarkable improvement. The structure showed Immediate occupancy performance level for DBE and 1.2 DBE. For MCE and 1.2 MCE the structure performed for Life safety. This indicates that the structure has to be modeled necessarily with infills.

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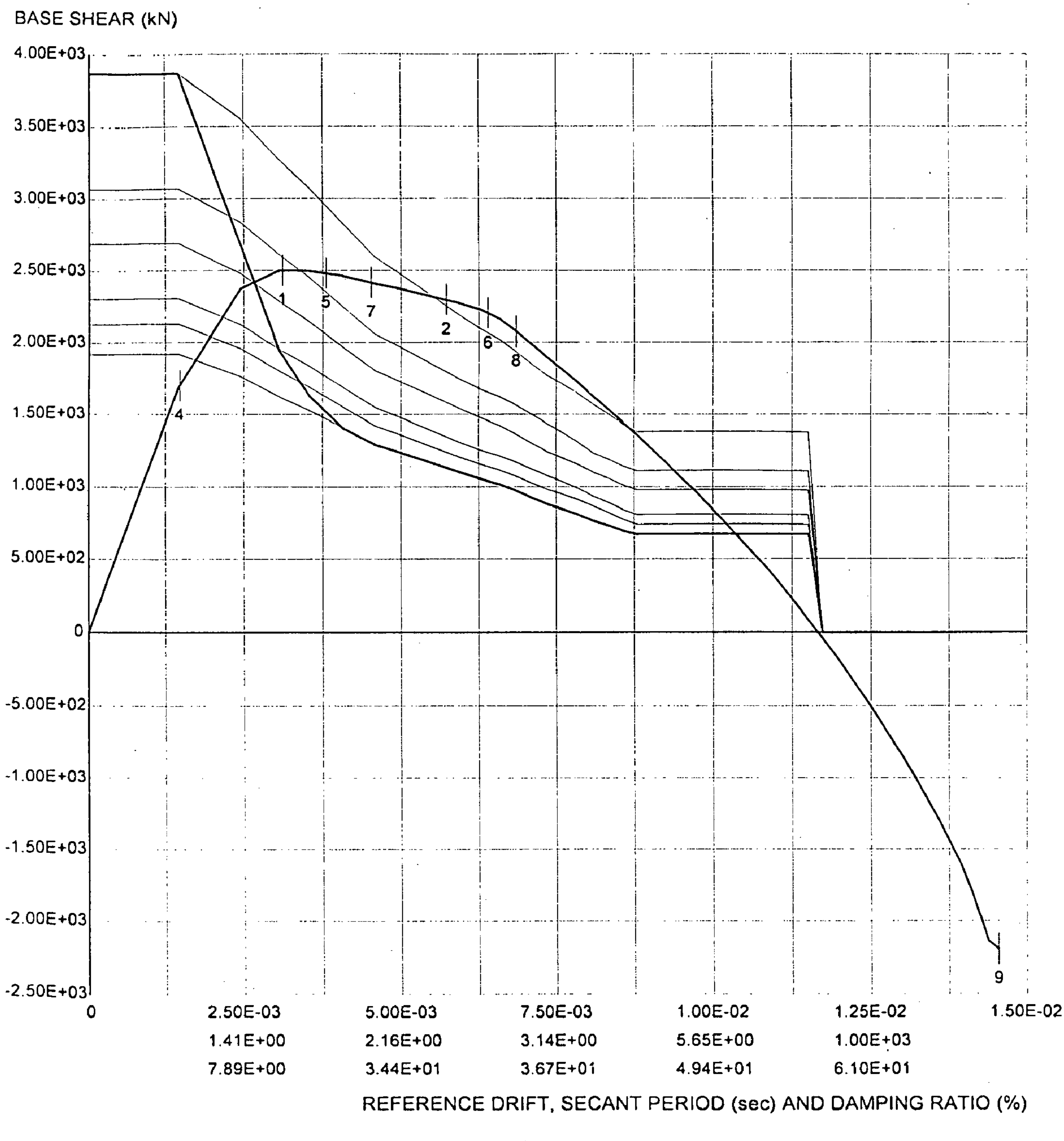
APPENDIX

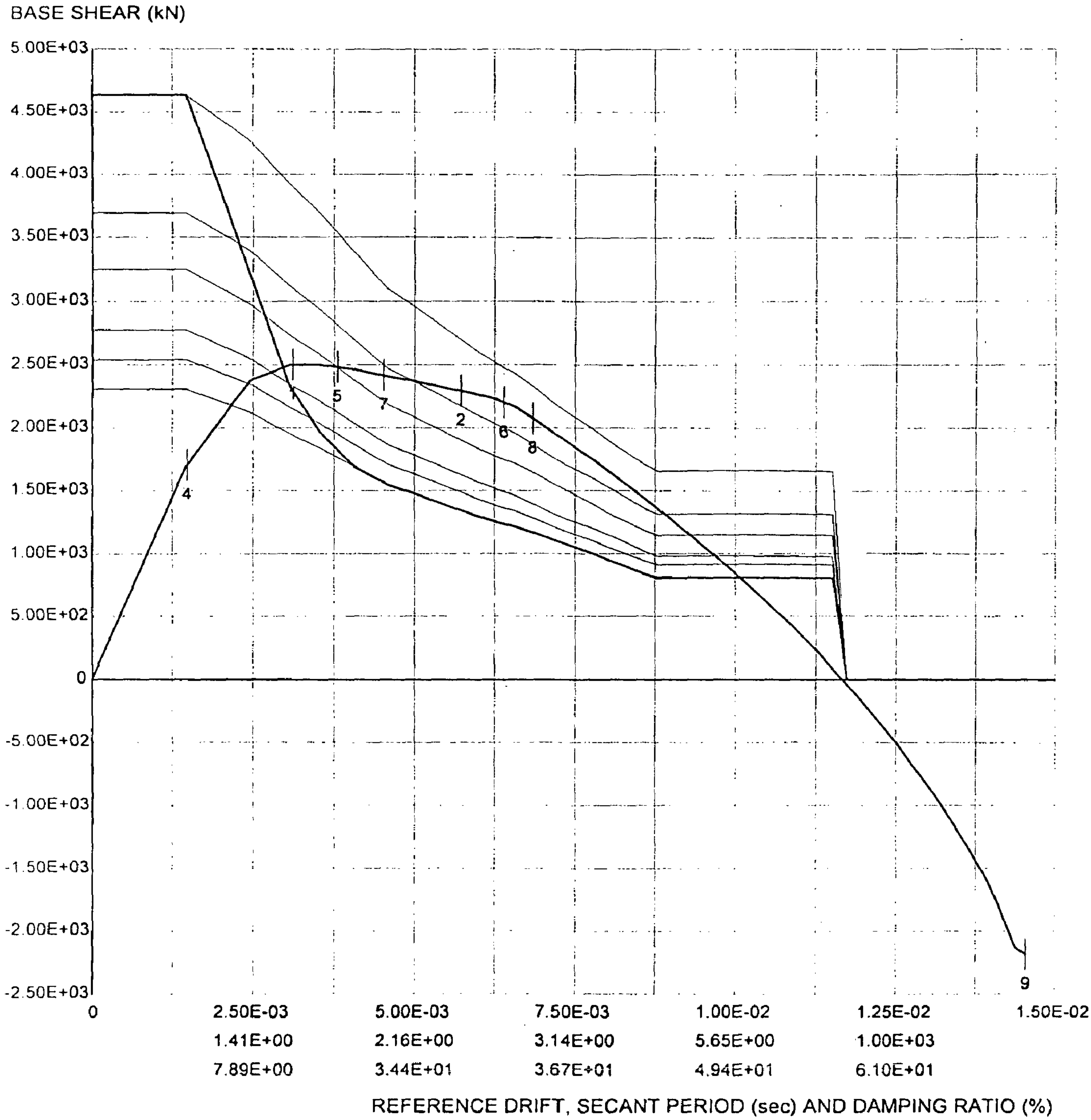
In this Appendix, the Pushover results corresponding to Linear Pushover load pattern in longitudinal direction are presented. Four types of ground motions namely DBE, 1.2 DBE, MCE and 1.2MCE have been used for obtaining the Demand Diagrams that are in turn used to obtain the performance points using Capacity Spectrum Method. The plots for bare frame designed for unit Importance factor are given in A-1, A-2, A-3 and A-4 graphs. The plots for the bare frame with 1.5 as Importance factor are plotted in B-1, B-2, B-3 and B-4 graphs. The plots for frame with Infills are given in C-1, C-2, C-3 and C-4 graphs. The limit state lists for these models are presented in the Table below and are shown in the corresponding plots by their numbers.

No.	Limit States		
	Bare frame designed for $I = 1$	**Bare frame designed for $I = 1.5$**	**Frame with Infills designed for $I = 1$**
1	Beam IO	Beam IO	Beam IO
2	Beam LS	Beam LS	Beam LS
3	Beam CP	Beam CP	Beam CP
4	Column IO	Column IO	Column IO
5	Column LS	Column LS	Column LS
6	Column CP	Column CP	Column CP
7	Structure Drift IO	Structure Drift IO	Infill LS
8	Structure Drift LS	Structure Drift LS	Structure Drift IO
9	Structure Drift CP	Structure Drift CP	Structure Drift LS
10	-	-	Structure Drift CP

A-1 DBE

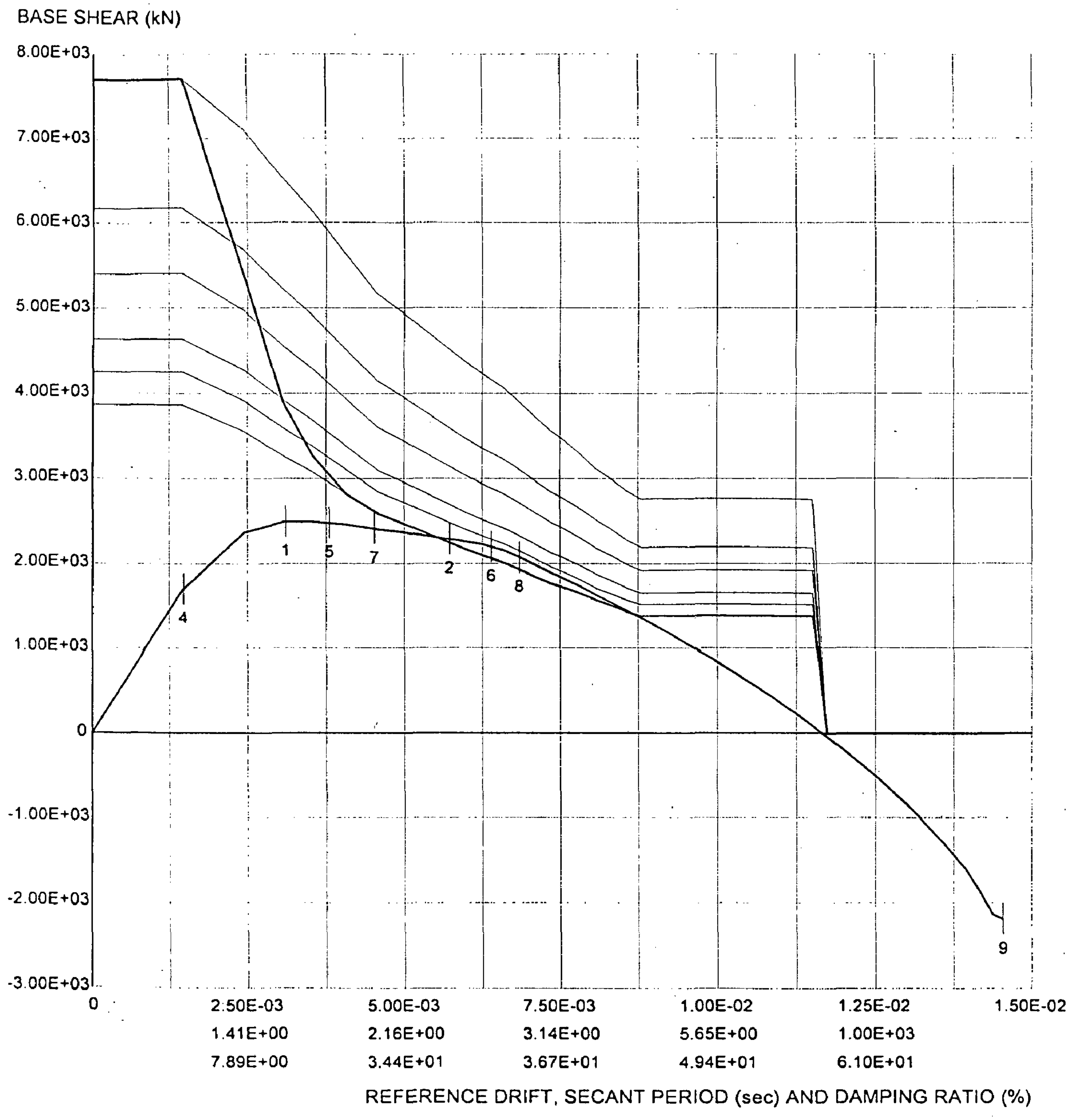

BASE SHEAR (kN)
4.00E+03
3.50E+03
3.00E+03
2.50E+03
2.00E+03
1.50E+03
1.00E+03
5.00E+02
0
-5.00E+02
-1.00E+03
-1.50E+03
-2.00E+03
-2.50E+03
1
5
7
2
6
8
4
9
0
2.50E-03
5.00E-03
7.50E-03
1.00E-02
1.25E-02
1.50E-02
1.41E+00
2.16E+00
3.14E+00
5.65E+00
1.00E+03
7.89E+00
3.44E+01
3.67E+01
4.94E+01
6.10E+01
REFERENCE DRIFT, SECANT PERIOD (sec) AND DAMPING RATIO (%)

A-2 1.2DBE

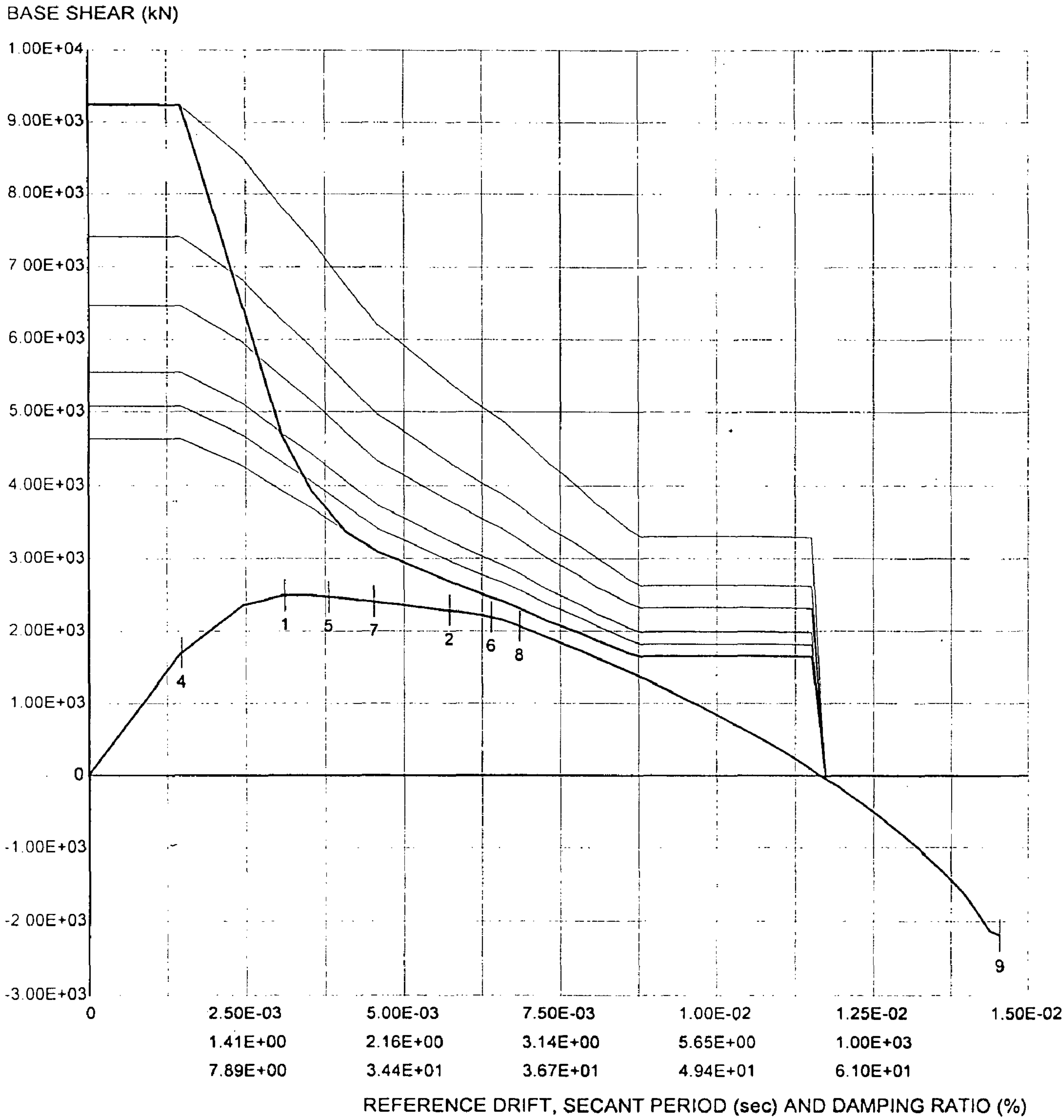


BASE SHEAR (kN)
5.00E+03
4.50E+03
4.00E+03
3.50E+03
3.00E+03
2.50E+03
2.00E+03
1.50E+03
1.00E+03
5.00E+02
0
-5.00E+02
-1.00E+03
-1.50E+03
-2.00E+03
-2.50E+03
1
2
4
5
6
7
8
9
0
2.50E-03
5.00E-03
7.50E-03
1.00E-02
1.25E-02
1.50E-02
1.41E+00
2.16E+00
3.14E+00
5.65E+00
1.00E+03
7.89E+00
3.44E+01
3.67E+01
4.94E+01
6.10E+01
REFERENCE DRIFT, SECANT PERIOD (sec) AND DAMPING RATIO (%)

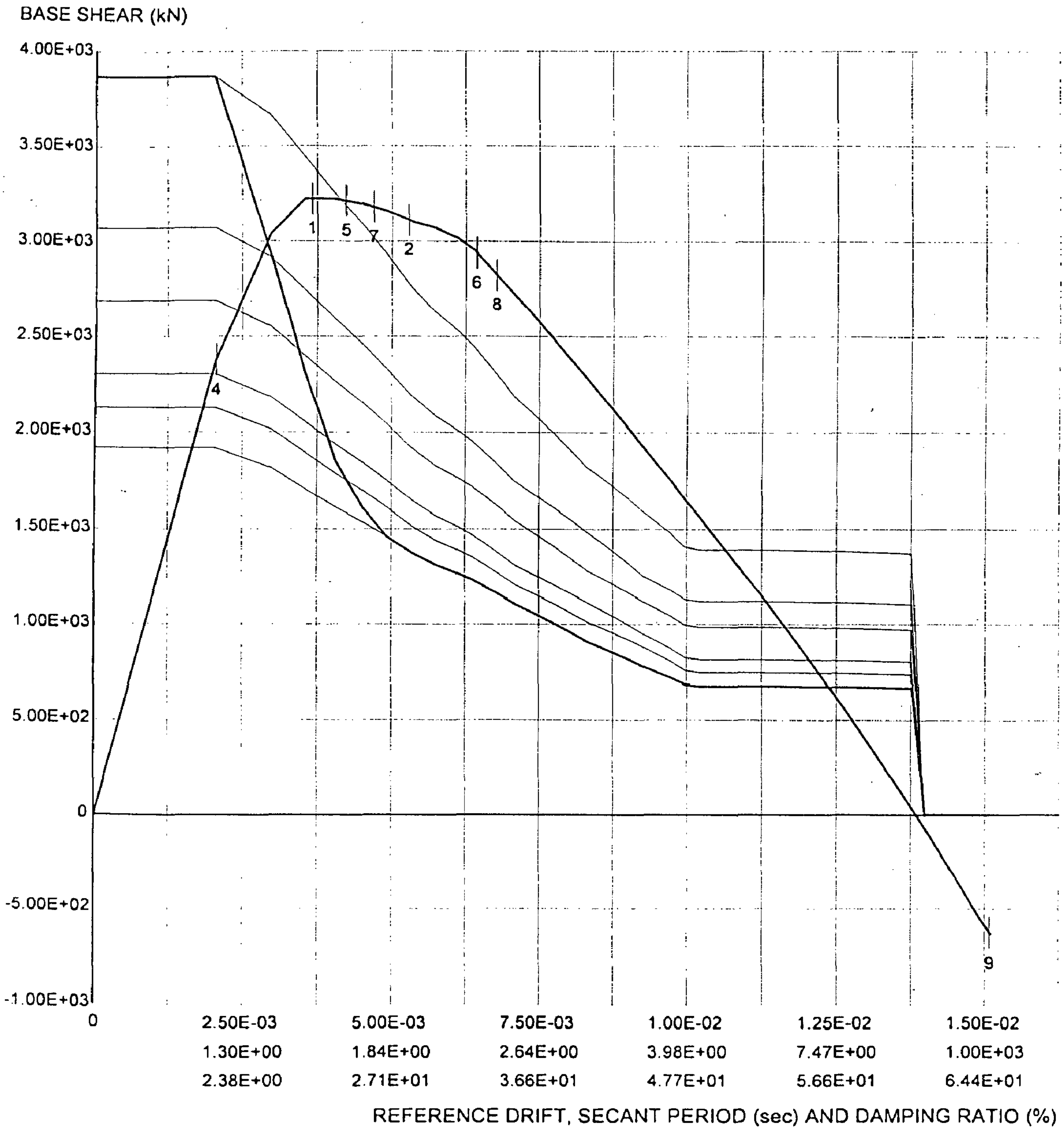
A-3 MCE


BASE SHEAR (kN)
8.00E+03
7.00E+03
6.00E+03
5.00E+03
4.00E+03
3.00E+03
2.00E+03
1.00E+03
0
-1.00E+03
-2.00E+03
-3.00E+03
1
5
7
2
6
8
4
9
0
2.50E-03
5.00E-03
7.50E-03
1.00E-02
1.25E-02
1.50E-02
1.41E+00
2.16E+00
3.14E+00
5.65E+00
1.00E+03
7.89E+00
3.44E+01
3.67E+01
4.94E+01
6.10E+01
REFERENCE DRIFT, SECANT PERIOD (sec) AND DAMPING RATIO (%)

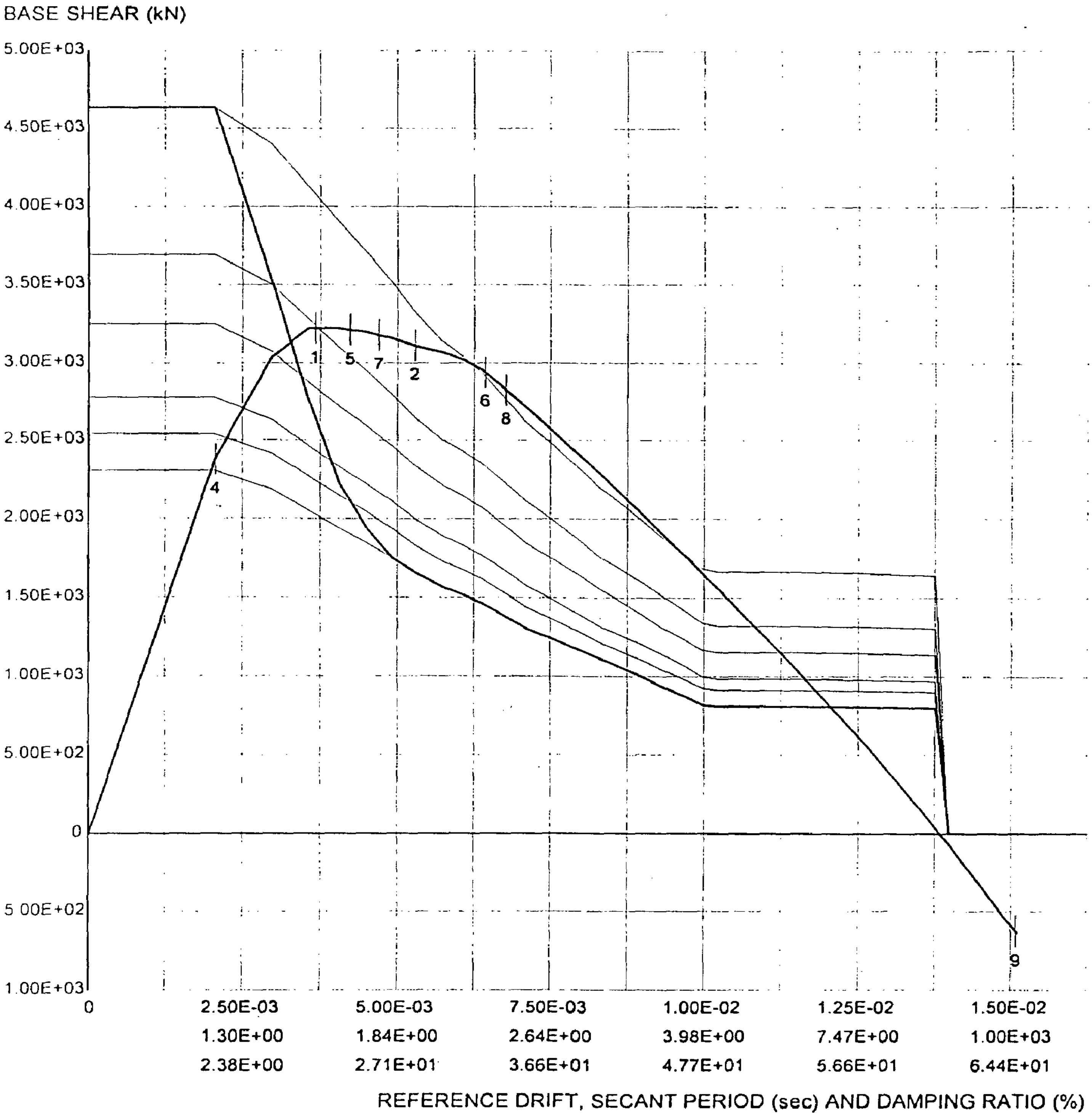
A-4 1.2 MCE


BASE SHEAR (kN)
1.00E+04
9.00E+03
8.00E+03
7.00E+03
6.00E+03
5.00E+03
4.00E+03
3.00E+03
2.00E+03
1.00E+03
0
-1.00E+03
-2.00E+03
-3.00E+03
1
5
7
2
6
8
4
9
0
2.50E-03
5.00E-03
7.50E-03
1.00E-02
1.25E-02
1.50E-02
1.41E+00
2.16E+00
3.14E+00
5.65E+00
1.00E+03
7.89E+00
3.44E+01
3.67E+01
4.94E+01
6.10E+01
REFERENCE DRIFT, SECANT PERIOD (sec) AND DAMPING RATIO (%)

B-1 DBE

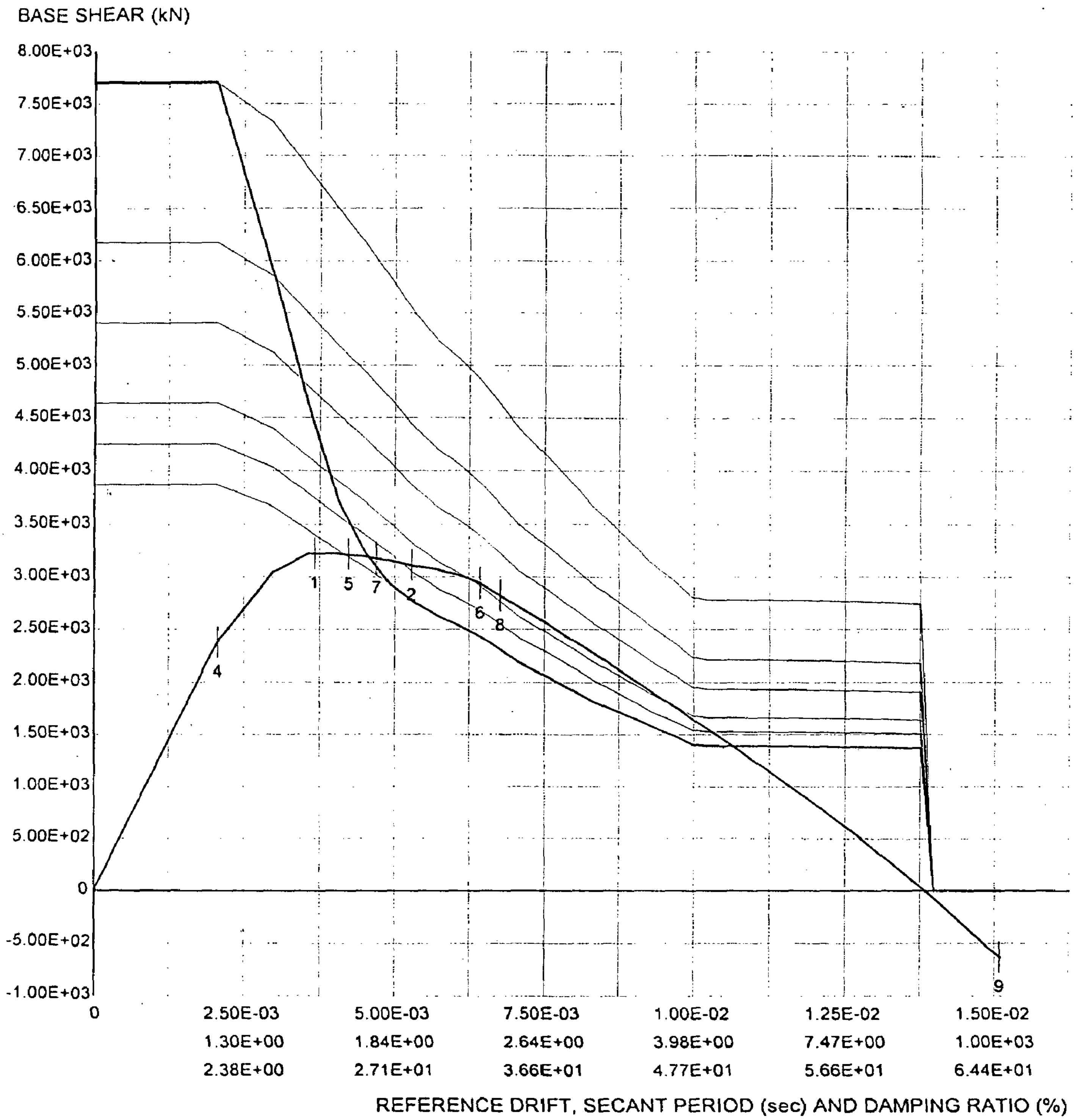

BASE SHEAR (kN)
4.00E+03
3.50E+03
3.00E+03
2.50E+03
2.00E+03
1.50E+03
1.00E+03
5.00E+02
0
-5.00E+02
-1.00E+03
1
5
7
2
6
8
4
9
0
2.50E-03
5.00E-03
7.50E-03
1.00E-02
1.25E-02
1.50E-02
1.30E+00
1.84E+00
2.64E+00
3.98E+00
7.47E+00
1.00E+03
2.38E+00
2.71E+01
3.66E+01
4.77E+01
5.66E+01
6.44E+01
REFERENCE DRIFT, SECANT PERIOD (sec) AND DAMPING RATIO (%)

B-2 1.2 DBE

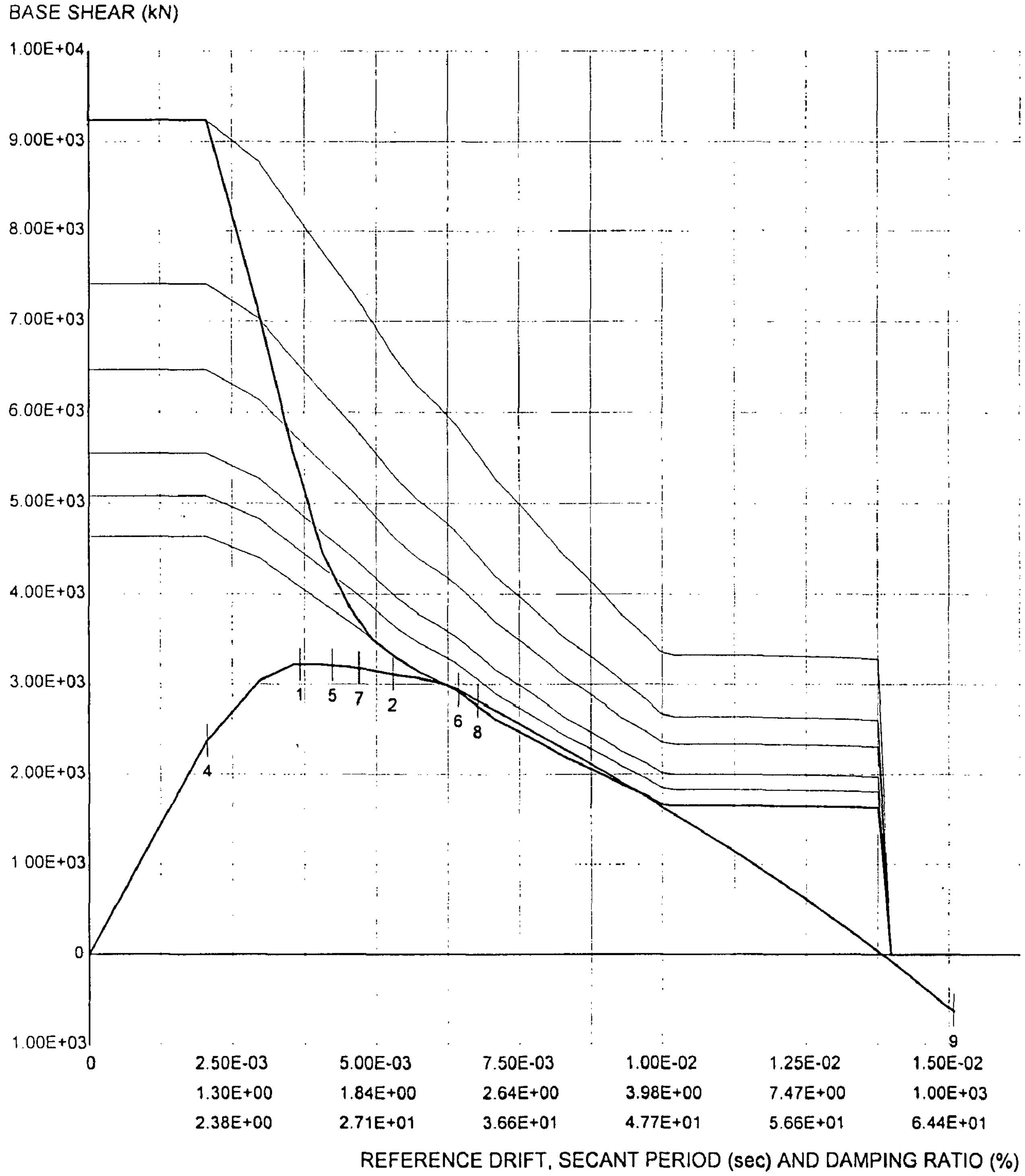


BASE SHEAR (kN)
5.00E+03
4.50E+03
4.00E+03
3.50E+03
3.00E+03
2.50E+03
2.00E+03
1.50E+03
1.00E+03
5.00E+02
0
5.00E+02
1.00E+03
1
5
7
2
6
8
4
9
0
2.50E-03
5.00E-03
7.50E-03
1.00E-02
1.25E-02
1.50E-02
1.30E+00
1.84E+00
2.64E+00
3.98E+00
7.47E+00
1.00E+03
2.38E+00
2.71E+01
3.66E+01
4.77E+01
5.66E+01
6.44E+01
REFERENCE DRIFT, SECANT PERIOD (sec) AND DAMPING RATIO (%)

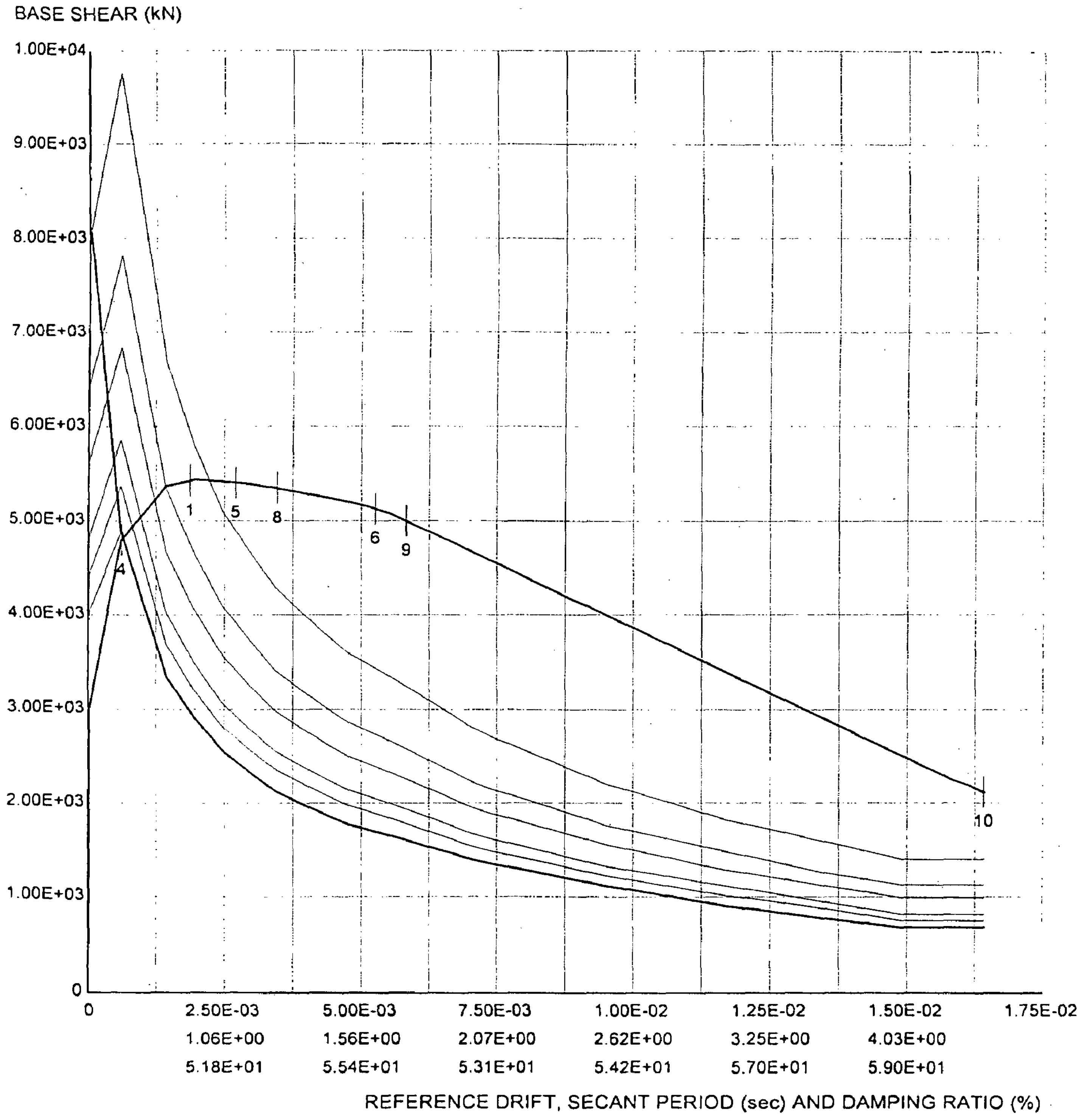
B-3 MCE


BASE SHEAR (kN)
8.00E+03
7.50E+03
7.00E+03
6.50E+03
6.00E+03
5.50E+03
5.00E+03
4.50E+03
4.00E+03
3.50E+03
3.00E+03
2.50E+03
2.00E+03
1.50E+03
1.00E+03
5.00E+02
0
-5.00E+02
-1.00E+03
1
5
7
2
6
8
4
9
0
2.50E-03
5.00E-03
7.50E-03
1.00E-02
1.25E-02
1.50E-02
1.30E+00
1.84E+00
2.64E+00
3.98E+00
7.47E+00
1.00E+03
2.38E+00
2.71E+01
3.66E+01
4.77E+01
5.66E+01
6.44E+01
REFERENCE DRIFT, SECANT PERIOD (sec) AND DAMPING RATIO (%)

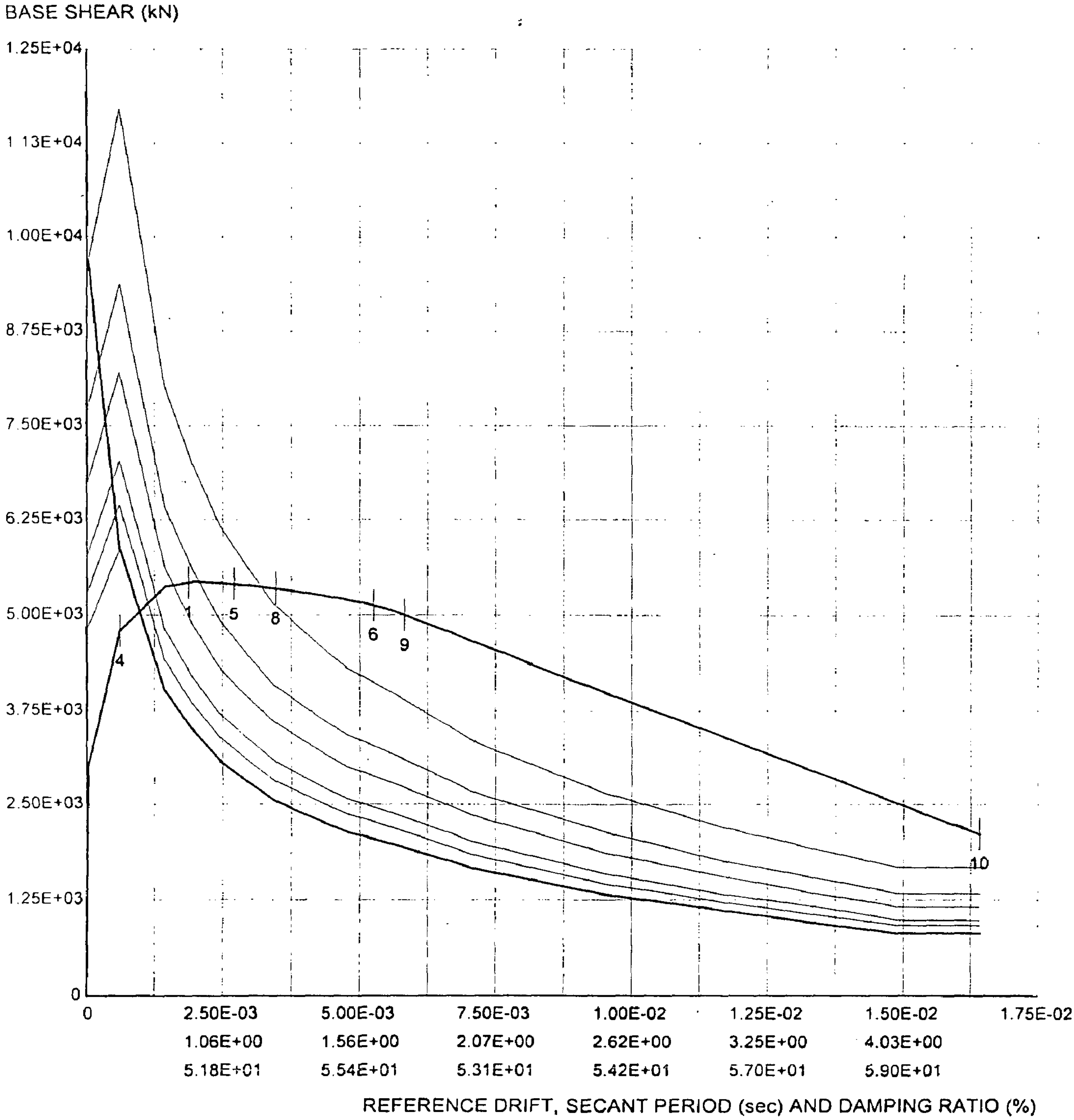
B-4 1.2MCE


BASE SHEAR (kN)
1.00E+04
9.00E+03
8.00E+03
7.00E+03
6.00E+03
5.00E+03
4.00E+03
3.00E+03
2.00E+03
1 00E+03
0
1.00E+03
1
5
7
2
6
8
4
9
0
2.50E-03
5.00E-03
7.50E-03
1.00E-02
1.25E-02
1.50E-02
1.30E+00
1.84E+00
2.64E+00
3.98E+00
7.47E+00
1.00E+03
2.38E+00
2.71E+01
3.66E+01
4.77E+01
5.66E+01
6.44E+01
REFERENCE DRIFT, SECANT PERIOD (sec) AND DAMPING RATIO (%)

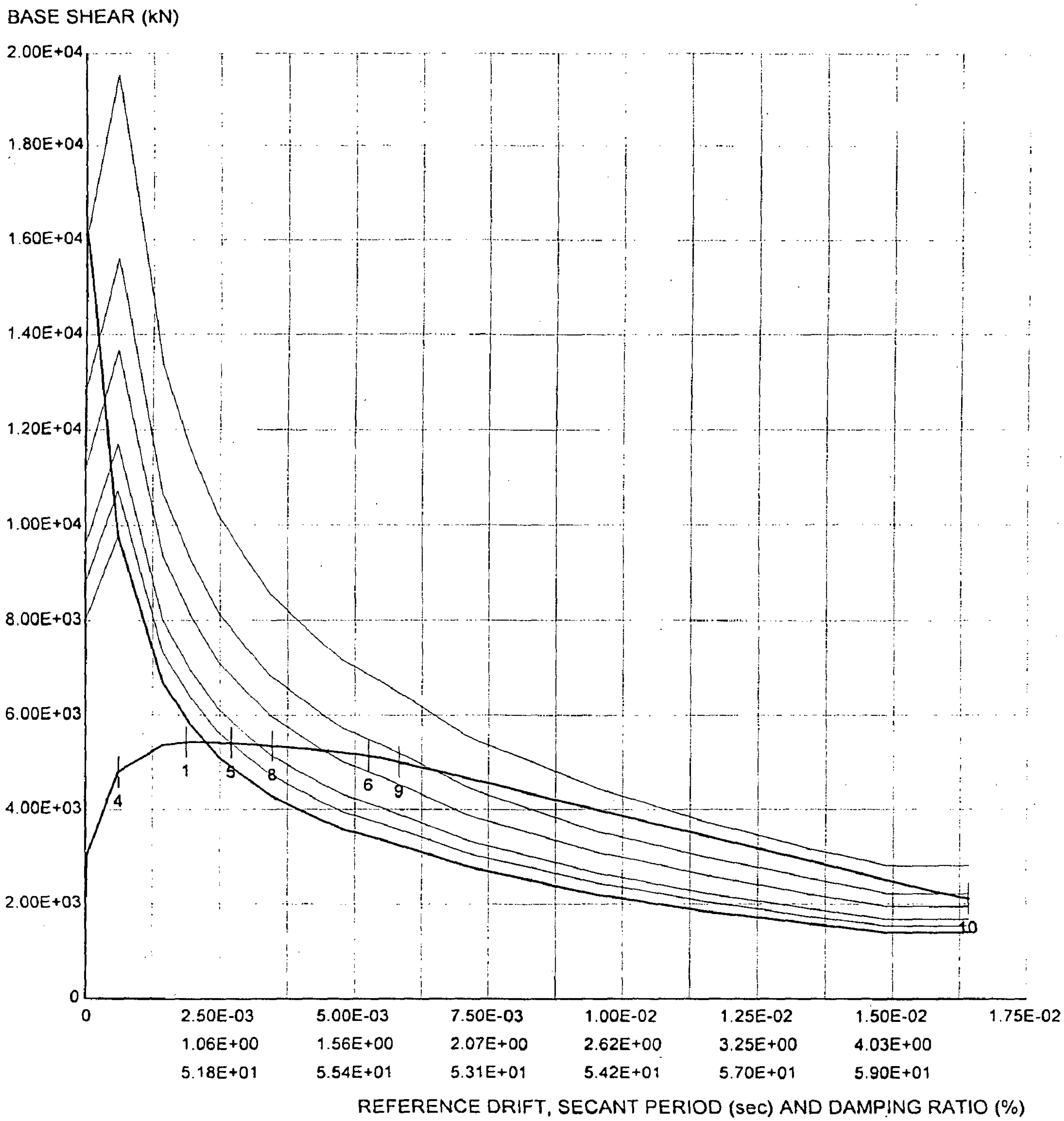
C-1 DBE


BASE SHEAR (kN)
1.00E+04
9.00E+03
8.00E+03
7.00E+03
6.00E+03
5.00E+03
4.00E+03
3.00E+03
2.00E+03
1.00E+03
0
1
5
8
6
9
4
10
0
2.50E-03
5.00E-03
7.50E-03
1.00E-02
1.25E-02
1.50E-02
1.75E-02
1.06E+00
1.56E+00
2.07E+00
2.62E+00
3.25E+00
4.03E+00
5.18E+01
5.54E+01
5.31E+01
5.42E+01
5.70E+01
5.90E+01
REFERENCE DRIFT, SECANT PERIOD (sec) AND DAMPING RATIO (%)

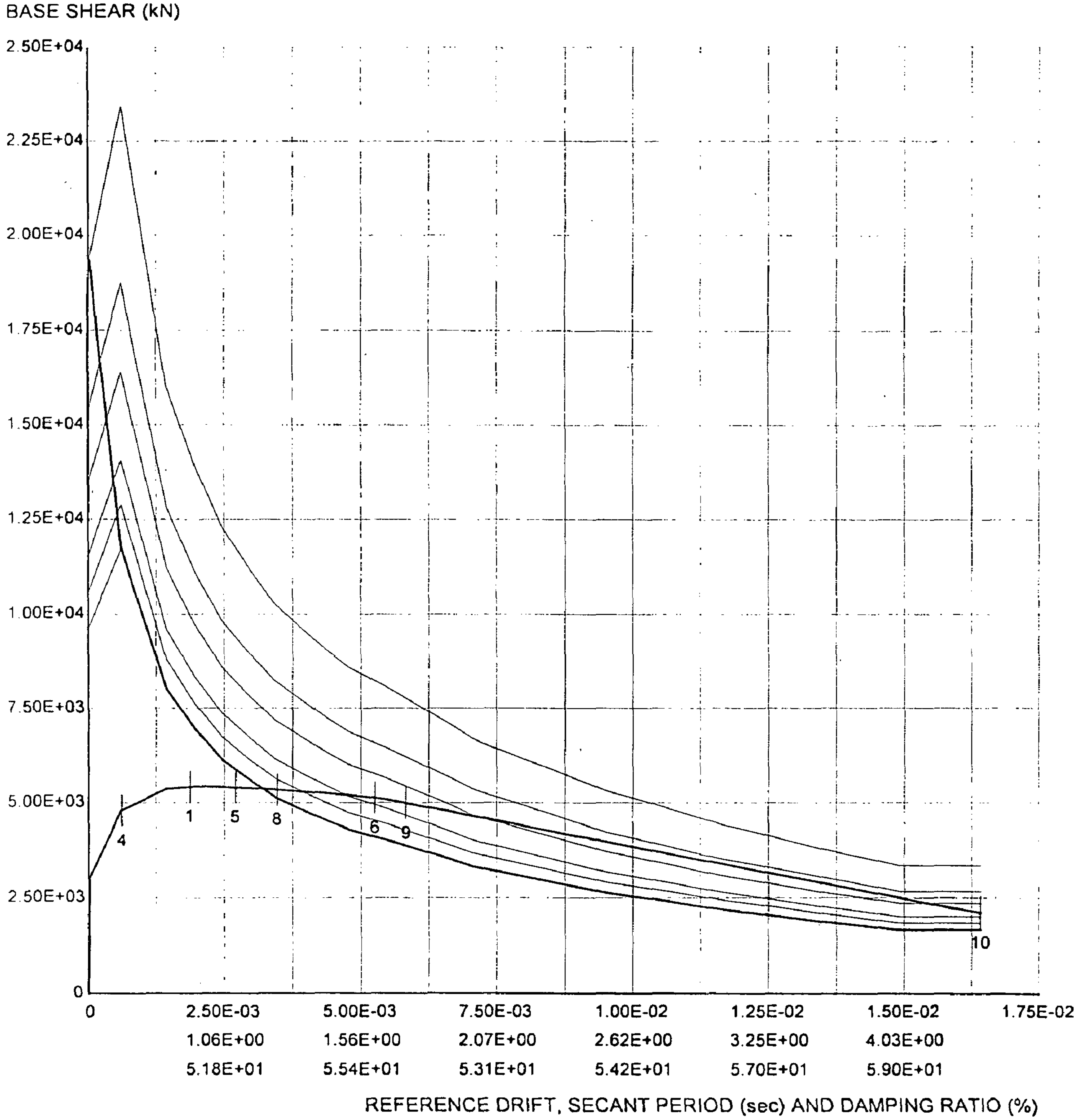
C-2 1.2 DBE


BASE SHEAR (kN)
1.25E+04
1 13E+04
1.00E+04
8.75E+03
7.50E+03
6.25E+03
5.00E+03
3.75E+03
2.50E+03
1.25E+03
0
1
5
8
6
9
4
10
0
2.50E-03
5.00E-03
7.50E-03
1.00E-02
1.25E-02
1.50E-02
1.75E-02
1.06E+00
1.56E+00
2.07E+00
2.62E+00
3.25E+00
4.03E+00
5.18E+01
5.54E+01
5.31E+01
5.42E+01
5.70E+01
5.90E+01
REFERENCE DRIFT, SECANT PERIOD (sec) AND DAMPING RATIO (%)

C-3 MCE


BASE SHEAR (kN)
2.00E+04
1.80E+04
1.60E+04
1.40E+04
1.20E+04
1.00E+04
8.00E+03
6.00E+03
4.00E+03
2.00E+03
0
4
1
5
8
6
9
10
0
2.50E-03
5.00E-03
7.50E-03
1.00E-02
1.25E-02
1.50E-02
1.75E-02
1.06E+00
1.56E+00
2.07E+00
2.62E+00
3.25E+00
4.03E+00
5.18E+01
5.54E+01
5.31E+01
5.42E+01
5.70E+01
5.90E+01
REFERENCE DRIFT, SECANT PERIOD (sec) AND DAMPING RATIO (%)

C-4 1.2 MCE


BASE SHEAR (kN)
2.50E+04
2.25E+04
2.00E+04
1.75E+04
1.50E+04
1.25E+04
1.00E+04
7.50E+03
5.00E+03
2.50E+03
0
4
1
5
8
6
9
10
0
2.50E-03
5.00E-03
7.50E-03
1.00E-02
1.25E-02
1.50E-02
1.75E-02
1.06E+00
1.56E+00
2.07E+00
2.62E+00
3.25E+00
4.03E+00
5.18E+01
5.54E+01
5.31E+01
5.42E+01
5.70E+01
5.90E+01
REFERENCE DRIFT, SECANT PERIOD (sec) AND DAMPING RATIO (%)