" STUDY OF SEISMIC PERFORMANCE OF A SINGLE STOREY STEEL FRAME BUILDING "

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CERTIFICATE

This is to certify that the project entitled "**STUDY OF SEISMIC PERFORMANCE OF A SINGLE STOREY STEEL FRAME BUILDING"** is a record of the bonafide dissertation work carried out by me, Rakesh Kumar Sharma, student of Master of Engineering in Civil (Structures) Engineering from Delhi College of Engineering, Delhi, 2007-2010 towards the partial fulfillment of the requirements for the award of the degree of Master of Engineering in Civil (Structures) Engineering.

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RAKESH KUMAR SHARMA

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LIST OF SYMBOLS & ABBREVIATIONS

y^t = Total Displacement

y = Relative Displacement

y_g = Ground Displacement

y_t = Independent

y_0 = Dependent

$p_t(t)$ = Force Function

C = Classical Damping Matrix

ξ_n = Damping Ratio

F_I = Inertia Forces

F_D = Damping Forces

F_S = Static Forces

$\boldsymbol{\omega}_n$ = Angular Frequency

$P_{eff}(t)$ = Effective Forces

k = Stiffness

m = Mass

c = Damping

a_0, a_1 = Rayleigh Constant

T_n = Natural Time Period

T_r = Transform Matrix

$\overline{K}_{tt}$/Kttc = Condensed Stiffness Matrix

M = Diagonal Mass Matrix

C = Damping Matrix

K = Stiffness Matrix

SMi = Member Stiffness Matrix

Rt = Rotation Matrix of Member

Δt = Time Increment

ΔF_I = Incremental Inertia Forces

ΔF_D = Incremental Damping Forces

ΔF_S = Incremental Static Forces

S_{de} = Elastic Displacement Spectrum

S_{ae} = Elastic Acceleration Spectrum

T = Time Period

μ = Ductility Ratio

R_μ = Reduction Factor

y_m = Max. Displacement

y_y = Yielding Displacement

S_a = Inelastic Design Acceleration

S_d = Inelastic Design Displacement

D = Peak Deformation

A = Pseudo-Acceleration

V = Pseudo-Velocity

g = Gravity Acceleration

V_b = Base Shear

E = Modulus of Elastic

Ix = Moment of Inertia in x-Direction

Iy = Moment of Inertia in y-Direction

Iz = Moment of Inertia in z-Direction

Ax = Area of Cross Section

G = Shear Modulus of Elastic

L = Length

ABSTRACT

Structural performance in linear and nonlinear region has been studied here, in capacity and demand format for a single storey 3-D frame.

To perform linear analysis, Wilson θ method has been used and its application has been extended in nonlinear region too.

The scaled El Centro Earthquake of various peak accelerations has been used to apply the lateral loads in the structure.

This approach is similar to pushover analysis in which capacity of the structure is established by applying gradually increasing lateral loads.

The above analysis has been done by bilinear idealization.

Instead of applying gradually increasing static lateral load, scaled El-Centro 1940 EQ record of various scaled up/down peaks has been used to apply the lateral load at the centroid of the deck floor of the frame. In a way, we have done a push over analysis to establish the capacity of the structure. Capacity curve has been drawn in base shear and Top storey displacement format.

CHAPTER 1. INTRODUCTION

There are so many forms of calamities in nature. Severe calamity may also be caused by direction of the invisible hand that is called “nature”. So, if one is not careful, the civilization will be ruined in no time. As for example the cyclone of Orissa & Tsunami (Dec. 2005) had caused vast losses.

Earthquake is a shaking, trembling, or concussion of the surface of earth, due to subterranean causes, often accompanied by a rumbling noise. It is basically a ground shaking originating from part of the earth’s crust, generally along cracks or fractures known as “faults”. The wave of shock sometimes traverses, destroying cities and many thousand lives; called also earth din, earthquake and earth shock,

Earthquakes may result in lack of basic necessities, loss of life, general property damage, road and bridge damage, and collapse of buildings or destabilization of the base of buildings which may lead to collapse in future earthquakes. If a structure has not been designed and constructed to absorb this swaying ground motion then major structural damage or outright collapse may happen, with grave risk to human life.

To fight against this challenge of nature, the structural engineers are supposed to suggest various approaches to retrofit or restrengthen the existing structures and innovate new design techniques for new constructions.

Earthquakes have caused considerable loss of life and damage to property so far and are considered as the greatest natural hazard to combat. Improved building practices leading to better structures for resisting seismic forces are effective means for reducing these losses.

1.1 PHILOSOPHY OF EQ RESISTANT DESIGN OF BUILDINGS:

1. The Building must resist frequent minor Earthquake without any significant structure damage.

2. Structure must withstand moderate Earthquake without significant structure damage.

3. a. During severe earthquakes occurrence, which are usually infrequent in this subcontinent deformation are permitted to go beyond elastic limit & ductile failure is permitted.

   b. Safety of life is a serious concern than the safety of the structure itself.

   c. Priority is given to life safety and to minimize the major economic and civic losses.

4. For the availability of sufficient evacuation time, the structural members and connection should have adequate ductility of energy dissipation capacity before failure.

1.2 DUCTILITY ASSESSMENT:

It should be noted that the designer has different means to compute the system ductility of the structure. One way is the study of the experimental behavior of members and structures. The other way, is 'time-history linear and nonlinear dynamic analysis'.

The main objective of this paper is to present basic principles and procedures for the above-mentioned analysis of earthquake-resistant steel frames and to describe, in a comparative way, the behavior of these frames.

Now a day's performance based Design is gaining interest and here the analysis of capacity and demand of the structure takes the centre stage.

In this research effort, study of capacity & demand curve has been done for single storey 3D frame. And instead of taken any arbitrary lateral loads, El-Centro Earthquake has been scaled up and down and Dynamic analysis of structure has been done instead of static analysis as is adopted in Pushover analysis, using Wilson θ method in both the linear & nonlinear region.

CHAPTER 2. LITERATURE REVIEW

2.1 BACKGROUND:

"Over the past decades, earthquake resistance design of building structures has been largely based on a ductility design concept worldwide. The performance of the intended ductile structures during major earthquakes (eg. Northridge 1994, Kobe 1995, chi- chi 1999 etc.) didn't prove to be satisfactory & indeed far below expectations.

High uncertainty of the ductility design strategy is primarily attributed to:
(Ref.-29)

- The desired "strong column weak beam" mechanism may not form due to existence of walls.
- Shear failure of columns due to inappropriate geometrical proportion or short column effect.
- Construction difficulty in grouting, especially at beam-column joint, because of complexity of steel reinforcement in ductility design."

This chapter presents an overview on related topics that provide the necessary background for the purpose of this research. The literature review concentrates on a range of earthquake engineering topics and structural modelling aspects. For the understanding of seismic capacity, a review of literature is required in experimental testing, current design practice, theoretical strength evaluation and modelling techniques such as finite element modelling. The literature review begins with a coverage of general earthquake engineering topics, which serves to set the context of the research.

2.2 EARTHQUAKE RESISTANT DESIGN TECHNIQUES

The objective of design codes is to have structures that will behave elastically under earthquakes that can be expected to occur more than once in the life of the building. It is also expected that the structure would survive major earthquakes without collapse that might occur during the life of the building. To avoid collapse during a large earthquake, members must be ductile enough to absorb and dissipate energy by post-elastic deformations. Nevertheless, during a large earthquake the deflection of the structure should not be such as to endanger life or cause a loss of structural integrity. Ideally, the damage should be repairable. In some cases, the order of ductility involved during a severe earthquake may be associated with large permanent deformations and in those cases; the resulting damage could be beyond repair. Even in the most seismically active areas of the world, the occurrence of a design earthquake is a rare event. In areas of the world recognised as being prone to major earthquakes, the design engineer is faced with the dilemma of being required to design for an event, which has a small chance of occurring during the design life time of the building. If the designer adopts conservative performance criteria for the design of the building, the client will be faced with extra costs, which may be out of proportion to the risks involved. On the other hand, to ignore the possibility of a major earthquake could be construed as negligence in these circumstances. To overcome this problem, buildings designed to these prescriptive provisions would

(1) Not collapse under very rare earthquakes;

(2) Provide life safety for rare earthquakes;

(3) Suffer only limited repairable damage in moderate shaking; and

(4) Be undamaged in more frequent, minor earthquakes.

The design seismic forces acting on a structure as a result of ground shaking are usually determined by one of the following methods:

- Static analysis, using equivalent seismic forces obtained from response spectra for horizontal earthquake motions.
- Dynamic analysis, either modal response spectrum analysis or time history analysis with numerical integration using earthquake records.

2.2.1 STATIC ANALYSIS

Although earthquake forces are of dynamic nature, for majority of buildings, equivalent static analysis procedures can be used. These have been developed on the basis of considerable amount of research conducted on the structural behaviour of structures subjected to base movements. These methods generally determine the shear acting due to an earthquake as equivalent static base shear. It depends on the weight of the structure, the dynamic characteristics of the building as expressed in the form of natural period or natural frequency, the seismic risk zone, and the type of structure, the geology of the site and importance of the building.

2.2.1.1 **LINEAR STATIC PROCEDURE (LSP)** :-(REF.-17)

Under the Linear Static Procedure (LSP), design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using a linearly elastic, static analysis.. In the LSP, the building is modeled with linearly-elastic stiffness and equivalent viscous damping that approximate values expected for loading to near the yield point. Design earthquake demands for the LSP are represented by static lateral forces whose sum is equal to the pseudo lateral load. The magnitude of the pseudo lateral load has been selected with the intention that when it is applied to the linearly elastic model of the building it will result in design displacement amplitudes approximating maximum displacements that are expected during the design earthquake. If the building responds essentially elastically to the design earthquake, the calculated internal forces will be reasonable approximations of those expected during the design earthquake. If the building responds in-elastically to the design earthquake, as will commonly be the case, the internal forces that would develop yielding in the building, will be less than the internal forces calculated on an elastic basis.

2.2.1.2 **NONLINEAR STATIC PROCEDURE (NSP) :-** (REF.-17)

Under the Nonlinear Static Procedure (NSP), a model directly incorporating inelastic material response is displaced to a target displacement, and resulting internal deformations and forces are determined. The nonlinear load-deformation characteristics of individual components and elements of the building are modeled directly. The mathematical model of the building is subjected to monotonically increasing lateral forces or displacements until either a target displacement is exceeded or the building collapses. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. The target displacement may be calculated by any procedure that accounts for the effects of nonlinear response on displacement amplitude.

Because the mathematical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake. Results of the NSP are to be checked using the applicable acceptance criteria. Calculated displacements and internal forces are compared directly with allowable values.

2.2.2 DYNAMIC ANALYSIS

2.2.2.1 LINEAR DYNAMIC PROCEDURE (LDP) :- (REF.-17)

Under the Linear Dynamic Procedure (LDP), design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using a linearly elastic, dynamic analysis. The basis, modeling approaches, and acceptance criteria of the LDP are similar to those for the LSP. The main exception is that the response calculations are carried out using either modal spectral analysis or Time- History Analysis. Modal spectral analysis is carried out using linearly-elastic response spectra that are not modified to account for anticipated nonlinear response. As with the LSP, it is expected that the LDP will produce displacements that are approximately correct, but will produce internal forces that exceed those that would be obtained in a yielding building. Results of the LDP are to be checked using the applicable acceptance criteria. Calculated displacements are compared directly with allowable values. Calculated internal forces typically will exceed those that the building can sustain because of anticipated inelastic response of components and elements. These obtained design forces are evaluated through the acceptance criteria, which include modification factors and alternative analysis procedures to account for anticipated inelastic response demands and capacities.

2.2.2.2 NONLINEAR DYNAMIC PROCEDURE (NDP) :- (REF.-17)

Under the Nonlinear Dynamic Procedure (NDP), design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using an inelastic response history dynamic analysis. The basis, modeling approaches, and acceptance criteria of the NDP are similar to those for the NSP. The main exception is that the response calculations are carried out using Time-History Analysis. With the NDP, the design displacements are not established using a target displacement, but instead are determined directly through dynamic analysis using ground motion histories. Calculated response can be highly sensitive to characteristics of individual ground motions; therefore, it is recommended to carry out the analysis with more than one ground motion record. Because the numerical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake. Results of the NDP are to be checked using the applicable acceptance criteria. Calculated displacements and internal forces are compared directly with allowable values.

2.2.2.2.1 MODELING AND ANALYSIS ASSUMPTIONS :- (REF.-17)

- **General**

The NDP shall conform to the criteria of this section. The analysis shall be based on characterization of the seismic hazard in the form of ground motion records. The modeling and analysis considerations set forth shall apply to the NDP unless the alternative considerations presented below are applied. The NDP requires Time-History Analysis of a nonlinear mathematical model of the building, involving a time step- by-time-step evaluation of building response, using discretized recorded or synthetic earthquake records as base motion input.

- **Ground Motion Characterization**

The earthquake shaking shall be characterized by ground motion time histories meeting the requirements.

- **Time-History Method**

Time-History Analysis shall be performed using horizontal ground motion time histories prepared. Multidirectional excitation effects shall be accounted for by meeting the requirements. The requirements may be satisfied by analysis of a three-dimensional mathematical model using simultaneously imposed pairs of earthquake ground motion records along each of the horizontal axes of the building.

2.2.2.2.2 ACCEPTANCE CRITERIA FOR NONLINEAR PROCEDURES :- (REF.-17)

- **Deformation-Controlled Actions**

Primary and secondary components shall have expected deformation capacities not less than the maximum deformations. Expected deformation capacities shall be determined considering all coexisting forces and deformations.

- **Force-Controlled Actions**

Primary and secondary components shall have lower bound strengths not less than the maximum design actions. Lower-bound strength shall be determined considering all coexisting forces and deformations.

2.2.2.2.3 BASIC TERMS USED IN NONLINEAR DYNAMIC TIME HISTORY ANALYSIS :- (REF.-33)

- **CAPACITY –CURVE**

It is the plot of the lateral force V on a structure, against the lateral deflection d, of the roof of the structure. This is often referred to as the 'push over' curve. Performance point and location of hinges in various stages can be obtained from pushover curves as shown in the fig. The range AB is elastic range, B to IO is the range of immediate occupancy, IO to LS is the range of life safety and LS to CP is the range of collapse prevention.

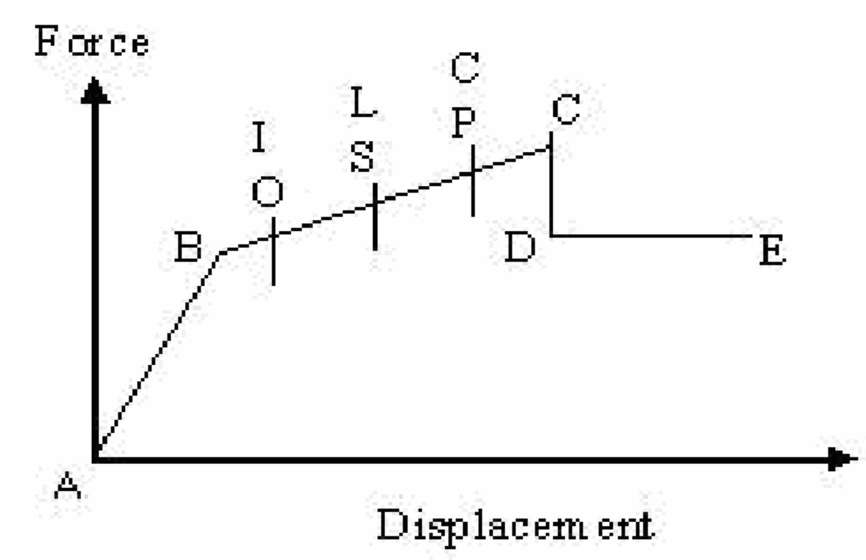


Fig: 2.1 Different stages of plastic hinge :- (Ref.-33)

- **CAPACITY-SPECTRUM**

It is the capacity curve transformed from shear force vs. roof displacement (V vs. d) coordinates into spectral acceleration vs. spectral displacement (S_a vs. S_d) coordinates.

- **DEMAND**

It is a representation of the earthquake ground motion or shaking that the building is subjected to. In nonlinear static analysis procedures, demand is represented by an estimation of the displacements or deformations that the structure is expected to undergo. This is in contrast to conventional, linear elastic analysis procedures in which demand is represented by prescribed lateral forces applied to the structure.

- **DEMAND –SPECTRUM**

It is the reduced response spectrum used to represent the earthquake ground motion in the capacity spectrum method.

- **PERFORMANCE POINT**

The intersection of the capacity spectrum with the appropriate demand spectrum in the capacity spectrum method (the displacement at the performance point is equivalent to the target displacement in the coefficient method). To have desired performance, every structure has to be designed for this level of forces. Desired performance with different damping ratios have been shown below:

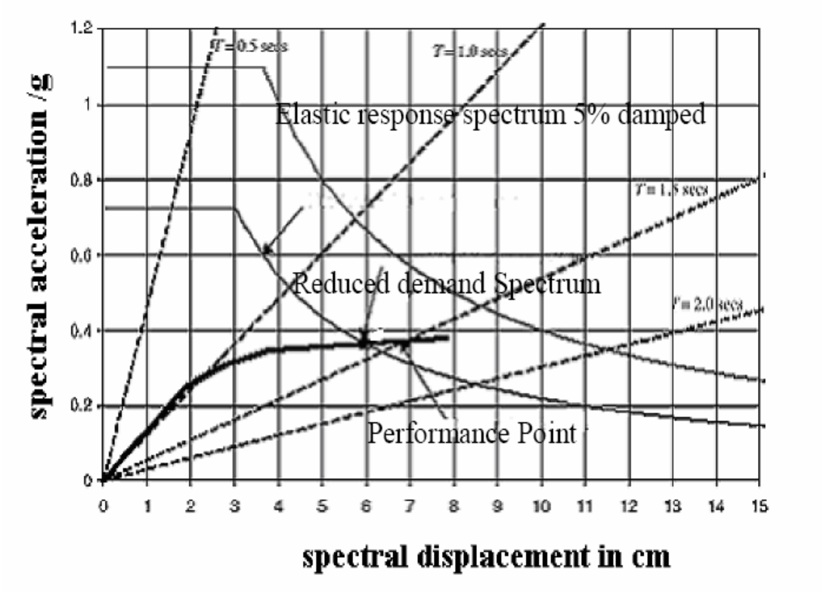


Fig: 2.2 Determination of performance point :-(Ref.-33)

- **YIELD (EFFECTIVE YIELD) POINT**

The point along the capacity spectrum where the ultimate capacity is reached and the initial linear elastic force-deformation relationship ends and effective stiffness begins to decrease.

- **COLLAPSE PREVENTION LEVEL**

This building performance level consists of the structural collapse prevention level with no consideration of nonstructural vulnerabilities, except that parapets and heavy appendages are rehabilitated.

- **INTERSECTION OF CAPACITY SPECTRUM AND DEMAND SPECTRUM**

When the displacement at the intersection of the demand and the capacity spectra, d_i is within 5% ($0.95\ d_{pi} \leq d_i \leq 1.05\ d_{pi}$) of the displacement of the trial performances point, a_{pi} , d_{pi} becomes the performance point. If the intersection of the demand spectrum and the capacity spectrum is not within the acceptable tolerance, then a new a_{pi} , d_{pi} is selected and the process is repeated. The performance point represents the maximum structural displacement expected for the demand earthquake ground motion.

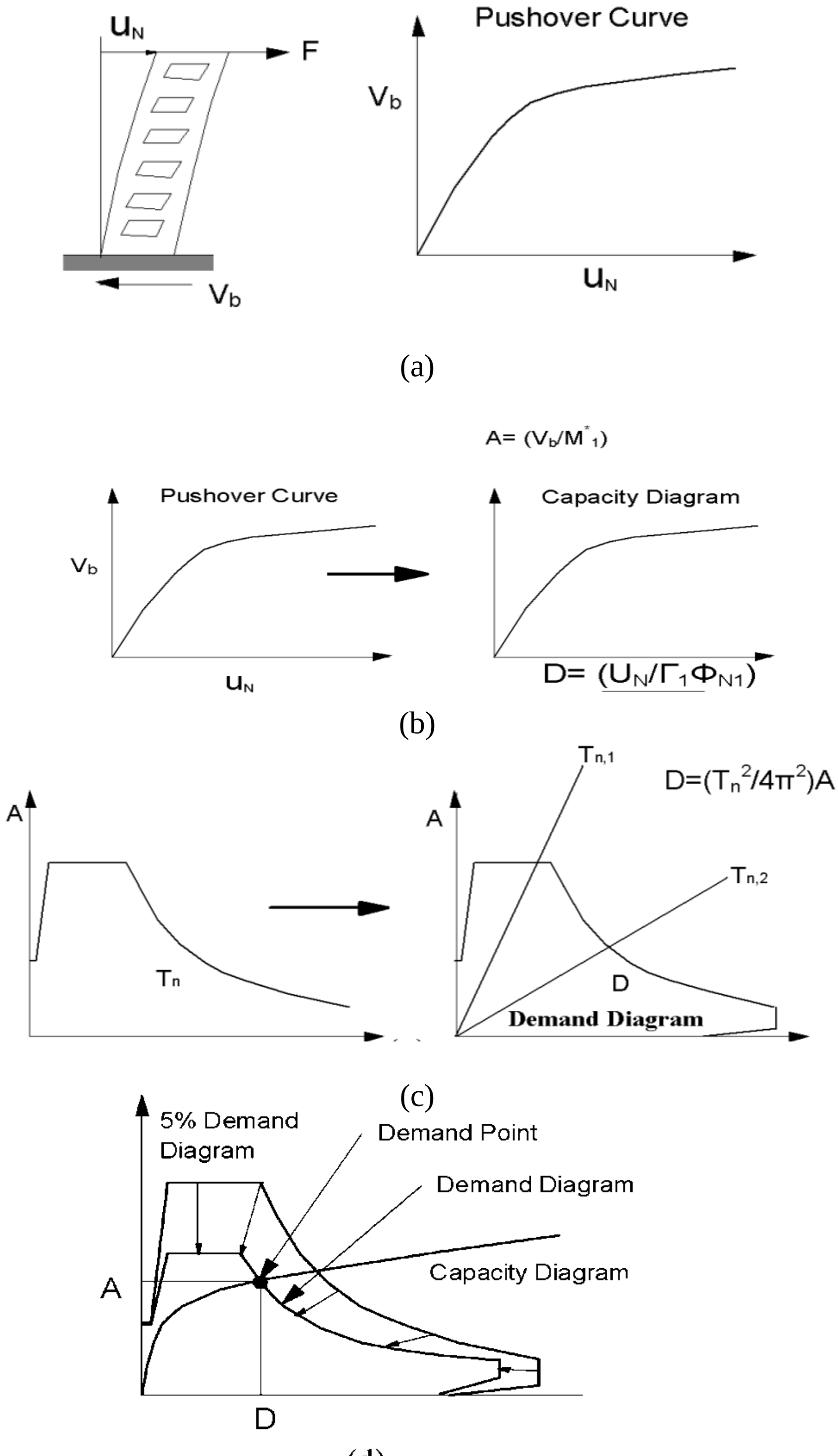


Fig: 2.3 Capacity spectrum method: (a) development of pushover curve, (b) conversion of pushover curve to capacity diagram, (c) conversion of elastic response spectrum from standard format to A-D format, and (d) determination of displacement demand

2.2.3 DISPLACEMENT-BASED SEISMIC DESIGN :-(REF.-26)

In recent years there have been extensive examinations of the current seismic design philosophy, which is based on provision of a required minimum strength, related to initial stiffness, seismic intensity and a force reduction or ductility factor considered to be a characteristic of a particular structural system and construction material. There are inappropriate two fundamental assumptions of the force-based design:

(1) That the initial stiffness of a structure determines its displacement response and

(2) That a ductility capacity can be assigned to a structural system regardless of its geometry, member strength, and foundation conditions.

The damage sustained by structures during seismic events is closely related to their displacements and deformation. For this reason, deformation-based design approaches have been developed to create a structure with controlled and predictable performance. This design process is consistent with the capacity design philosophy, as it requires control over deformation demand and supply of the energy dissipation zones. The direct displacement-based design have now matured to the stage where seismic assessment of existing structures, or design of new structures can be carried out to ensure that particular deformation-based criteria are met.

2.3 BEHAVIOUR OF REINFORCED CONCRETE STRUCTURES IN EARTHQUAKES

The overall behaviour of a structure when subjected to earthquake forces is affected by a number of factors.

2.3.1 FACTORS AFFECTING THE EARTHQUAKE PERFORMANCE OF REINFORCED CONCRETE STRUCTURES

The poor performance of buildings was generally due to a combination of inadequate strength and stiffness of the overall seismic resisting system and a poor distribution of strength and stiffness over successive storey's, leading to soft storey formation, a lack of provision of an adequate load path through the structure leading to partial or complete failure of the structure, and poor detailing of joints and connection leading to various types of non-ductile failures. (Ref.-27)

- **Ductility Capacity:** (Ref.-24)

The term ductility in structural design is used to mean the ability of a structure to undergo large inelastic deformations in the post-elastic range without a substantial reduction in strength. Ductility is an essential design requirement for a structure to behave satisfactorily under severe earthquake excitation. The ductility demand of a structure under seismic loading is dependent on the construction material, the design elastic strength and the structural system. The required ductility of a structure, element or section can be expressed in terms of the maximum imposed deformations. Often it is convenient to express the maximum deformation in terms of ductility factors, where the ductility factor is defined as the maximum deformation divided by the corresponding deformation present when yielding first occurs. The use of ductility factors permits the maximum deformations to be expressed in non-dimensional terms as indices of post-elastic deformation for design and analysis. Ductility factors have been commonly expressed in terms of the various parameters related to deformations, i.e. displacements, rotations, curvatures and strains.

- **P-Delta effect:**

P-delta effects reduce seismic performance because the effective lateral loads are increased as lateral displacements increase. This has the effect of further increasing the lateral displacement, and placing higher demand on the structural system. Damage will therefore occur sooner than in similar systems without significant P-delta effect. The importance of P-delta effects on the seismic performance of structures depends upon both the extent of vertical load being carried by the lateral resisting system and the stiffness of that system. If vertical loads are carried by columns, which are not part of the lateral load resisting system, then P-delta effects are not likely to be significant. Stiffer structural systems, such as shear walls, are less prone to P-delta effects because the lower lateral displacements control the additional over turning moments due to vertical loads. P-delta effects are significant for flexible systems, e.g. Moment-resisting frames, which carry both vertical and lateral loads to the foundation. They are most significant for fully ductile systems, because the relative values of vertical to lateral load are increased and the lateral load resisting system is more flexible than for structures with limited ductility.

- **Effects of strong beams and weak columns:**

Under earthquake and gravity loading, the critical bending moments develop in the vicinity of the frame joints. If these moments exceed the limit state capacity of the sections, plastic hinges will develop. These hinges may develop mainly in beams, columns or in a combination of locations. Beam hinge mechanism is more suitable for achieving ductility in concrete frames than column mechanism because:

1. A greater number of plastic hinges need to form before a collapse mechanism develops leading to smaller inelastic rotations in each hinge.
2. Columns are more critical because they carry the total gravity load from the structure above and damage to them could lead to catastrophic failures.
3. Beam hinges are more ductile because they carry lower axial loads than column hinge.

- **Effects of drift:**

In flexible buildings, there can be relatively large lateral movements between consecutive storeys, which is called the inter-storey drift. This can damage the structure and can also lead to unacceptable damage to the cladding and non-structural elements. This effect can be controlled with careful design and detailing. The control of the estimated lateral drift is another design aspect, which has a significant effect on the seismic performance of structures. Australian code (AS1170.4, 1993) requires that the maximum inter-storey drift be restricted to 1.5% of the storey height.

2.4 FINITE ELEMENT ANALYSIS :-(REF.-5)

The application of the finite element modelling (FEM) to RC structures has been underway for the last 20 years, during which time it has proven to be a very powerful tool in engineering analysis. The wide dissemination of computers and the development of the finite element method have provided means for analysis of much more complex systems in a much more realistic way. For any type of structure, the more complicated its structural geometric configuration is, the more a computer-based numerical solution becomes necessary. It has also been shown that experimental investigations are time consuming, capital intensive and even often impractical. The FEM is now firmly accepted as a very powerful general technique for the numerical solution of a variety of problems encountered in engineering. For concrete structures in particular, because of complexities of concrete behaviour in tension and compression together with integrity of concrete and steel, extreme difficulties are encountered in modelling and obtaining closed form solutions, even for very simple problems.

Finite element analysis has advantages over most other numerical analysis methods, including versatility and physical appeal. The major advantages of finite element analysis can be summarised as (Ref.-10)

1. Finite element analysis is applicable to any field problem.
2. There is no geometric restriction. The body analysed may have any shape.
3. Boundary conditions and loading are not restricted.
4. Material properties are not restricted to isotropy and may change from one element to another or even within an element.
5. Components that have different behaviours, and different mathematical descriptions, can be combined.
6. A finite element analysis closely resembles the actual body or region.
7. The approximation is easily improved by grading the mesh.

Some disadvantages of finite element analysis are:

1. It is fairly complicated, making it time-consuming and expensive to use.
2. It is possible to use finite element analysis programs while having little knowledge of the analysis method or the problem to which it is applied. Finite element analyses carried out without sufficient knowledge may lead to results that are worthless and some critics say that most finite element analysis results are worthless.(Ref.-10)

Specifically developed computer programs are used in finite element analyses of reinforced concrete structures. However, many commercially available general-purpose codes provide some kind of simplified material models intended to be employed in the analysis of concrete structures.

CHAPTER 3. PERFORMANCE LEVELS & CAPACITY CURVE OF THE STRUCTURE

3.1 PERFORMANCE OF STRUCTURE: - (REF.-6)

A performance objective specifies the desired seismic performance of the building. Seismic performance is described by designating the maximum allowable damage state (performance level) for an identified seismic hazard (earthquake ground motion). A performance objective may include consideration of damage states for several levels of ground motion and would then be termed a dual- or multiple-level performance objective. Once the building owner selects a performance objective, the engineer can identify the seismic demand to be used in the analysis and the acceptability criteria to be used for evaluation and design of the building's structural and nonstructural systems.

3.2 STRUCTURAL PERFORMANCE LEVELS AND RANGES: - (REF.-6)

A performance level describes a limiting damage condition which may be considered satisfactory for a given building and a given ground motion. The limiting condition is described by the physical damage within the building, the threat to life safety of the building's occupants created by the damage, and the post-earthquake serviceability of the building.

Structural performance levels are given names and number designations, while nonstructural performance levels are given names and letter designations. Building Performance Levels are a combination of a structural performance level and a nonstructural performance level and are designated by the applicable number and letter combination such as 1-A, 3-C, etc.

The structural performance levels:

- SP-1, Immediate occupancy,
- SP-2, Damage control,
- SP-3, Life safety,
- SP-4, Limited safety,
- SP-5, Structural safety,
- SP-6, Not considered,

Immediate Occupancy, Life Safety, and Structural Stability-are discrete damage states, and can, be used directly in evaluation and retrofit procedures to define technical criteria. The other structural performance designations-Damage Control, Limited Safety, and Not Considered-are important placeholders in the numbering scheme to allow direct reference to the wide variety of building performance levels that might be desirable to owners for evaluation or retrofit.

3.2.1 IMMEDIATE OCCUPANCY (SP-1):

The Post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical and lateral force resisting systems of the building retain nearly all of their Pre-earthquake characteristics and capacities.

The risk of life-threatening injury from structural failure is negligible, and the building should be safe for unlimited egress, ingress, and occupancy.

"We have defined SP-1 level failure, when stresses reach 250 N/mm2 and yielding starts in the member of structure and plastic hinge will form."Up to this level structure will in Elastic range and stiffness will be Constant.

3.2.2 DAMAGE CONTROL (SP-2):

This term is actually not a specific level but a range of post-earthquake damage states that could vary from SP-1, Immediate Occupancy to SP-3, Life Safety. It provides a placeholder for the many situations where it may be desirable to limit structural damage beyond the Life Safety level, but occupancy is not the issue. Examples of damage control include protection of significant architectural features of historic buildings or valuable contents.

Commentary: The Damage Control range, also sometimes called Limited Damage is defined to allow reference to performance levels between Immediate Occupancy and Life safety. Although not specifically defined in other current documents, the expected performance of most new buildings for the 10 percent/50-year event would probably fall in this range. A performance equivalent to that expected of new buildings is also sometimes called Repairable Damage, but economic or technological reparability is so undefined that the term is no more useful than Damage-Control. It is expected that many projects may have special demands for which criteria greater than Life Safety will be appropriate.

3.2.3 LIFE SAFETY (SP-3):

The post-earthquake damage state in which significant damage to the structure may have occurred but in which some margin against either total or partial structural collapse remains. The level of damage is lower than that for the Structural Stability level. Major structural components have not become dislodged and fallen, threatening life safety either within or outside the building. While injuries during the earthquake may occur, the risk of life-threatening injury from structural damage is very low. It should be expected that extensive structural repairs will likely be necessary prior to reoccupation of the building.

“We have defined SP-3 level as that level when deformation is 75% of the Displacement, when the structure will be at collapse prevention point.”

3.2.4 LIMITED SAFETY (SP-4):

This term is actually not a specific level but a range of post-earthquake damage states that are less than SP-3, Life Safety and better than SP-5, Structural Stability. It provides a placeholder for the situation where a 'retrofit may not meet all the structural requirements of the Life Safety level, but is better than the level of Structural Stability. These circumstances include cases when the complete Life Safety level is not-cost effective, or when only some critical structural deficiencies are mitigated (The nonstructural performance level used in this range varies and will depend on the intent of the damage control).

3.2.5 STRUCTURAL STABILITY (SP-5):

This level is the limiting post-earthquake structural damage state in which the building's structural system is on the verge of experiencing 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. However, all significant components of the gravity load resisting system continue to carry their gravity demands. Although the building retains its overall stability, significant risk of injury due to falling hazards may exist both within and outside the building and significant aftershocks may lead to collapse. It should be expected that significant major structural repair will be necessary prior to re-occupancy. In the older concrete building types considered in this document, it is very likely that the damage will not be technically or economically repairable. Falling hazards are not specifically prevented to achieve this performance level. Therefore NP-E (nonstructural performance not considered) is normally combined with SP-5.

“We have defined SP-5 level, when the structure is on the verge of collapse.”

3.2.6 NOT CONSIDERED (SP-6):

This is not a performance level but provides a placeholder for situations where only nonstructural seismic evaluation or retrofit is performed.

Commentary: Although unusual, nonstructural seismic improvements are sometimes made with no review of the structure. This might occur in locations of high and obvious vulnerability, such as at a computer room or for important equipment.

3.3 THE NONSTRUCTURAL PERFORMANCE LEVELS: (REF.-6)

The nonstructural performance levels - Operational, Immediate Occupancy, Life Safety and Hazards Reduced are discrete damage states and can be used directly in evaluation and retrofit procedures to define technical criteria. ·The other nonstructural performance designation – Not Considered - is an important placeholder to allow direct reference to the wide variety of building performance levels that might be desirable to for evaluation or retrofit.

- NP-A, Operational,
- NP-B, Immediate occupancy,
- NP-C, Life safety,
- NP-D, Reduced hazard,
- NP-E, Not considered,

3.3.1 OPERATIONAL (NP-A):

It is just a post-earthquake damage state, in which nonstructural elements and systems are generally in place and functional. Although minor disruption and cleanup should be expected, all equipment and machinery should be working. However, external utilities, which may not be available due to significant off-site damage, must be locally backed up. Contingency plans to deal with possible difficulties with external communication, transportation, and availability of supplies should be in place.

3.3.2 IMMEDIATE OCCPANCY (NP-B):

Immediate occupancy is a post-earthquake damage state, in which nonstructural elements and systems are generally in place and functional but Minor disruption and cleanup should be expected, particularly due to damage or shifting of contents. Although equipment and machinery are generally anchored or braced, their ability to function after strong shaking is not considered and some limitations on use or functionality may exist. All external utilities may not be locally backed up. Seismic safety status should not be affected.

3.3.3 LIFE SAFETY (NP-C):

This post-earthquake damage state could include considerable damage to nonstructural components and systems but should not include collapse or falling of items heavy enough to cause severe injuries either within or outside the building. Secondary hazards from breaks in high-pressure, toxic, or fire suppression piping should not be present. Nonstructural systems, equipment, and machinery may not be functional without replacement or repair, while injuries during the earthquake may occur; the risk of life-threatening injury from nonstructural damage is very low.

3.3.4 REDUCED HAZARD (NP-D):

This post-earthquake damage state could include extensive damage to nonstructural components and systems but should not include collapse or falling of large and heavy items that could cause significant injury to groups of people, such as parapets, masonry exterior walls, cladding, or large, heavy ceilings. While isolated serious injury could occur, risk of failures that could put (large numbers of people at risk within or outside the building is very low.

3.3.5 NOT CONSIDERED (NP-E):

Nonstructural elements, other than those -that have an effect on structural response, are not evaluated.

Commentary: This is not a performance level, but provides a designation for the common case where nonstructural elements are not surveyed or evaluated unless they have a direct effect on structural response, such as infill masonry walls or other heavy partitions. The designation is needed to accurately describe the Building Performance Level of Structural Stability for which nonstructural elements are, in fact, not considered.

3.4 ACCEPTANCE CRITERIA :- (REF.-20)

Three discrete levels structural performance levels are defined in the guidelines. These are respectively termed the immediate occupancy level, the life safety level and the collapse prevention level. The immediate occupancy level is a damage state in which the building has experienced only Very limited damage. The life safety level is a damage state in which the building has experienced significant damage to its structural components including yielding, cracking, spalling, and buckling and perhaps, limited fracturing.

The collapse prevention level is a state in which extreme damage, short of collapse has occurred to the structure. Building damages to this extent may experience large permanent lateral drifts as well as extensive localized failures of structural strength may also be significantly reduced. Such building would not be safe for post earthquake re-occupancy and may not be economically practical to repair and restore to service.

Both the collapse prevention and life safety performance levels have associated with them, a target margin against collapse. Margin may be thought of as the inherent factor of safety between the loadings that the structure is designed to resist and the loading that would produce failure. When buildings are subjected to strong earthquake ground shaking, they generally behave in an inelastic manner. Although the building code provisions for earthquake design have typically been based on providing minimum specified levels force-resisting capacity in a structure, force is not a particularly useful design parameter for predicting the margin of a structure that is responding within the inelastic range of behavior.

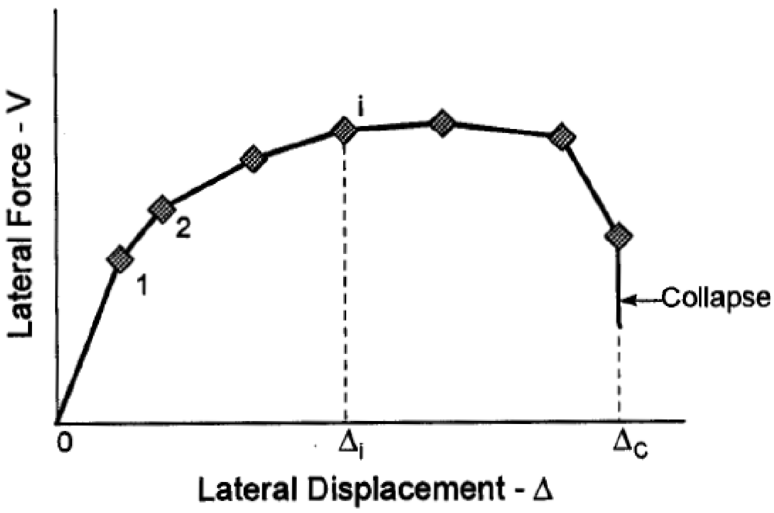


Fig: 3.1 Lateral force-displacement curves

Figure 3.1 depicts the force-deformation behavior of a simple structure with the ability to respond to ground shaking in a non-linear or inelastic manner. The vertical axis of this figure is the applied lateral force on the structure, V, and the horizontal axis is the resulting lateral displacement of the structure. At point “0” there is no lateral loading of the structure. The portion of the curve between points “0” and “1” represents the range of elastic behavior, in which no damage has occurred and the structure retains all its strength and stiffness. As the structure is loaded beyond point “1” on the curve, damage events such as cracking, yielding, or buckling of elements, indicated by the symbol “♦” in the fig, start to occur. Each damage event results in a degradation of the structure’s lateral stiffness and may also result in a degradation of lateral strength. As can be seen from the figure, once several damage events have occurred, lateral force becomes a relatively insensitive parameter by which to judge the structure's performance, since a wide range of different damage states can occur at a relatively constant level of applied lateral force. Because of this, FEMA -273 selected lateral displacement as the basic parameter by which structural performance is judged or controlled. In the figure ,at any point "i" on the curve ,the margin against failure can be defined as the ratio of the displacement at which collapse is likely to occur, Δc, to the displacement at point "i" ,Δi.

Figure 3.2 presents a curve similar to that of figure 3.1, except that the FEMA-273 performance levels have been superimposed on the curve .As can be seen, the Collapse Prevention performance level is defined to have a margin of 1.0 against collapse .That is, the maximum lateral displacement induced in a structure meeting the Collapse Prevention performance level should not exceed the lateral displacement at which the collapse is likely to occur. The Life safety performance level is defined to have a margin of 1.33.This implies that the maximum lateral displacement induced into a structure meeting the Life Safety level is 1/1.33 or ¾ of the lateral displacement at which collapse is likely to initiate .The Immediate Occupancy level does not have a specific margin associated with it, but instead, is achieved by maintaining damage to individual components of the structure to very low levels.

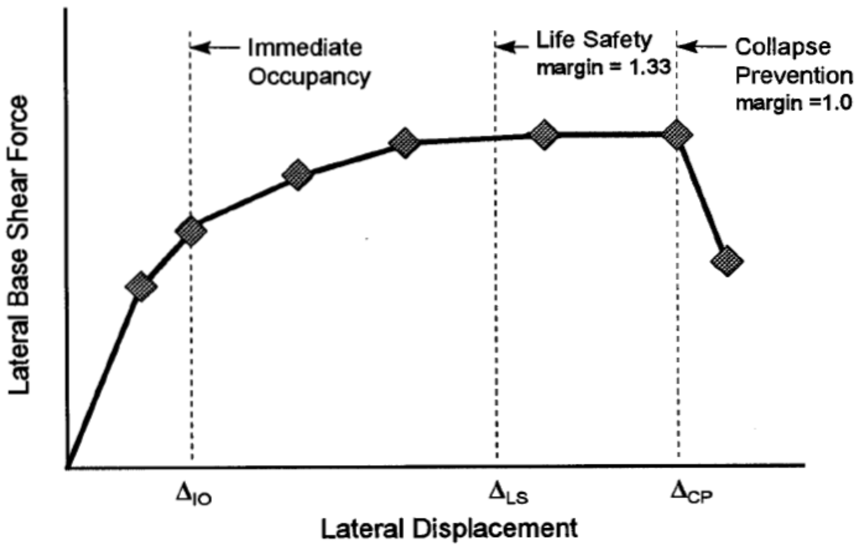


Fig: 3.2 Performance levels and margin

The acceptability of structural performance is evaluated on a structural component basis. In the guidelines, structural components are classified as being force controlled or deformation controlled, Deformation controlled components are those that significant ductility while force controlled component do not. In order to develop acceptance criteria for the document, an extensive literature search was conducted to determine typical hysteretic curves for the various types of structural components common in our existing building, based on past research. Where research data could be found for an element type, it was represented by the “back bone” curve. The back bone curve is an envelope of the hysteretic behavior of the component, taking into account the degradation that occurs under repeated cycles of motion to the same displacement level.

The acceptance criteria (permissible forces and deformation) for the various structural components and performance levels were determined from these idolized backbone curves. As stated earlier, for collapse prevention performance, the intent is that there is a margin of 1.0 against failure. Therefore, for deformation controlled components, the acceptance criterion is that the predicted member deformation not exceeds point “2”on the corresponding back bone curve. For force controlled components, the computed force on the component cannot exceed a lower bound estimate of the yield (or ultimate) capacity of the component. For life safety performance, the desired margin has a value of 1.33. Therefore the, acceptance criteria for life safety performance is that the member deformation for deformation controlled elements not exceed 75 % (1/1.33) times the deformation at point”2”of the corresponding backbone curves. For force controlled components, the total force on the component cannot exceed 75% of a lower bound estimate of the yield (or ultimate) strength of the component. These permissible values are tabulated in the guidelines, categorized by component type.

3.5 DAMAGE CONTROL AND BUILDING PERFORMANCE LEVELS (REF.-17)

Damage Control and Building Performance Levels				
	Building Performance Levels			
	Collapse Prevention Le	**Life Safety Level**	**Immediate Occupancy Level**	**Operational Level**
Overall Damage	Severe	Moderate	Light	Very Light
General	Little residual stiffness a strength, but load-bearin columns and walls funct Large permanent drifts. Some exits blocked. Infi and unbraced parapets failed or at incipient fail Building is near collapse	Some residual strength a stiffness left in all storie Gravity-load-bearing elements function. No o of-plane failure of walls tipping of parapets. Som permanent drift. Damag partitions. Building may beyond economical repa	No permanent drift. Structure substantially retains original strength stiffness. Minor crackin facades, partitions, and ceilings as well as struct elements. Elevators can restarted. Fire protectior operable.	No permanent drift; structure substantially retains original strengt and stiffness. Minor cracking of facades, partitions, and ceilings well as structural elements. All systems important to normal operation are function

Nonstructural componer	Extensive damage.	Falling hazards mitigate but many architectural, mechanical, and electric systems are damaged.	Equipment and contents generally secure, but ma not operate due to mechanical failure or la of utilities.	Negligible damage occurs. Power and oth utilities are available, possibly from standby sources.
Comparison with performance intended fo buildings designed, unde the NEHRP Provisions, the Design Earthquake	Significantly more dama and greater risk.	Somewhat more damage and slightly higher risk.	Much less damage and lower risk.	Much less damage and lower risk.

Table 2-4 Structural Performance Levels and Damage1—Vertical Elements

Elements	Type	Structural Performance Levels		
		Collapse Prevention S-5	**Life Safety S-3**	**Immediate Occupancy S**
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in sh columns.	Extensive damage to bea Spalling of cover and sh cracking (< 1/8" width) ductile columns. Minor spalling in nonductile columns. Joint cracks < wide.	Minor hairline cracking. Limited yielding possible few locations. No crushin (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 ducti elements. Limited cracki and/or splice failure in s nonductile columns. Sev damage in short columns	Minor spalling in a few places in ductile columns beams. Flexural cracking beams and columns. Shea cracking in joints < 1/16" width.
	Drift2	4% transient or permanent	2% transient; 1% perma	1% transient; negligible permanent

Structural Performance Levels and Damage1—Vertical Elements (continued)

Elements	Type	Structural Performance Levels		
		Collapse Prevention S-5	**Life Safety S-3**	**Immediate Occupancy 1**
Steel Moment Frame	Primary	Extensive distortion of bea and column panels. Many fractures at moment connections, but shear connections remain intact.	Hinges form. Local buckling of some beam elements. Severe joint distortion; isolated mom connection fractures, bu shear connections remai intact. A few elements r experience partial fractu	Minor local yielding at few places. No fracture Minor buckling or observable permanent distortion of members.
	Secondary	Same as primary.	Extensive distortion of beams and column pane Many fractures at mome connections, but shear connections remain inta	Same as primary.
	Drift2	5% transient or permanent	2.5% transient; 1% permanent	0.7% transient; negligil permanent

Braced Steel Frames	Primary	Extensive yielding and buckling of braces. Many braces and their connectio may fail.	Many braces yield or buckle but do not totally fail. Many connections fail.	Minor yielding or buckling of braces.
	Secondary	Same as primary.	Same as primary.	Same as primary.
	Drift2	2% transient or permanent	1.5% transient; 0.5% permanent	0.5% transient; negligi permanent
Concrete Walls	Primary	Major flexural and shear cracks and voids. Sliding a joints. Extensive crushing buckling of reinforcement. Failure around openings. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Some boundary element distress, including limit buckling of reinforceme Some sliding at joints. Damage around opening Some crushing and flex cracking. Coupling bear extensive shear and flexural cracks; some crushing, but concrete generally remains in pla	Minor hairline crackin walls, < 1/16" wide. Coupling beams experience cracking < 1/8" width.
	Secondary	Panels shattered and virtua disintegrated.	Major flexural and shea cracks. Sliding at joints. Extensive crushing. Fail around openings. Sever boundary element dama Coupling beams shatter and virtually disintegrat	Minor hairline crackin walls. Some evidence c sliding at construction joints. Coupling beams experience cracks < 1/ width. Minor spalling.
	Drift2	2% transient or permanent	1% transient; 0.5% permanent	0.5% transient; negligi permanent

3.6 FAILURE MECHANISMS OF FREE STANDING MASONRY WALL (REF.-21)

Consider the free standing masonry walls shown in Fig 3.3. In Fig 3.3(a), the ground motion is acting transverse to a free standing wall A. The force acting on the mass of the wall tends to overturn it. The seismic resistance of the wall is by virtue of its weight and tensile strength of mortar and it is obviously very small. This wall will collapse by overturning under the ground motion.

The free standing wall B fixed on the ground in Fig 3.3(b) is subjected to ground motion in its own plane. In this case, the wall will offer much greater resistance because of its large depth in the plane of bending. Such a wall is termed a shear wall. The damage modes of an unreinforced shear wall depend on the length-to-width ratio of the wall. A wall with small length to- depth ratio will generally develop a horizontal crack due to bending tension and then slide due to shearing. A wall with moderate length-to-width ratio and bounding frame diagonally cracks due to shearing as shown at Fig 3.3 (c).

A wall with large length-to-width ratio on the other hand, may develop diagonal tension cracks at both sides and horizontal cracks at the middle as shown at Fig 3.3 (d).

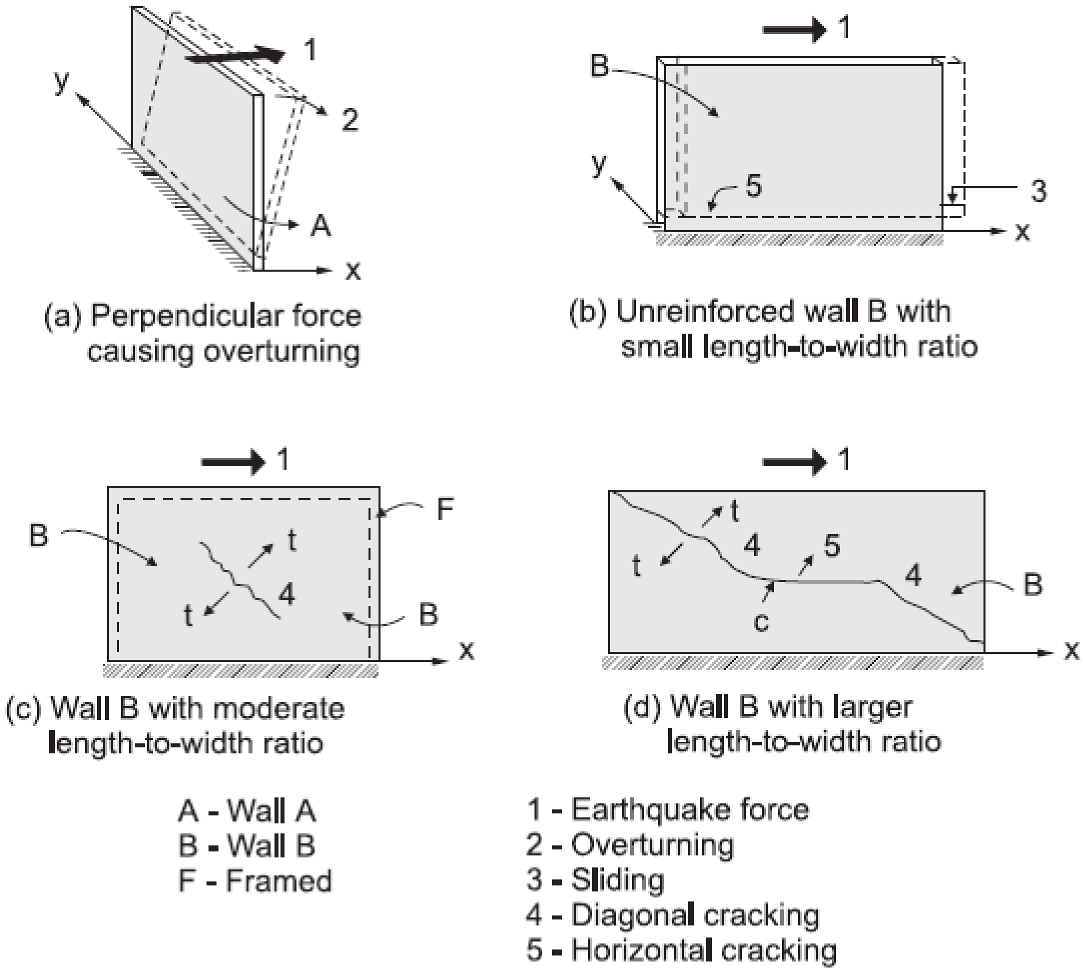


Fig: 3.3 Failure mechanisms of free standing walls

3.6.1 BRICK WALL/ INTERNAL PARTITION FAILURE CALCULATION

If we consider two perpendiculars panel of the storey and which one will fail first that will be considered as Brick wall failure.

In both the cases cantilever case will be critical and will fail first.

Width of wall (b) =3000mm

Thickness of wall (w) =230mm

Height of wall (h) =3000mm

Modulus of elasticity=6100 N/mm^2

Section Modulus of the wall (Z) = $b*d^2/6=3000*230*230/6=26450000 \ mm^3$
Permissible bending stress (f) = 0.5 N/mm^2 (Ref.-21)

Moment (M) =f*Z=0.5*26450000=13225000 N-mm

Shear (V) = M/h=13225000/3000=4408.33 N

Brick wall will fail when 4408.33 N Base shear force will be applied to the structure and correspondingly 1.4mm deflection will be there as shown in Capacity curve.

It is considered as SP-2 and because the Bending stress of masonry is very less as compare to steel, so SP-2 will come first.

CHAPTER 4. METHODLOGY ADOPTED TO OBTAIN CAPACITY & DEMAND CURVE

4.1 STATIC CONDENSATION: - (REF.-2)

The static condensation method is used to eliminate from dynamic analysis those DOF's of a structure to which zero mass is assigned; however, all the DFO's are included in the static analysis.

$$\begin{bmatrix} m_{tt} & 0 \\ 0 & 0 \end{bmatrix} \begin{Bmatrix} \ddot{y}_t \\ \ddot{y}_0 \end{Bmatrix} + \begin{bmatrix} k_{tt} & k_{t0} \\ k_{0t} & k_{00} \end{bmatrix} \begin{Bmatrix} y_t \\ y_0 \end{Bmatrix} = \begin{Bmatrix} p_t(t) \\ 0 \end{Bmatrix}$$

Where y_0 denotes the DOF's with zero mass and y_t DOF's with mass, also known as dynamic (Independent) DOF's.

By above equation

$$y_0 = -k^{-1}_{00}\, k_{0t}\, y_t$$

$$T_r = -k^{-1}_{00}\, k_{0t}$$

$$\bar{k}_{tt} = k_{tt} - k^{T}_{0t}\, k^{-1}_{00}\, k_{0t}$$

Where $\bar{k}_{tt}$ is the condensed stiffness matrix?

4.1.1 RAYLEIGH DAMPING: - (REF.-2)

Classical damping matrix consistent with the experimental data can be written as

$$C = a_0M + a_1K$$

The damping ratio for the nth mode of such a system is

$$\zeta_n = a_0/2 * 1/\omega_n + a_1/2 * \omega_n$$

The coefficients a_0 and a_1 can be determined from specified damping ratio ζ_1 and ζ_2 for the i^{th} and j^{th} modes, respectively. Expressing the above equation for these two modes in matrix form leads to

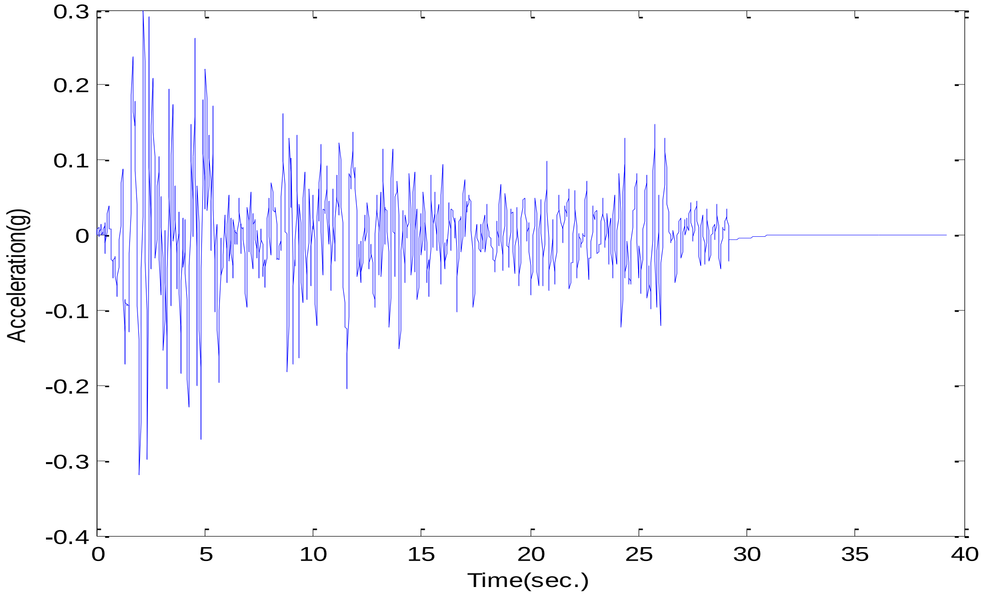
$$[1/\omega_i \quad \omega_i]\,[a_0] = 2[\zeta_i]$$

$$[1/\omega_j \quad \omega_j]\,[a_1] = 2[\zeta_j]$$

These two algebraic equations can be solved to determine the coefficients a_0 and a_1. If both modes are assumed to have the same damping ratio ζ_n which is reasonable based on experimental data then,

$$a_0 = 2\zeta * \omega_i * \omega_j / (\omega_i + \omega_j)$$

$$a_1 = 2\zeta / (\omega_i + \omega_j)$$



Peak Ground acceleration (PGA)-0.319g

Fig: 4.1 Time history plot of Elcentro earthquake

4.2 BASIC CONCEPT :- (REF.-2)

EQUATION OF MOTION:

$$y_j^t = y_j(t) + y_g(t)$$

In vector form

$$y_t = y(t) + y_g(t)1$$

Equation of dynamic equilibrium: - when no external force is applied

$$f_I + f_D + f_S = 0$$

Where

f_I = Inertia forces = $m\ddot{y}$

f_D = Damping forces = $c\dot{y}$

f_S = Static forces = ky

Only the relative motion u between the masses and the base due to structural deformations produce elastic and damping forces. But the inertia forces f_I are related to the total acceleration $\ddot{y}^t$ of the masses

$$f_I = m\ddot{y}^t$$

$$m\ddot{y} + c\dot{y} + ky = - m1\ddot{y}_g(t)$$

The equations of motion for a symmetric-plan multistory building subjected to earthquake ground acceleration $\ddot{y}_g^{(t)}$ are the same as those for external forces, known as the effective earthquake forces.

$$P_{eff}(t) = - m1\ddot{y}_g(t)$$

4.2.1 INELASTIC SYSTEM: - (REF.-2)

$$f_s = f_s(y, \dot{y})$$

$$m\ddot{y} + c\dot{y} + f_s(y, \dot{y}) = - m1\ddot{y}_g(t)$$

Where, $f_s(y, \dot{y})$ = resisting forces for an elastoplastic system.

For a given $\ddot{y}_g(t)$, y(t) depends on three system parameters: ω_n, ζ and y_y in addition to the form of the force- deformation relation; here the elastoplastic form has been selected. To demonstrate this fact

$$\ddot{y} + 2\zeta\omega_n\dot{y} + \omega_n^2 y_y \tilde{f}_s(y, \dot{y}) = -\ddot{y}_g(t)$$

Where

$$\omega_n = \sqrt{(k/m)} \quad \zeta = c/2m\omega_n \quad \tilde{f}_s(y, \dot{y}) = f_s(y, \dot{y})/f_y$$

It is clear that y (t) depends on ω_n, ζ and y_y .the quantity ω_n is the natural frequency ($T_n=2\pi/\omega_n$ is the natural period) of the inelastic system vibrating within its linearly elastic range (i.e. $y \leq y_y$) .It is also the natural frequency of the corresponding linear system .Similarly ζ is the damping ratio of the system based on the critical damping of the inelastic system vibrating within its linearly elastic range. The function $\tilde{f}_s(y, \dot{y})$ describe the force deformation relation in partially dimensionless form. This can be solved by nonlinear time history analysis (NLTHA).

4.3 MATHEMATICS BY WILSON THETA METHOD

4.3.1 TIME HISTORY ANALYSIS :-(REF.-4)

Time history analysis is used to determine the dynamic response of a structure to arbitrary loading. The dynamic equilibrium equations to be solved are given by:

$$M \ddot{y}(t) + C \dot{y}(t) + K \mathbf{y}(t) = r(t)$$

Where K is the stiffness matrix; C is the damping matrix; M is the diagonal mass matrix; y, $\dot{y}$, and $\ddot{y}$ are the displacements, velocities, and accelerations of the structure and the applied load. If the load includes ground acceleration, the displacements velocities and accelerations are relative to this ground motion.

Any number of time- history Load Cases can be defined. Each time-history case can differ in the load applied and in the type of analysis to be performed.

There are several options that determine the type of time-history analysis to be performed:

- <u>Linear vs. Non linear:</u>
- <u>Modal vs. Direct-integration</u>: These are two different solution methods each with advantages and disadvantages. Under ideal circumstances, both methods should yield the same results to a given problem.
- <u>Transient vs. Periodic</u>: Transient analysis considers the applied load as a One - time event, with a beginning and end. Periodic analysis considers the load to repeat in definitely, with all transient response damped out.

In a non linear analysis, the stiffness, damping, and load may all depend upon the displacements, velocities, and time. This requires an iterative solution to the equations of motion.

Types of nonlinearity:

- Material nonlinearity
- Geo metric nonlinearity

For nonlinear direct-integration time-history analysis, all of the available Nonlinearities may be considered.

4.3.2 NON LINEAR SINGLE DEGREE OF FREEDOM MODEL :-(REF.-4)

Consider a model for a single degree of freedom system and corresponding free body diagram. The dynamic equilibrium in the system is established by equating to zero the sum of inertial force $F_I(t)$, the damping force $F_D(t)$, the spring force $F_S(t)$, and the external force F(t). Hence at time t_i the equilibrium of these forces is expressed as

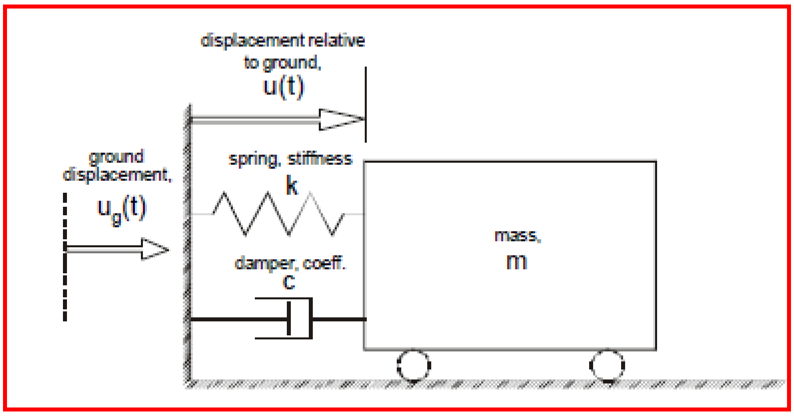


Fig: 4.2 Single degree of freedom system Modal

$$F_I\,(t_i) + F_D\,(t_i) + F_S\,(t_i) = F\,(t_i) \qquad (1)$$

And at short time Δt later as

$$F_I\,(t_i + t) + F_D\,(t_i + t) + F_S\,(t_i + t) = F\,(t_i + t) \qquad (2)$$

Subtracting (1) from (2) we get,

$$\Delta F_I + \Delta F_D + \Delta F_S = \Delta F_t \qquad (3)$$

Where the incremental forces in the equation are defined as follows:

$$\Delta F_I = F_I\,(t_i + t) - F_I\,(t_i)$$
$$\Delta F_D = F_D\,(t_i + t) - F_D\,(t_i)$$
$$\Delta F_S = F_S\,(t_i + t) - F_S\,(t_i)$$
$$\Delta F_t = F\,(t_i + t) - F\,(t_i) \qquad (4)$$

If we assume that the damping force is a function of the velocity and the spring force a function of displacement, and the inertial force remains proportional to the acceleration, we may then express the incremental forces as

$$\Delta F_I = m_i \Delta\ddot{y}$$
$$\Delta F_D = c_i \Delta\dot{y}$$
$$\Delta F_S = k_i \Delta y \quad (5)$$

Where the incremental displacement $\Delta y_{i,}$ incremental velocity $\Delta\dot{y}_i$ and the incremental acceleration $\Delta\ddot{y}_i$ are given by

$$\Delta y_i = y\,(t_i + t) - y\,(t_i) \quad (6)$$
$$\Delta\dot{y}_i = \dot{y}\,(t_i + t) - \dot{y}\,(t_i) \quad (7)$$
$$\Delta\ddot{y}_i = \ddot{y}\,(t_i + t) - \ddot{y}\,(t_i) \quad (8)$$

The coefficient k_i in equations (5) is defined as the current evaluation for the derivative of the spring force with respect to the displacement, namely,

$$k_{i\,=}\ (dF_s/\ dy)\ y_{=yi}$$

Similarly, the coefficient c_i is defined as the current value of the derivative of the damping force with respect to the velocity, that is,

$$c_i = (dF_D/d\dot{y})_{\,\dot{y}=\dot{y}i}$$

The substitution of equation in (3) results in a convenient form for the incremental equation namely,

$$m\ \Delta\ddot{y} + c\ \Delta\dot{y} + k\ \Delta y = \Delta F_t$$

If it is the current time i, the method of integration is known as explicit method. If the time (i+1) at the end of the time step is used, the method is known as implicit method.

4.3.3 INTEGRATION OF THE NON LINEAR EQUATION OF MOTION

In this method, the response is evaluated at successive increments Δt of time, usually taken of equal length of time for computational convenience. At the beginning of each interval, the condition of dynamic equilibrium is established. Then the response for a time increment Δt is evaluated approximately on the basis that the coefficients k(y) and c ($\dot{y}$) remain constant during the interval Δt. The nonlinear characteristics of these coefficients at the beginning of each time increment. The response is then obtained using the displacement and velocity calculated at the end of the each time interval as the initial conditions for the next time step.

As we said for each time interval, the stiffness coefficient k(y) and the damping coefficient c ($\dot{y}$) remain constant until the next step; thus the nonlinear behavior of the system is approximated by a sequence of successively changing linear systems.

4.4 MATHEMATICAL OR NUMERICAL STABILITY :- (REF.-2)

Numerical procedures that lead to bounded solutions if the time step is shorter than stability limit are called conditionally stable procedures. Procedures that lead to bounded solutions regardless of the time-step length are called unconditionally stable procedures.

Two of the most popular methods are the constant acceleration and the linear acceleration method. As the names of these methods imply, in the first method the acceleration is assumed to remain constant during the time interval Δt, while in the second method, the acceleration is assumed to vary linearly during the interval. As may be expected, the constant acceleration method is simpler, but less accurate when compared with the linear acceleration method for the same value of the time increment. The constant acceleration method is unconditionally stable, whereas the linear acceleration method is conditionally stable for $\Delta t \leq 0.551\ T_N$. For a given time step that does not approach this stability limit, the linear acceleration method is more accurate than the average acceleration method.

Numerical procedure involves two significant approximations:

1. The acceleration is assumed to vary linearly during the time interval Δt
2. The damping and stiffness properties of the system are evaluated at the initiation of each time increment and assumed to remain constant during the time interval.

In general, these two assumptions introduce errors which are small if the time step is short. However, these errors generally might tend to accumulate from step to step. This accumulation of errors should be avoided by imposing a total dynamic equilibrium condition at each step in the analysis. This is accomplished by expressing the acceleration at each step using the differential equation of motion in which the displacement and velocity as well as the stiffness and damping forces are evaluated at that time step.

There still remains the problem of the selection of the proper time increment Δt. As in any numerical method, the accuracy of the step by step integration method depends upon the magnitude of the increment selected. The following factors should be considered in the selection of Δt

1. The natural period of the structure
2. The rate of variation of the loading function and
3. The complexity of the stiffness and damping functions.

In general, it has been found that sufficiently accurate results can be obtained if the time interval is taken to be no longer than the one-tenth of the natural period of the structure. The second consideration is that the interval should be small enough to represent properly the variation of the load with respect to time. The third point that should be considered is any abrupt variation in the rate of change of the stiffness or damping function.

4.4.1 WILSON- Θ METHOD :-(REF.-4)

The basic assumption of the Wilson θ method is that the acceleration varies linearly over the time interval from t to $(t + \theta \Delta t)$ where $\theta \geq 1.0$. The value of the factor θ is determined to obtain optimum stability of the numerical process and accuracy of the solution. It has been shown by Wilson that, for $\theta \geq 1.38$, the method becomes unconditionally stable.

The equations expressing the incremental equilibrium conditions for a multi degree of freedom system can be derived as a matrix equivalent of the incremental equation of motion for the single degree of freedom system.

Thus taking the difference between dynamic equilibrium condition defined at time t_i and $(t_i + \tau)$, where $\tau = \theta \Delta t$, we obtain the incremental equation

$$M\, \delta\ddot{y}_i + C_{(\dot{y})}\, \delta\dot{y}_i + K_{(y)}\, \delta y_i = \delta F_i \tag{1}$$

In which the δ indicates that the increment are associated with the extended time $\tau = \theta \Delta t$. Thus

$$\delta y_i = y\,(t_i + \tau) - y\,(t_i) \tag{2}$$

$$\delta\dot{y}_i = \dot{y}\,(t_i + \tau) - \dot{y}\,(t_i) \tag{3}$$

$$\delta\ddot{y}_i = \ddot{y}\,(t_i + \tau) - \ddot{y}\,(t_i) \tag{4}$$

and

$$\delta F_i = F\,(t_i + \tau) - F\,(t_i) \tag{5}$$

In writing equation (1) we assume for the single degree of freedom systems, that the stiffness and damping are obtained for each time step as the initial values to the tangent to the corresponding curves rather than the slope of the secant line which require iteration. Hence the stiffness coefficient is defined as

$$K_{ij} = dF_{si} / dy_j \tag{6}$$

And the damping coefficient as

$$C_{ij}=dF_{Di} / dy_j \tag{7}$$

In which F_{si} and F_{Di} are respectively the elastic and damping forces at nodal coordinate i; y_i and $\dot{y}_i$ are respectively the displacement and velocity at nodal coordinate j.

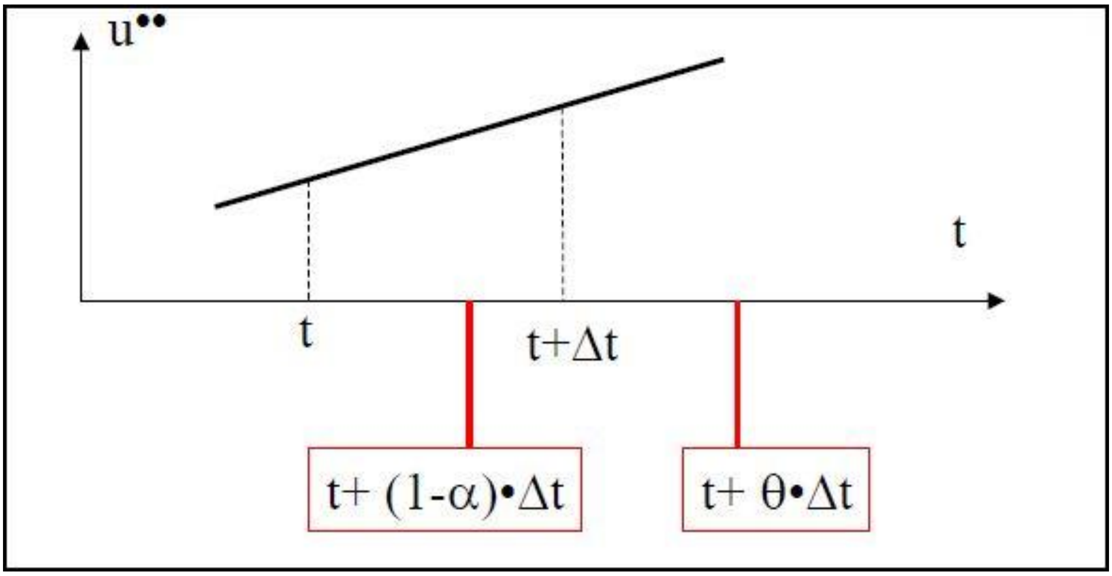


Fig: 4.3 Linear variation of acceleration over normal and extended time steps.

From the figure we can write the linear expression for the acceleration during the extended time step as

$$\ddot{y}\ (t) = \ddot{y}_i + (t - t_i)^* \ \delta\ddot{y}_i /\tau \tag{8}$$

In which $\delta\ddot{y}_i$ is given by equation (4)

Integrating equation (8) twice yields

$$\dot{y}\ (\ t) = \dot{y}_i \ + \ddot{y}_i\ (t - t_i\ ) + 0.5^*\ (\delta\ddot{y}^*(t - t_i\ )^2)/\tau \tag{9}$$

and

$$y(t) = y_i + \dot{y}_i (t - t_i) + 0.5\, \ddot{y}_i (t - t_i)^2 + \delta\ddot{y}_i (t - t_i)^3/(6\tau) \quad (10)$$

Evaluation of equations (9) and (10) at the end of the extended intervals $t= t_i +\tau$ gives

$$\delta\dot{y}_i = \ddot{y}_i \tau + \tau /2 * \delta\ddot{y}_i \quad (11)$$

and

$$\delta y_i = \dot{y}_i \tau + 0.5 * \ddot{y}_i * \tau^2 + \tau^2 /6 * \delta\ddot{y}_i \quad (12)$$

In which δy and $\delta\dot{y}_i$ are defined by equations (2) and (3), respectively. Now equation (12) is solved for the incremental acceleration $\delta\ddot{y}_i$ and substituted in equation (11). We obtain

$$\delta\ddot{y}_i = 6/ \tau^2 * \delta y_i - 6/ \tau * \dot{y}_i - 3 * \ddot{y}_I \quad (13)$$

and

$$\delta\dot{y}_i = 3/ \tau * \delta\dot{y}_i - 3 * \dot{y}_i - \tau/2 * \ddot{y}_i \quad (14)$$

Finally, substituting equations (13) and (14) into the incremental equation of motion equation (1) results in as equation of the incremental displacement δy_i which may be conveniently written as

$$\overline{K}\, \delta y_i = \overline{\delta F_i} \quad (15)$$

Where

$$\overline{K} = K + 6M/\tau^2 + 3C/ \tau \quad (16)$$

And

$$\overline{\delta F_i} = \delta F_i + (M * 6/ \tau + 3C) * \dot{y}_i + (3M + C * \tau/2) * \ddot{y}_i \quad (17)$$

where

$$\delta F_i = F_{i+1} + (F_{i+2} - F_{i+1}) (\theta-1) - F_i$$

`

Equation (15) has the same form as the static incremental equilibrium equation and may be solved for the incremental displacements δy_i by simply solving a system of linear equations.

To obtain the incremental accelerations $\delta\ddot{y}_i$ for the extended time interval, the value of δy_i obtained from the solution of the equation (15) is substituted into equation (13). The incremental acceleration $\Delta\ddot{y}_i$ for the normal time interval is the obtained by a simple linear interpolation. Hence,

$$\Delta\ddot{y} = \delta\ddot{y} / \theta \qquad (18)$$

To calculate the incremental velocity $\Delta\dot{y}_i$ and displacement Δy_i corresponding to normal interval Δt, uses made of eq. (11) and (12) with the extended time interval parameter τ substituted for Δt, that is

$$\Delta\dot{y}_i = \ddot{y}_i \Delta t + 0.5 * \Delta\ddot{y}_i \Delta t \qquad (19)$$

$$\Delta y_i = \dot{y}_i \Delta t + 0.5 * \ddot{y}_i \Delta t^2 + 1/6 * \Delta\ddot{y}_i \Delta t^2 \qquad (20)$$

Finally the displacement y_{i+1} and velocity $\dot{y}_{i+1}$ at the end of normal time interval are calculated by

$$y_{i+1} = y_i + \Delta y_i \qquad (21)$$

$$\dot{y}_{i+1} = \dot{y}_i + \Delta\dot{y}_i \qquad (22)$$

The initial acceleration for the next step should be calculated from the condition fo dynamic equilibrium at the time t + Δt.

$$\ddot{y}_{i+1} = M^{-1}(F_{i+1} - C_{i+1} \dot{y}_{i+1} - K_{i+1} y_{i+1}) \qquad (23)$$

In which the products represent $C_{i+1} \dot{y}_{i+1}$ and $K_{i+1} y_{i+1}$, respectively, the damping force and the stiffness force vectors evaluated at the end of time step. Once the displacement, velocity and acceleration vectors have been determined at time, the outline procedure is repeated to calculate these quantities at the next time step and the process is continued to any desired final time.

4.4.1.1 ALGORITHM FOR STEP BY STEP SOLUTION OF A LINEAR SYSTEM USING THE WILSON Θ INTEGRATION METHOD :-(REF-4)

1. Assemble the system stiffness matrix K, mass matrix M, and the damping matrix C.
2. Set the initial values for displacement y_0, velocity $\dot{y}_0$ and force F_0.
3. Calculate the initial acceleration from

$$M\ddot{y}_0 = F_0 - C\,\dot{y}_0 - K\,y_0$$

4. Select a time step Δt , the factor θ (usually taken as 1.4) , and calculate the constants τ , a_1, a_2 , a_3, and a_4 from the relations

$$\tau = \theta\,\Delta t,$$

$$a_1 = 3/\tau,$$

$$a_2 = 6/\tau,$$

$$a_3 = \tau/2,$$

$$a_4 = 6/\tau^2$$

Form the effective stiffness matrix $\bar{K}$, namely

$$\bar{K} = K + a_4 M + a_1 C$$

4.4.1.2 FOR EACH TIME STEP

1. Calculate by linear interpolation the incremental load δF_i for the time interval t_i to $t_i + \tau$

$$\delta F_i = F_{i+1} + (F_{i+2} - F_{i+1})\,(\theta - 1)\,F_i$$

2. Calculate the effective incremental load $\overline{\delta F_i}$ for the time interval t_i to $t_i + \tau$ from the relation

$$\overline{\delta F_i} = \delta F_i + (a_2 M + 3C) * \dot{y}_i + (3M + a_3\,C) * \ddot{y}_i$$

3. Solve for the incremental displacement from

$$\bar{K}\,\delta y_i = \overline{\delta F_i}$$

4. Calculate the incremental acceleration for the extended time interval τ, from

$$\delta\ddot{y}_i = a_4\,\delta y_i - a_2\,\dot{y}_i - 3\,\ddot{y}$$

5. Calculate the incremental acceleration for the normal time interval

$$\Delta\ddot{y} = \delta\ddot{y}/\theta$$

6. Calculate the incremental velocity $\Delta\dot{y}_i$ and incremental displacement Δy_i from t_i to $t_{i+\tau}+\Delta t$ from the relations

$$\Delta\dot{y}_i = \ddot{y}_i \Delta t + 0.5 *\Delta\ddot{y}_i *\Delta t$$

$$\Delta y_i = \dot{y}_i \Delta t + 0.5* \ddot{y}_i \Delta t^2 + 1/6*\Delta\ddot{y}_i \Delta t^2$$

7. Calculate velocity and displacement at $t_{i+1} = t_i$ to $(t_{i+\tau} + \Delta t)$

$$y_{i+1} = y_i + \Delta y_i$$

$$\dot{y}_{i+1} = \dot{y}_i + \Delta\dot{y}_i$$

8. calculate the acceleration at time directly from the equilibrium equation of motion, namely

$$M\ddot{y}_{i+1} = F_{i+1} - C\,\dot{y}_{i+1} - K_{i+1}$$

CHAPTER 5. DEMAND CURVE AND METHODLOGY ADOPTED TO OBTAIN DEMAND CURVE

5.1 SEISMIC DEMAND IN A-D FORMAT: - (REF-12)

Starting from the acceleration spectrum, we will determine the inelastic spectra in acceleration –displacement (AD) format.

For an elastic SDOF system, the following relationship is applies-

$$S_{de} = \frac{T^2}{4\pi^2} S_{ae}$$

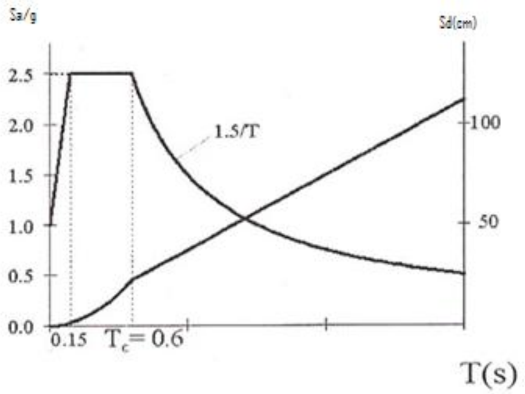


Fig: 5.1 Traditional Format

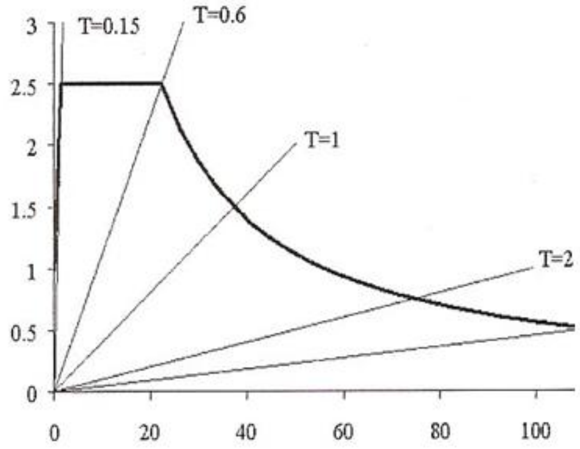


Fig: 5.2 Typical elastic acceleration S_{ae} and displacement spectrum S_{de} for 5% damping normalized to 1.0g peak ground acceleration (Peter Fajfar, M.EERI)

Where S_{ae} =elastic acceleration,

S_{de} =Elastic displacement spectrum, For an inelastic SDOF system with a bilinear force-deformation relationship, the acceleration spectrum (S_a) and the displacement spectrum (S_d) can be determined as (Vidic et al. 1994)

$$S_a = \frac{S_{ae}}{R_\mu}$$

$$S_d = \frac{\mu}{R_\mu} S_{de} = \frac{\mu}{R_\mu} \frac{T^2}{4\pi^2} S_{ae} = \mu \frac{T^2}{4\pi^2} S_a$$

Where μ is the ductility factor defined as the ratio between the maximum displacement and the yield displacement, and R_μ is reduction factor due to ductility, i.e., due to the hysteretic energy dissipation of ductile structure.

Several proposals have been made for the reduction factor R_μ. An excellent over view has been presented by Miranda and Bertero 1994. In this method, we will make use of bilinear spectrum for the reduction factor R_μ.

R_μ - μ - T Equations:

$$R_\mu = \begin{cases} (\mu - 1)\, T/T_c + 1 & T < T_c \\ \mu & T \geq T_c \end{cases}$$

Where Yield strength reduction factor = $R_\mu = f_0/f_y = y_0/y_y$

Ductility factor= $\mu = y_m / y_y$

T_c = Characteristic period of the ground motion. It is typically defined as the transition period where the constant acceleration segment of the response spectrum (the short-period range) passes to the constant velocity segment of the spectrum (the medium-period range). Equation 3 and 5 suggest that, in the medium –and long period range, the equal displacement rule applies, i.e., the displacement of the inelastic system is equal to the displacement of the corresponding elastic system with the same period. Equation 4 and 5 represent a simple version of the formulae proposed by Vidic et al 1994.

Starting from the elastic design spectrum the demand spectra (for the constant ductility factor (μ)) in A-D format can be obtained

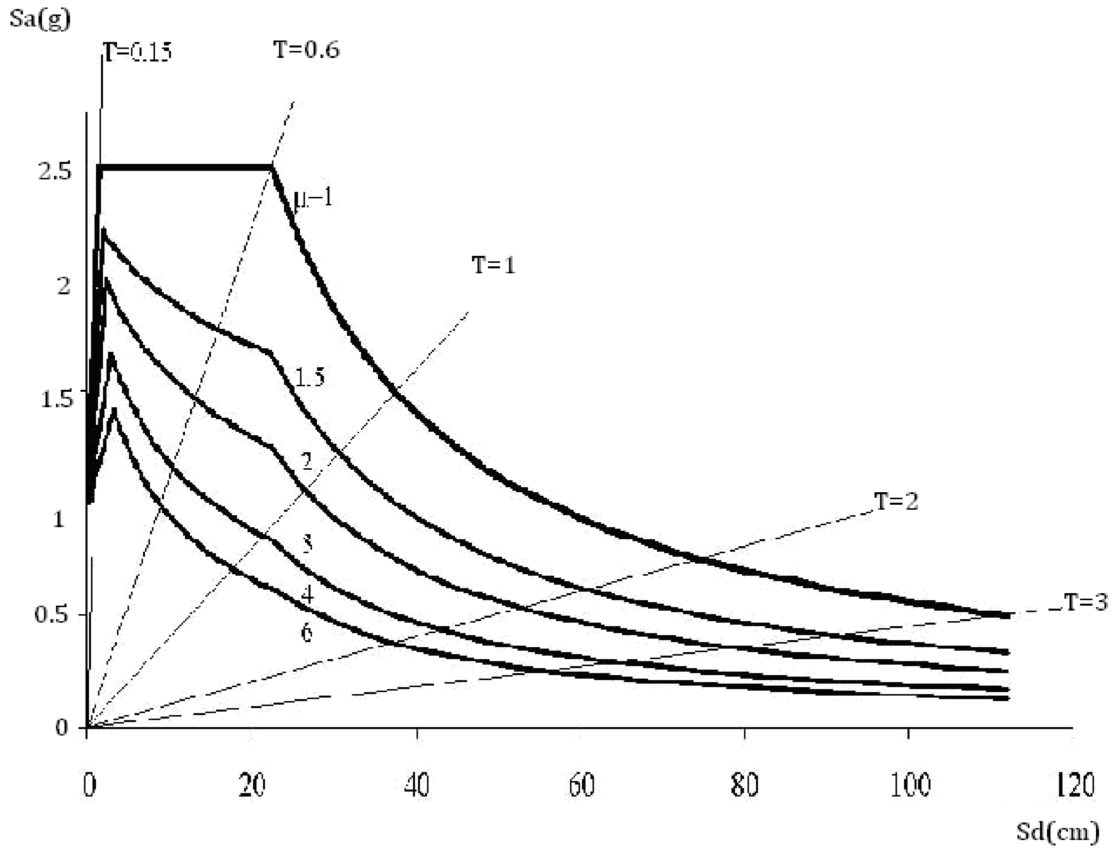
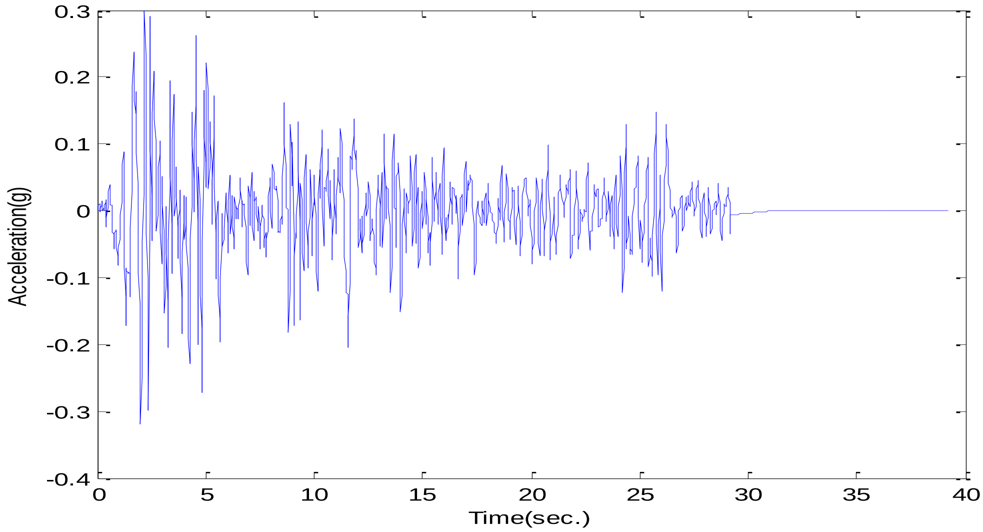


Fig: 5.3 Demand spectra for constant ductility in AD format normalized to1.0g peak ground acceleration (Peter Fajfar, M.EERI)

CHAPTER 6. PLOT OF DEMAND CURVE (BASED ON EL-CENTRO EARTHQUAKE)



Peak Ground acceleration (PGA)-0.319g

Fig: 6.1 Time history plot of Elcentro earthquake

6.1 RESPONSE SPECTRUMS :-(REF-2)

6.1.1 DEFORMATION RASPONSE SPECTRUM

The spectrum is developed for El-Centro ground motion. The time variation of the deformation induced by this ground motion in three SDF system is presented. For each system the peak value of deformation D=u_0 is determined from the deformation history. For a constant damping ratio a graph is plotted between natural time period and Peak deformation.

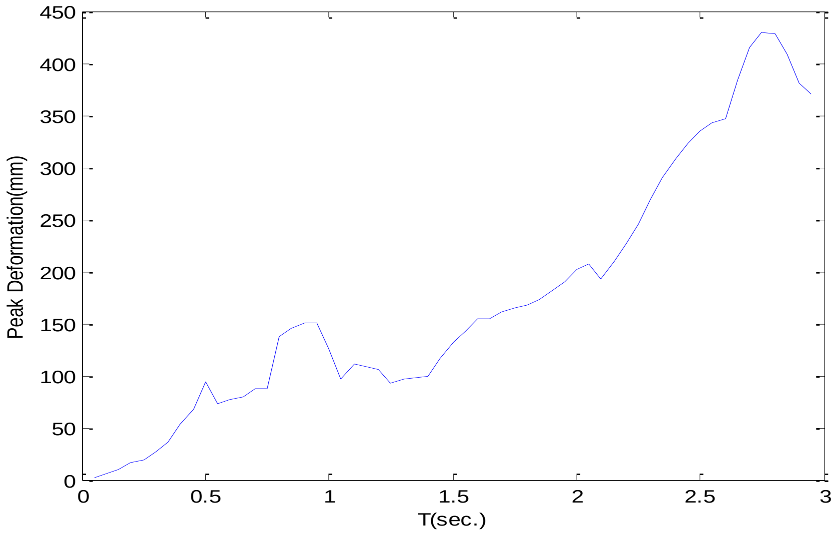


Fig: 6.2 Deformation response spectrum

6.1.2 PSEUDO-VELOCITY RESPONSE SPECTRUM: - (REF-2)

Consider a quantity V for an SDF system with natural frequency ω_n related to its peak deformation D=u0 due to earthquake ground motion

$$V = \omega_n D = \frac{2\pi}{T_n} D$$

The quantity V has units of velocity.

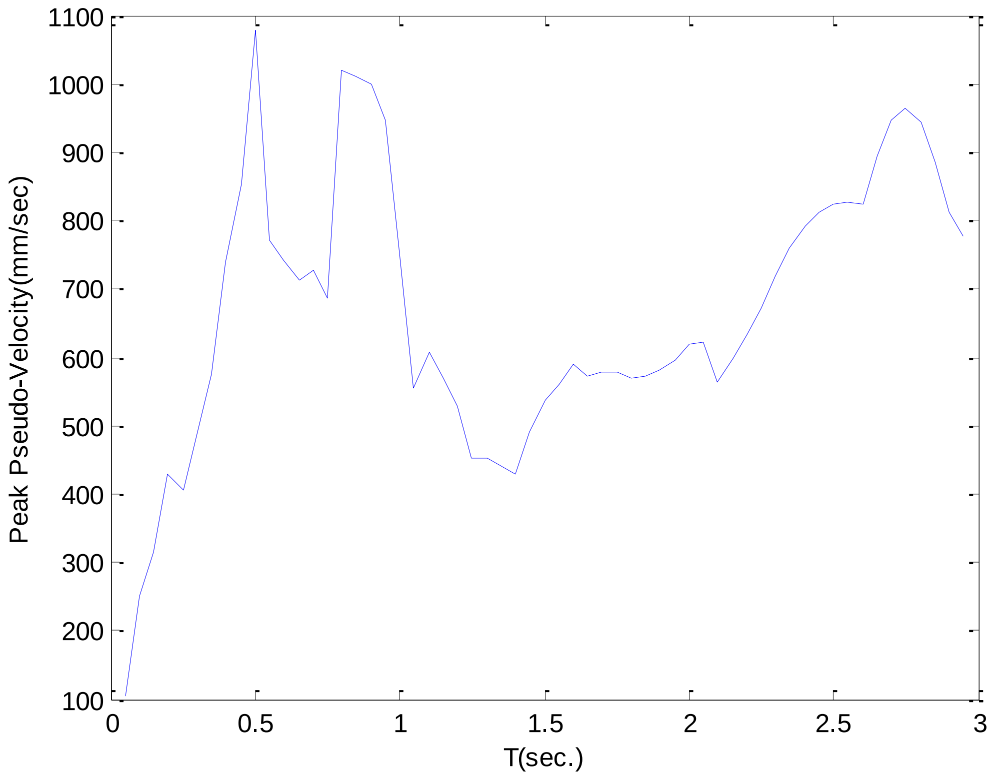


Fig: 6.3 Pseudo-velocity Response Spectrum

6.1.3 PSEUDO-ACCELERATION RESPONSE SPECTRUM: - (REF-2)

Consider a quantity A for an SDF system with natural frequency ω_n related to its peak deformation D=u_0 due to earthquake ground motion

$$A = \omega_n^2 D = (\frac{2\pi}{T_n})^2 D$$

The quantity A has units of acceleration and is related to the peak value of base shear Vbo (or the peak value of the equivalent static force f_{so})

$$V_{bo} = \frac{A}{g} \omega$$

Where w is the weight of the structure and g the gravitational acceleration when written in this form, A/g may be the base shear coefficient or lateral force coefficient.

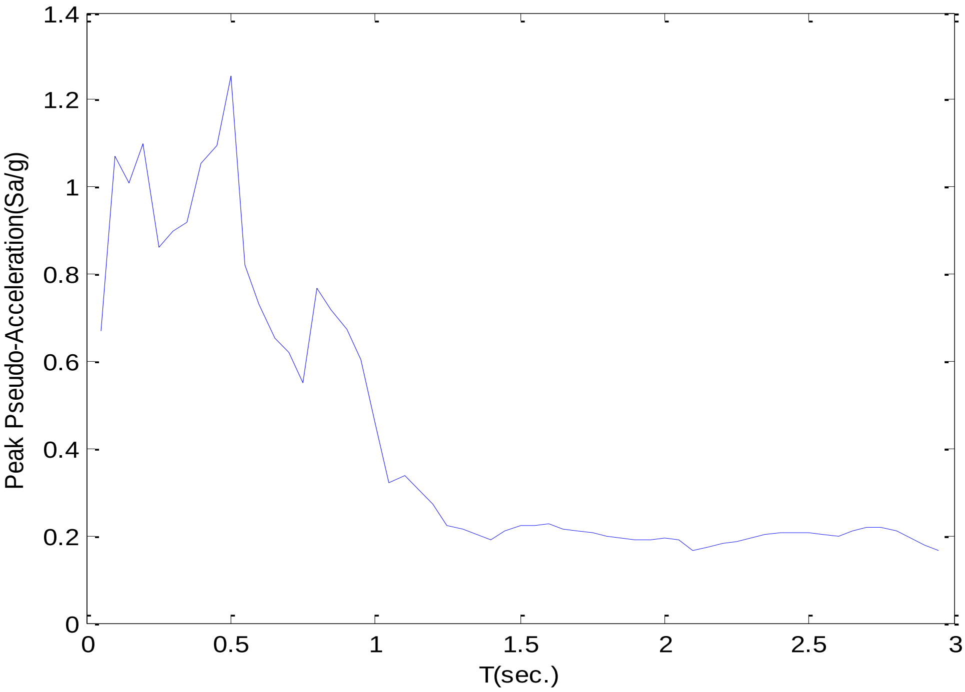


Fig: 6.4 Pseudo-Acceleration Response Spectrum

6.2 ELASTIC DESIGN SPECTRUM: - (REF-2)

In this section we introduce the concept of earthquake design spectrum for elastic systems and present a procedure to construct it from estimated peak values for ground acceleration, ground velocity and, ground displacement.

The design spectrum should satisfy certain requirement because it is intended for the design of new structure, or the seismic safety evaluation of existing structure, to resist future earthquake. For this purpose the response spectrum for a ground motion recorded during a past earthquake is inappropriate. The jaggedness in the response spectrum is characteristic of that one excitation. The response spectrum for another ground motion recorded at the same site during a different earthquake is also jagged, but the peaks and valleys are not necessarily at the same periods. Similarly, it is not possible to predict the jagged response spectrum in all its detail for a ground motion that may occur in the future. Thus the design spectrum should consist of a set of smooth curves or a series of straight lines with one curve for each level of damping.

The design spectrum should, in a general sense, be representative of ground motions recorded at the site during past earthquake. If none have been recorded at the site, the design spectrum should be based on ground motions recorded at other sites under similar conditions.

The design spectrum is based on statistical analysis of the response spectra for the ensemble of ground motions. Suppose that I is the number of ground motions in the ensemble, the i^{th} ground motion is denoted by $\ddot{u}_g^i(t)$, and $u_{go}^i, \dot{u}_{go}^i$ *and* $\ddot{u}_{go}^i$ are its peak displacement, velocity and acceleration, respectively. Each ground motion is normalized (scaled up and down) so that all the ground motions have the same peak ground acceleration, say $\ddot{u}_{go}$; other bases for normalization can be chosen. At each period T_n these are as many spectral values as the number I of ground motion records in the ensemble: $D^i, V^i, and\ A^i\ \ (i = 1,2, \dots \dots, I)$, the deformation, pseudo-velocity and pseudo-acceleration spectral ordinates. Such data were generated for an ensemble of 10 earthquake records, and selected aspects of the results are presented. The quantities $u_{go}, \dot{u}_{go},\ and\ \ddot{u}_{go}$ in the normalized scales are the average values of peak ground displacement, velocity and acceleration-averaged over the I ground motions. Statistical analysis of these data provides the probability distribution for the spectral ordinate, its mean value, and its standard deviation at each period T_n. The probability distribution are shown schematically at three selected T_n values, indicating that the coefficient of variation (= standard deviation /mean value) varies with T_n. Connecting all the mean values gives the mean response spectrum. Similarly connecting all the mean-plus-one-standard-deviation values gives the mean-plus-one-standard-deviation response spectrum. Observe that these two response spectra much smoother than the response spectrum for an individual ground motion. Such a smooth spectrum curve lends itself to idealization by a series of straight lines much better than the spectrum for an individual ground motion.

$$\frac{S_a}{g} = \begin{pmatrix} 1 + 21.6T & 0 \le T \le 0.125 \\ 3.7 & 0.125 \le T \le 0.6 \\ 2.22/T & 0.6 \le T \le 3 \end{pmatrix}$$

Where T_c = 0.6 sec.

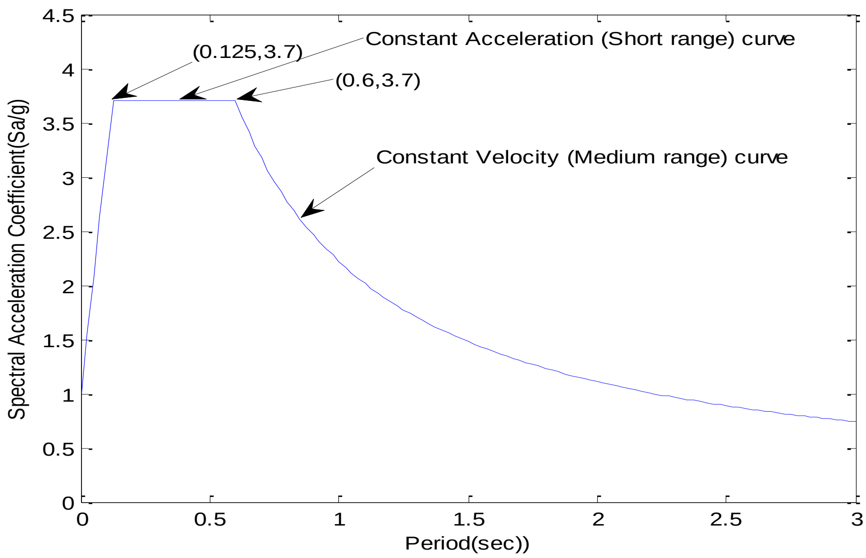


Fig: 6.5 Elastic pseudo-Acceleration Design Spectrum (84.1th percentile) for ground motion with $\ddot{u}_{go} = 1g$, $\dot{u}_{go}$ =48 in/sec., and u_{go}=36 in.; ξ=2%,(Ref.-2)

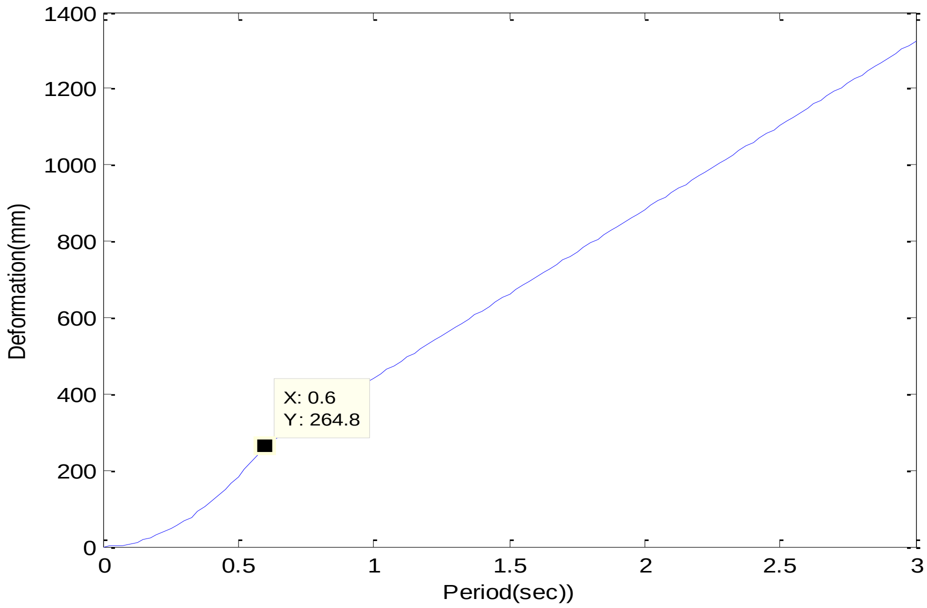


Fig: 6.6 Elastic Deformation Design Spectrum

$$S_a = \frac{S_{ae}}{R_\mu}$$

$$S_d = \frac{\mu}{R_\mu} * S_{de} = \frac{\mu}{R_\mu}\frac{T^2}{4\pi^2}S_{ae} = \mu\frac{T^2}{4\pi^2}S_a$$

Where μ is the ductility factor defined as the ratio between the maximum displacement and the yield displacement, and R_μ is reduction factor due to ductility, i.e., due to the hysteretic energy dissipation of ductile structure.

Several proposals have been made for the reduction factor R_μ. An excellent over view has been presented by Miranda and Bertero 1994. In this method, we will make use of bilinear spectrum for the reduction factor R_μ.

R_μ - μ - T Equations:

$$R_\mu = \begin{cases} (\mu - 1)\frac{T}{T_c} + 1 & T < T_c \\ \mu & T \geq T_c \end{cases}$$

Where Yield strength reduction factor = $R_\mu = f_0/f_y = y_0/y_y$

Ductility factor= $\mu = y_m / y_y$

6.2.1 IN-ELASTIC PSEUDO-ACCELERATION DESIGN SPECTRUM

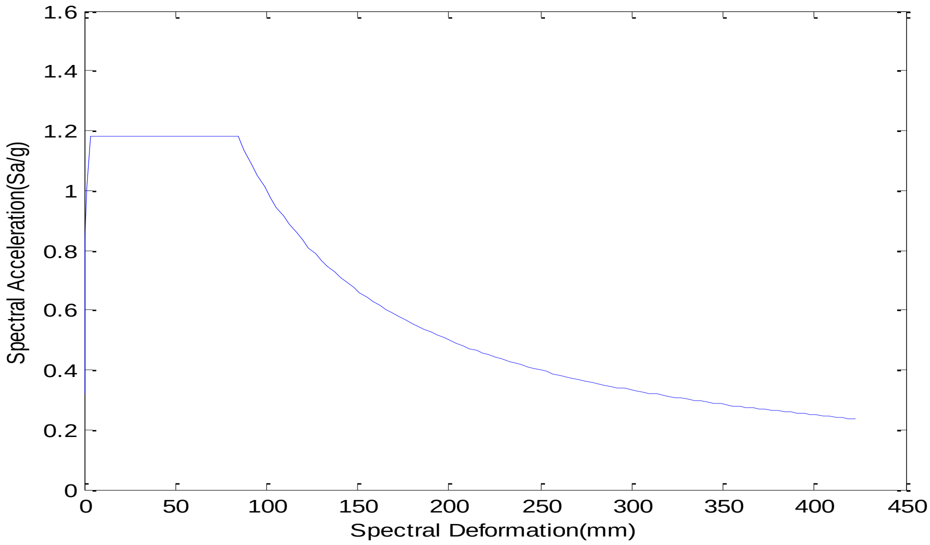


Fig: 6.7 In-Elastic pseudo-Acceleration Design Spectrum (84.1th percentile) for ground motion with $\ddot{u}_{go} = 0.319g$, ξ=2%, μ =1

6.2.2 IN-ELASTIC PSEUDO-ACCELERATION DESIGN SPECTRUM (FOR VARIOUS VALUE OF DUCTILITY RATIO)

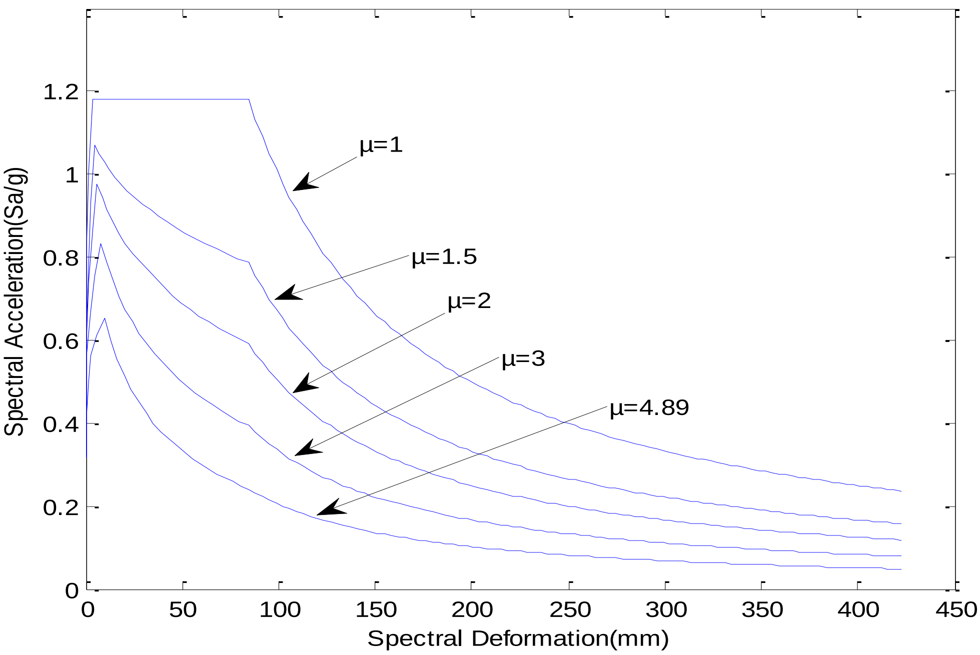


Fig: 6.8 In-Elastic pseudo-Acceleration Design Spectrum (84.1th percentile) for ground motion with $\ddot{u}_{go} = 0.319g$, ξ=2%, μ =1, 1.5, 2, 3,4.89

CHAPTER 7. PROBLEM STATEMENT

INPUT DATAS

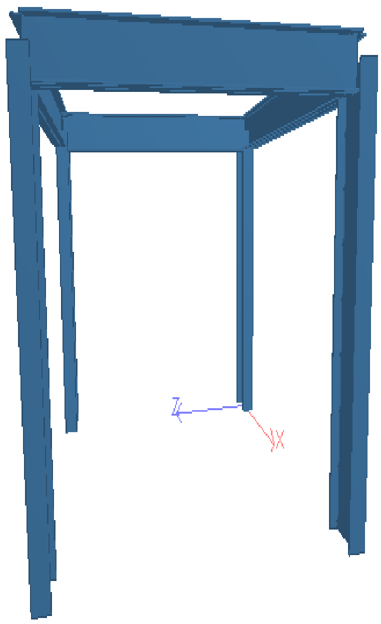


Fig: 7.1 Modeling of single story steel frame building

PROPERTIES OF THE BEAM SECTION:

ISMB 600@ 122.6 Kg/m

Material: Steel

Length (L) = 3000 mm.

Modulus of elasticity (E) $=2*10^5$ N/mm^2

Moment of Inertia (Ix) =1659000 mm^4

Moment of Inertia (Iy) =26510000 mm^4

Moment of Inertia (Iz) =918130000 mm^4

Area of cross section (Ax) =15621 mm^2

Shear modulus of elasticity (G) =80000 N/mm^2

Total mass of the frame (m) =2400 Kg.

PROPERTIES OF THE COLUMN SECTION:

ISHB 450@ 92.5 Kg/m

Material: Steel

Length (L) = 3000 mm.

Modulus of elasticity (E) $=2*10^5$ N/mm^2

Moment of Inertia (Ix) = $663000 mm^4$

Moment of Inertia (Iy) = $30450000\ mm^4$

Moment of Inertia (Iz) = $403499000\ mm^4$

Area of cross section (Ax) = $11789\ mm^2$

Shear modulus of elasticity (G) =80000 N/mm^2

ASSUMPTIONS

Capacity curve of a building is the plot between the base shear and the roof displacement. It approximates how the structure behaves after exceeding its elastic limit. The curve is obtained using linear analysis software with following assumptions:

1. The members of the building are assumed to yield simultaneously in groups. In reality the members will yield one by one and there will be change in stiffness at yield of every member.

2. The proper definition of the yield point in capacity curve has not been achieved and the yield point is assumed to be the point at which modulus of elasticity of most elements in the structure becomes zero.

3. The acceleration spectra as provided in the code are assumed to be constructed on the same principles as the elastic design spectra. As a result the codal design spectra are linearly scaled up and down for different magnitude of earthquakes.

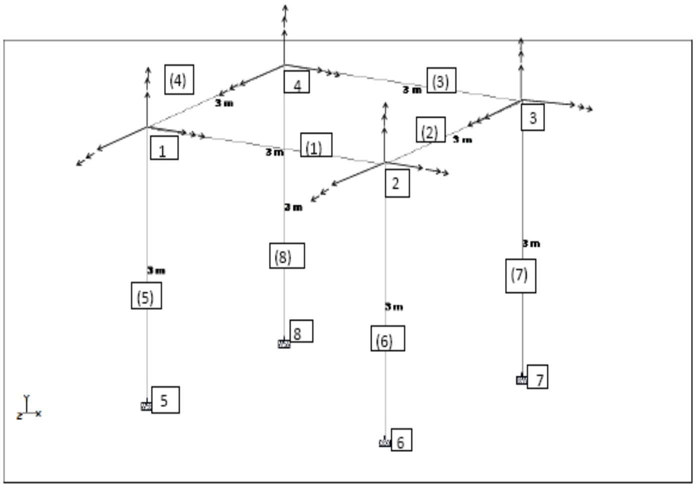
4. The stress strain behavior of the elements of the structure is assumed to be in conjunction with the stress strain curve of the steel element used in testing.

5. Idealization is done in the stress strain curve to remove the region of discontinuity.

6. The equations used for the conversion of the elastic demand spectra into inelastic demand spectra as proposed by "Peter Fajfar" are assumed to be valid in the present case.

7. The effect of vertical acceleration is not considered in the analysis.

EXAMPLE



Where Joints 1,2,3,4,5,6,7,8

Members (1),(2),(3),(4),(5),(6),(7),(8)

Steel Section ISMB 600@ 122.6 Kg/m. is taken FOR BEAMS

step.1 labelling for Beam

L=	3000	E=	200000	Ix=	1659000	Iy=	26510000	Iz=	918130000
EIx=	3.318E+11	EIy=	5.302E+12	EIz=	1.83626E+14	Ax=	15621	G=	80000
EAx=	3124200000	GIx=	1.3272E+11	m=	2400	Zz=	3060433.333	Zy=	252476.1905

step.2. Record basic member stiffness matrix,k for beam.

SMi=

EAx/L	0	0	0	0	0	(-EAx/L)	0	0	0	0	0
0	EIz*12/L^3	0	0	0	EIz*6/L^2	0	(-EIz*12/L^3)	0	0	0	EIz*6/L^2
0	0	EIy*12/L^3	0	(-EIy*6/L^2)	0	0	0	(-EIy*12/L^3)	0	(-EIy*6/L^2)	0
0	0	0	GIx/L	0	0	0	0	0	(-GIx/L)	0	0
0	0	(-EIy*6/L^2)	0	EIy*4/L	0	0	0	EIy*6/L^2	0	EIy*2/L	0
0	EIz*6/L^2	0	0	0	EIz*4/L	0	(-EIz*6/L^2)	0	0	0	EIz*2/L
(-EAx/L)	0	0	0	0	0	EAx/L	0	0	0	0	0
0	(-EIz*12/L^3)	0	0	0	(-EIz*6/L^2)	0	EIz*12/L^3	0	0	0	(-EIz*6/L^2)
0	0	(-EIy*12/L^3)	0	EIy*6/L^2	0	0	0	EIy*12/L^3	0	EIy*6/L^2	0
0	0	0	(-GIx/L)	0	0	0	0	0	GIx/L	0	0
0	0	(-EIy*6/L^2)	0	EIy*2/L	0	0	0	EIy*6/L^2	0	EIy*4/L	0
0	EIz*6/L^2	0	0	0	EIz*2/L	0	(-EIz*6/L^2)	0	0	0	EIz*4/L

Master Matrix for beam

Member Matrix for Member 1,2,3 and 4

SMi=

1041400	0	0	0	0	0	-1041400	0	0	0	0	0
0	81611.55556	0	0	0	122417333.3	0	-81611.55556	0	0	0	122417333.3
0	0	2356.444444	0	-3534666.67	0	0	0	-2356.444444	0	-3534666.67	0
0	0	0	44240000	0	0	0	0	0	-44240000	0	0
0	0	-3534666.667	0	7069333333	0	0	0	3534666.667	0	3534666667	0
0	122417333.3	0	0	0	2.44835E+11	0	-122417333.3	0	0	0	1.22417E+11
-1041400	0	0	0	0	0	1041400	0	0	0	0	0
0	-81611.5556	0	0	0	-122417333.3	0	81611.55556	0	0	0	-122417333.3
0	0	-2356.444444	0	3534666.667	0	0	0	2356.444444	0	3534666.667	0
0	0	0	-44240000	0	0	0	0	0	44240000	0	0
0	0	-3534666.667	0	3534666667	0	0	0	3534666.667	0	7069333333	0
0	122417333.3	0	0	0	1.22417E+11	0	-122417333.3	0	0	0	2.44835E+11

Steel Section ISHB 450@ 92.5 Kg/m. is taken COLUMNS

Record basic member stiffness matrix,k for column.

labelling for Column=			Ixc=	663000	Iyc=	30450000	Izc=	403499000	EIxc=	1.326E+11
			EIyc=	6.09E+12	EIzc=	8.06998E+13	Axc=	11789	EAxc=	2357800000
			GIxc=	53040000000	h=	3000	Zzc=	1793328.889	Zyc=	243600

SMi=

EAxc/h	0	0	0	0	0	(-EAxc/h)	0	0	0	0	0
0	EIzc*12/h^3	0	0	0	EIzc*6/h^2	0	(-EIzc*12/h^3)	0	0	0	EIzc*6/h^2
0	0	EIyc*12/h^3	0	(-EIyc*6/h^2)	0	0	0	(-EIyc*12/h^3)	0	(-EIyc*6/h^2)	0
0	0	0	GIxc/h	0	0	0	0	0	(-GIxc/h)	0	0
0	0	(-EIyc*6/h^2)	0	EIyc*4/h	0	0	0	EIyc*6/h^2	0	EIyc*2/h	0
0	EIzc*6/h^2	0	0	0	EIzc*4/h	0	(-EIzc*6/h^2)	0	0	0	EIzc*2/h
(-EAxc/h)	0	0	0	0	0	EAxc/h	0	0	0	0	0
0	(-EIzc*12/h^3)	0	0	0	(-EIzc*6/h^2)	0	EIzc*12/h^3	0	0	0	(-EIzc*6/h^2)
0	0	(-EIyc*12/h^3)	0	EIyc*6/h^2	0	0	0	EIyc*12/h^3	0	EIyc*6/h^2	0
0	0	0	(-GIxc/h)	0	0	0	0	0	GIxc/h	0	0
0	0	(-EIyc*6/h^2)	0	EIyc*2/h	0	0	0	EIyc*6/h^2	0	EIyc*4/h	0
0	EIzc*6/h^2	0	0	0	EIzc*2/h	0	(-EIzc*6/h^2)	0	0	0	EIzc*4/h

Master Matrix for column

Member Matrix for Member column 5,6,7 and 8

SMi=

785933.3333	0	0	0	0	0	-785933.3333	0	0	0	0	0
0	35866.57778	0	0	0	53799866.67	0	-35866.57778	0	0	0	53799866.67
0	0	2706.666667	0	-4060000	0	0	0	-2706.666667	0	-4060000	0
0	0	0	17680000	0	0	0	0	0	-17680000	0	0
0	0	-4060000	0	8120000000	0	0	0	4060000	0	4060000000	0
0	53799866.67	0	0	0	1.076E+11	0	-53799866.67	0	0	0	53799866667
-785933.3333	0	0	0	0	0	785933.3333	0	0	0	0	0
0	-35866.5778	0	0	0	-53799866.67	0	35866.57778	0	0	0	-53799866.67
0	0	-2706.666667	0	4060000	0	0	0	2706.666667	0	4060000	0
0	0	0	-17680000	0	0	0	0	0	17680000	0	0
0	0	-4060000	0	4060000000	0	0	0	4060000	0	8120000000	0
0	53799866.67	0	0	0	53799866667	0	-53799866.67	0	0	0	1.076E+11

Rotation Matrix of Member(2),Member(4)

Rt1=

0	0	1	0	0	0	0	0	0	0	0	0
0	1	0	0	0	0	0	0	0	0	0	0
-1	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	1	0	0	0	0	0	0
0	0	0	0	1	0	0	0	0	0	0	0
0	0	0	-1	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	1	0	0	0
0	0	0	0	0	0	0	1	0	0	0	0
0	0	0	0	0	0	-1	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	1
0	0	0	0	0	0	0	0	0	0	1	0
0	0	0	0	0	0	0	0	0	-1	0	0

Rotation Matrix of Member(5),Member(6),Member(7),Member(8)

Rt2=

0	1	0	0	0	0	0	0	0	0	0	0
-1	0	0	0	0	0	0	0	0	0	0	0
0	0	1	0	0	0	0	0	0	0	0	0
0	0	0	0	1	0	0	0	0	0	0	0
0	0	0	-1	0	0	0	0	0	0	0	0
0	0	0	0	0	1	0	0	0	0	0	0
0	0	0	0	0	0	0	1	0	0	0	0
0	0	0	0	0	0	-1	0	0	0	0	0
0	0	0	0	0	0	0	0	1	0	0	0
0	0	0	0	0	0	0	0	0	0	1	0
0	0	0	0	0	0	0	0	0	-1	0	0
0	0	0	0	0	0	0	0	0	0	0	1

Sg=transpose Rt*SM*Rt

Global Siffness Marix for Member 2 & 4.

Tran.Rt1*SM2*Rt1=

2356.444444	0	0	0	3534666.667	0	-2356.444444	0	0	0	3534666.667	0
0	81611.55556	0	-122417333	0	0	0	-81611.55556	0	-122417333.3	0	0
0	0	1041400	0	0	0	0	0	-1041400	0	0	0
0	-122417333	0	2.44835E+11	0	0	0	122417333.3	0	1.22417E+11	0	0
3534666.667	0	0	0	7069333333	0	-3534666.667	0	0	0	3534666667	0
0	0	0	0	0	44240000	0	0	0	0	0	-44240000
-2356.444444	0	0	0	-3534666.67	0	2356.444444	0	0	0	-3534666.67	0
0	-81611.5556	0	122417333.3	0	0	0	81611.55556	0	122417333.3	0	0
0	0	-1041400	0	0	0	0	0	1041400	0	0	0
0	-122417333	0	1.22417E+11	0	0	0	122417333.3	0	2.44835E+11	0	0
3534666.667	0	0	0	3534666667	0	-3534666.667	0	0	0	7069333333	0
0	0	0	0	0	-44240000	0	0	0	0	0	44240000

Global Siffness Marix for Member 5, 6, 7 & 8.

Tran.Rt2*SM5*Rt2=

35866.57778	0	0	0	0	-53799866.67	-35866.57778	0	0	0	0	-53799866.67
0	785933.3333	0	0	0	0	0	-785933.3333	0	0	0	0
0	0	2706.666667	4060000	0	0	0	0	-2706.666667	4060000	0	0
0	0	4060000	8120000000	0	0	0	0	-4060000	4060000000	0	0
0	0	0	0	17680000	0	0	0	0	0	-17680000	0
-53799866.67	0	0	0	0	1.076E+11	53799866.67	0	0	0	0	53799866667
-35866.57778	0	0	0	0	53799866.67	35866.57778	0	0	0	0	53799866.67
0	-785933.333	0	0	0	0	0	785933.3333	0	0	0	0
0	0	-2706.666667	-4060000	0	0	0	0	2706.666667	-4060000	0	0
0	0	4060000	4060000000	0	0	0	0	-4060000	8120000000	0	0
0	0	0	0	-17680000	0	0	0	0	0	17680000	0
-53799866.67	0	0	0	0	53799866667	53799866.67	0	0	0	0	1.076E+11

STATIC CONDENSATION: $\begin{pmatrix} m_{tt} & 0 \\ 0 & 0 \end{pmatrix} \begin{pmatrix} \ddot{u}_t \\ \ddot{u}_o \end{pmatrix} + \begin{pmatrix} k_{tt} & k_{to} \\ k_{0t} & k_{00} \end{pmatrix} \begin{pmatrix} u_t \\ u_o \end{pmatrix} = \begin{pmatrix} p_t(t) \\ 0 \end{pmatrix}$

$$u_o = -k_{00}^{-1} k_{0t} u_t$$

Transform matrix $T_r = -k_{00}^{-1} k_{0t}$

$$\hat{k}_{tt} = k_{tt} - k_{0t}^T k_{00}^{-1} k_{0t}$$

	600	0	0	0	0	0	0	0	0	0	0	0
	0	600	0	0	0	0	0	0	0	0	0	0
	0	0	600	0	0	0	0	0	0	0	0	0
	0	0	0	600	0	0	0	0	0	0	0	0
	0	0	0	0	600	0	0	0	0	0	0	0
mtt=	0	0	0	0	0	600	0	0	0	0	0	0
	0	0	0	0	0	0	600	0	0	0	0	0
	0	0	0	0	0	0	0	600	0	0	0	0
	0	0	0	0	0	0	0	0	600	0	0	0
	0	0	0	0	0	0	0	0	0	600	0	0
	0	0	0	0	0	0	0	0	0	0	600	0
	0	0	0	0	0	0	0	0	0	0	0	600

	1079623.022	0	0	-1041400	0	0	0	0	0	-2356.444444	0	0
	0	949156.4444	0	0	-81611.5556	0	0	0	0	0	-81611.5556	0
	0	0	1046463.111	0	0	-2356.444444	0	0	0	0	0	-1041400
	-1041400	0	0	1079623.022	0	0	-2356.444444	0	0	0	0	0
	0	-81611.5556	0	0	949156.4444	0	0	-81611.55556	0	0	0	0
ktt=	0	0	-2356.444444	0	0	1046463.111	0	0	-1041400	0	0	0
	0	0	0	-2356.44444	0	0	1079623.022	0	0	-1041400	0	0
	0	0	0	0	-81611.5556	0	0	949156.4444	0	0	-81611.5556	0
	0	0	0	0	0	-1041400	0	0	1046463.111	0	0	-2356.444444
	-2356.444444	0	0	0	0	0	-1041400	0	0	1079623.022	0	0
	0	-81611.5556	0	0	0	0	0	-81611.55556	0	0	949156.4444	0
	0	0	-1041400	0	0	0	0	0	-2356.444444	0	0	1046463.111

	0	-3534666.67	53799866.67	0	0	0	0	0	0	0	-3534666.67	0
	122417333.3	0	122417333.3	0	0	122417333.3	0	0	0	122417333.3	0	0
	-4060000	-3534666.67	0	0	-3534666.67	0	0	0	0	0	0	0
	0	0	0	0	-3534666.67	53799866.67	0	-3534666.667	0	0	0	0
	0	0	-122417333.3	122417333.3	0	-122417333.3	122417333.3	0	0	0	0	0
kt0=	0	3534666.667	0	-4060000	3534666.667	0	0	0	0	0	0	0
	0	0	0	0	3534666.667	0	0	3534666.667	53799866.67	0	0	0
	0	0	0	-122417333	0	0	-122417333.3	0	-122417333.3	0	0	-122417333.3
	0	0	0	0	0	0	-4060000	3534666.667	0	0	3534666.667	0
	0	3534666.667	0	0	0	0	0	0	0	0	3534666.667	53799866.67
	-122417333.3	0	0	0	0	0	0	0	122417333.3	-122417333.3	0	122417333.3
	0	0	0	0	0	0	0	-3534666.667	0	-4060000	-3534666.67	0

	0	122417333.3	-4060000	0	0	0	0	0	0	0	-122417333	0
	-3534666.667	0	-3534666.667	0	0	3534666.667	0	0	0	3534666.667	0	0
	53799866.67	122417333.3	0	0	-122417333	0	0	0	0	0	0	0
	0	0	0	0	122417333.3	-4060000	0	-122417333.3	0	0	0	0
	0	0	-3534666.667	-3534666.67	0	3534666.667	3534666.667	0	0	0	0	0
k0t=	0	122417333.3	0	53799866.67	-122417333	0	0	0	0	0	0	0
	0	0	0	0	122417333.3	0	0	-122417333.3	-4060000	0	0	0
	0	0	0	-3534666.67	0	0	3534666.667	0	3534666.667	0	0	-3534666.667
	0	0	0	0	0	0	53799866.67	-122417333.3	0	0	122417333.3	0
	0	122417333.3	0	0	0	0	0	0	0	0	-122417333	-4060000
	-3534666.667	0	0	0	0	0	0	0	3534666.667	3534666.667	0	-3534666.667
	0	0	0	0	0	0	0	-122417333.3	0	53799866.67	122417333.3	0

	2.52999E+11	0	0	-44240000	0	0	0	0	0	1.22417E+11	0	0
	0	14156346667	0	0	3534666667	0	0	0	0	0	3534666667	0
	0	0	3.52479E+11	0	0	1.22417E+11	0	0	0	0	0	-44240000
	-44240000	0	0	2.52999E+11	0	0	1.22417E+11	0	0	0	0	0
	0	3534666667	0	0	14156346667	0	0	3534666667	0	0	0	0
k00=	0	0	1.22417E+11	0	0	3.52479E+11	0	0	-44240000	0	0	0
	0	0	0	1.22417E+11	0	0	2.52999E+11	0	0	-44240000	0	0
	0	0	0	0	3534666667	0	0	14156346667	0	0	3534666667	0
	0	0	0	0	0	-44240000	0	0	3.52479E+11	0	0	1.22417E+11
	1.22417E+11	0	0	0	0	0	-44240000	0	0	2.52999E+11	0	0
	0	3534666667	0	0	0	0	0	3534666667	0	0	14156346667	0
	0	0	-44240000	0	0	0	0	0	1.22417E+11	0	0	3.52479E+11

Condensed Stiffness Matrix

	1068813.854	-13868.3767	-588.6204091	-1037862.94	13868.37674	588.6204091	-293.0168761	1.291939754	588.6204091	-886.7545307	-1.29193975	-588.6204091
	-13868.37674	806206.9234	1323.902185	-13868.3767	-18508.1791	0.156011985	-1.291939754	15.28757618	0.156011985	-1.291939754	-1780.6985	1323.902185
	-588.6204091	1323.902185	1044906.858	-588.620409	0.156011985	-885.284959	588.6204091	-0.156011985	-293.9238415	588.6204091	-1323.90218	-1041064.895
	-1037862.935	-13868.3767	-588.6204091	1068813.854	13868.37674	588.6204091	-886.7545307	1.291939754	588.6204091	-293.0168761	-1.29193975	-588.6204091
Kttc	13868.37674	-18508.1791	0.156011985	13868.37674	806206.9234	1323.902185	1.291939754	-1780.698505	1323.902185	1.291939754	15.28757618	0.156011985
	588.6204091	0.156011985	-885.284959	588.6204091	1323.902185	1044906.858	-588.6204091	-1323.902185	-1041064.895	-588.6204091	-0.15601198	-293.9238415
	-293.0168761	-1.29193975	588.6204091	-886.754531	1.291939754	-588.6204091	1068813.854	13868.37674	-588.6204091	-1037862.935	-13868.3767	588.6204091
	1.291939754	15.28757618	-0.156011985	1.291939754	-1780.6985	-1323.902185	13868.37674	806206.9234	-1323.902185	13868.37674	-18508.1791	-0.156011985
	588.6204091	0.156011985	-293.9238415	588.6204091	1323.902185	-1041064.895	-588.6204091	-1323.902185	1044906.858	-588.6204091	-0.15601198	-885.284959
	-886.7545307	-1.29193975	588.6204091	-293.016876	1.291939754	-588.6204091	-1037862.935	13868.37674	-588.6204091	1068813.854	-13868.3767	588.6204091
	-1.291939754	-1780.6985	-1323.902185	-1.29193975	15.28757618	-0.156011985	-13868.37674	-18508.17914	-0.156011985	-13868.37674	806206.9234	-1323.902185
	-588.6204091	1323.902185	-1041064.895	-588.620409	0.156011985	-293.9238415	588.6204091	-0.156011985	-885.284959	588.6204091	-1323.90218	1044906.858

Transform Matrix

	0	-0.00032608	2.09532E-05	0	-3.8427E-08	5.90402E-09	0	3.84266E-08	-4.62959E-09	0	0.000326084	-1.01385E-05
	0.000208108	0	0.000208108	-4.158E-05	0	-0.000208108	4.158E-05	0	4.158E-05	-0.000208108	0	-4.158E-05
	-0.000173569	-0.00025778	0	6.02812E-05	0.000257777	0	1.72075E-08	2.40138E-08	0	-2.77611E-08	-2.4014E-08	0
	0	-3.8427E-08	5.90402E-09	0	-0.00032608	2.09532E-05	0	0.000326084	-1.01385E-05	0	3.84266E-08	-4.62959E-09
Tr=	-4.158E-05	0	0.000208108	0.000208108	0	-0.000208108	-0.000208108	0	4.158E-05	4.158E-05	0	-4.158E-05
	6.02812E-05	-0.00025778	0	-0.00017357	0.000257777	0	-2.77611E-08	2.40138E-08	0	1.72075E-08	-2.4014E-08	0
	0	-3.8427E-08	-4.62959E-09	0	-0.00032608	-1.01385E-05	0	0.000326084	2.09532E-05	0	3.84266E-08	5.90402E-09
	-4.158E-05	0	-4.158E-05	0.000208108	0	4.158E-05	-0.000208108	0	-0.000208108	4.158E-05	0	0.000208108
	1.72075E-08	-2.4014E-08	0	-2.7761E-08	2.40138E-08	0	-0.000173569	0.000257777	0	6.02812E-05	-0.00025778	0
	0	-0.00032608	-1.01385E-05	0	-3.8427E-08	-4.62959E-09	0	3.84266E-08	5.90402E-09	0	0.000326084	2.09532E-05
	0.000208108	0	-4.158E-05	-4.158E-05	0	4.158E-05	4.158E-05	0	-0.000208108	-0.000208108	0	0.000208108
	-2.77611E-08	-2.4014E-08	0	1.72075E-08	2.40138E-08	0	6.02812E-05	0.000257777	0	-0.000173569	-0.00025778	0

CALCULATION AFTER YIELDING OF SUPPORT

After Hinge Formation

Steel Section ISMB 600@ 122.6 Kg/m. is taken FOR BEAMS
step.1

labelling for Beam

L=	3000	E=	200000	Ix=	1659000	Iy=	2.7E+07	Iz=	918130000
EIx=	3.32E+11	EIy=	5.302E+12	EIz=	1.83626E+14	Ax=	15621	G=	80000
EAx=	3.12E+09	GIx=	1.3272E+11	m=	2400	Zz=	3060433	Zy=	252476.1905

step.2. Record basic member stiffness matrix,k for beam.

SMi=

EAx/L	0	0	0	0	0	(-EAx/L)	0	0	0	0	0
0	EIz*12/L^3	0	0	0	EIz*6/L^2	0	(-EIz*12/L^3)	0	0	0	EIz*6/L^2
0	0	EIy*12/L^3	0	(-EIy*6/L^2)	0	0	0	(-EIy*12/L^3)	0	(-EIy*6/L^2)	0
0	0	0	GIx/L	0	0	0	0	0	(-GIx/L)	0	0
0	0	(-EIy*6/L^2)	0	EIy*4/L	0	0	0	EIy*6/L^2	0	EIy*2/L	0
0	EIz*6/L^2	0	0	0	EIz*4/L	0	(-EIz*6/L^2)	0	0	0	EIz*2/L
(-EAx/L)	0	0	0	0	0	EAx/L	0	0	0	0	0
0	(-EIz*12/L^3)	0	0	0	(-EIz*6/L^2)	0	EIz*12/L^3	0	0	0	(-EIz*6/L^2)
0	0	(-EIy*12/L^3)	0	EIy*6/L^2	0	0	0	EIy*12/L^3	0	EIy*6/L^2	0
0	0	0	(-GIx/L)	0	0	0	0	0	GIx/L	0	0
0	0	(-EIy*6/L^2)	0	EIy*2/L	0	0	0	EIy*6/L^2	0	EIy*4/L	0
0	EIz*6/L^2	0	0	0	EIz*2/L	0	(-EIz*6/L^2)	0	0	0	EIz*4/L

Master Matrix for beam 1,2,3,4

SMi=

1041400	0	0	0	0	0	-1041400	0	0	0	0	0
0	81611.55556	0	0	0	122417333.3	0	-81611.55556	0	0	0	122417333.3
0	0	2356.444444	0	-3534666.67	0	0	0	-2356.44444	0	-3534666.667	0
0	0	0	44240000	0	0	0	0	0	-4.4E+07	0	0
0	0	-3534666.667	0	7069333333	0	0	0	3534666.667	0	3534666667	0
0	122417333.3	0	0	0	2.44835E+11	0	-122417333.3	0	0	0	1.22417E+11
-1041400	0	0	0	0	0	1041400	0	0	0	0	0
0	-81611.55556	0	0	0	-122417333	0	81611.55556	0	0	0	-122417333
0	0	-2356.444444	0	3534666.667	0	0	0	2356.444444	0	3534666.667	0
0	0	0	-44240000	0	0	0	0	0	4.4E+07	0	0
0	0	-3534666.667	0	3534666667	0	0	0	3534666.667	0	7069333333	0
0	122417333.3	0	0	0	1.22417E+11	0	-122417333.3	0	0	0	2.44835E+11

Steel Section ISHB 450@ 92.5 Kg/m. is taken COLUMNS

Record basic member stiffness matrix,k for column.

labelling for Column=			Ixc=	663000	Iyc=	30450000	Izc=	403499000	EIxc=	1.326E+11
			EIyc=	6.09E+12	EIzc=	8.06998E+13	Axc=	11789	EAxc=	2357800000
			GIxc=	53040000000	h=	3000	Zzc=	1793328.889	Zyc=	243600

SMi=

EAxc/h	0	0	0	0	0	(-EAxc/h)	0	0	0	0	0
0	EIz*3/h^3	0	0	0	EIzc*3/h^2	0	(-EIz*3/h^3)	0	0	0	EIzc*3/h^2
0	0	EIyc*3/h^3	0	(-EIyc*3/h^2)	0	0	0	(-EIyc*3/h^3)	0	(-EIyc*3/h^2)	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	(-EIyc*3/h^2)	0	EIyc*3/h	0	0	0	EIy*3/h^2	0	0	0
0	EIzc*3/h^2	0	0	0	EIzc*3/h	0	(-EIzc*3/h^2)	0	0	0	0
(-EAxc/h)	0	0	0	0	0	EAxc/h	0	0	0	0	0
0	(-EIzc*3/h^3)	0	0	0	(-EIzc*3/h^2)	0	EIzc*3/h^3	0	0	0	(-EIzc*3/h^2)
0	0	(-EIyc*3/h^3)	0	EIyc*3/h^2	0	0	0	EIy*3/h^3	0	EIyc*3/h^2	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	(-EIyc*3/h^2)	0	0	0	0	0	EIy*3/h^2	0	EIyc*3/h	0
0	EIzc*3/h^2	0	0	0	0	0	(-EIzc*3/h^2)	0	0	0	EIzc*3/h

Master Matrix for column 5,6,7,8

SMi=

785933	0	0	0	0	0	-785933.3333	0	0	0	0	0
0	8966.644444	0	0	0	26899933.33	0	-20402.88889	0	0	0	26899933.33
0	0	676.6666667	0	-2030000	0	0	0	-676.666667	0	-2030000	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	-2030000	0	6090000000	0	0	0	1767333.333	0	0	0
0	26899933.33	0	0	0	80699800000	0	-26899933.33	0	0	0	0
-785933	0	0	0	0	0	785933.3333	0	0	0	0	0
0	-8966.644444	0	0	0	-26899933.3	0	8966.644444	0	0	0	-26899933.3
0	0	-676.6666667	0	2030000	0	0	0	589.1111111	0	2030000	0
0	0	0	0	0	0	0	0	0	0	0	0
0	0	-2030000	0	0	0	0	0	1767333.333	0	6090000000	0
0	26899933.33	0	0	0	0	0	-26899933.33	0	0	0	80699800000

Rotation Matrix of Member(2),Member(4)

Rt1=

0	0	1	0	0	0	0	0	0	0	0	0
0	1	0	0	0	0	0	0	0	0	0	0
-1	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	1	0	0	0	0	0	0
0	0	0	0	1	0	0	0	0	0	0	0
0	0	0	-1	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	1	0	0	0
0	0	0	0	0	0	0	1	0	0	0	0
0	0	0	0	0	0	-1	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	1
0	0	0	0	0	0	0	0	0	0	1	0
0	0	0	0	0	0	0	0	0	-1	0	0

er(5),Member(6),Member(7),Member(8)

Rt2=

0	1	0	0	0	0	0	0	0	0	0	0
-1	0	0	0	0	0	0	0	0	0	0	0
0	0	1	0	0	0	0	0	0	0	0	0
0	0	0	0	1	0	0	0	0	0	0	0
0	0	0	-1	0	0	0	0	0	0	0	0
0	0	0	0	0	1	0	0	0	0	0	0
0	0	0	0	0	0	0	1	0	0	0	0
0	0	0	0	0	0	-1	0	0	0	0	0
0	0	0	0	0	0	0	0	1	0	0	0
0	0	0	0	0	0	0	0	0	0	1	0
0	0	0	0	0	0	0	0	0	-1	0	0
0	0	0	0	0	0	0	0	0	0	0	1

Sg=transpose Rt*SM*Rt

Global Stiffness Matrix of 2 & 4

2356.444444	0	0	0	3534666.667	0	-2356.444444	0	0	0	3534666.667	0
0	81611.55556	0	-122417333	0	0	0	-81611.5556	0	-122417333.3	0	0
0	0	1041400	0	0	0	0	0	-1041400	0	0	0
0	-122417333.3	0	2.44835E+11	0	0	0	122417333.3	0	1.22417E+11	0	0
3534666.667	0	0	0	7069333333	0	-3534666.667	0	0	0	3534666667	0
0	0	0	0	0	44240000	0	0	0	0	0	-44240000
-2356.444444	0	0	0	-3534666.67	0	2356.444444	0	0	0	-3534666.67	0
0	-81611.55556	0	122417333.3	0	0	0	81611.55556	0	122417333.3	0	0
0	0	-1041400	0	0	0	0	0	1041400	0	0	0
0	-122417333.3	0	1.22417E+11	0	0	0	122417333.3	0	2.44835E+11	0	0
3534666.667	0	0	0	3534666667	0	-3534666.667	0	0	0	7069333333	0
0	0	0	0	0	-44240000	0	0	0	0	0	44240000

Global Stiffness Matrix of 5,6,7&8

8966.644444	0	0	0	0	-26899933.33	-20402.88889	0	0	0	0	-26899933.33
0	785933.3333	0	0	0	0	0	-785933.333	0	0	0	0
0	0	676.6667	2030000	0	0	0	0	-676.667	2030000	0	0
0	0	2030000	6090000000	0	0	0	0	-1767333	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
-26899933.33	0	0	0	0	80699800000	26899933.33	0	0	0	0	0
-8966.644444	0	0	0	0	26899933.33	8966.644444	0	0	0	0	26899933.33
0	-785933.3333	0	0	0	0	0	785933.3333	0	0	0	0
0	0	-676.6667	-2030000	0	0	0	0	589.111	-2030000	0	0
0	0	2030000	0	0	0	0	0	-1767333	6090000000	0	0
0	0	0	0	0	0	0	0	0	0	0	0
-26899933.33	0	0	0	0	0	26899933.33	0	0	0	0	80699800000

STATIC CONDENSATION:

$$\begin{pmatrix} m_{tt} & 0 \\ 0 & 0 \end{pmatrix}\begin{pmatrix} \ddot{u}_t \\ \ddot{u}_0 \end{pmatrix} + \begin{pmatrix} k_{tt} & k_{t0} \\ k_{0t} & k_{00} \end{pmatrix}\begin{pmatrix} u_t \\ u_0 \end{pmatrix} = \begin{pmatrix} p_t(t) \\ 0 \end{pmatrix}$$

$$u_o = -k_{00}^{-1} k_{0t} u_t$$

Transfrom Matrix $$T_r = -k_{00}^{-1} k_{0t}$$

$$\hat{k}_{tt} = k_{tt} - k_{0t}^{T} k_{00}^{-1} k_{0t}$$

mtt=

600	0	0	0	0	0	0	0	0	0	0	0
0	600	0	0	0	0	0	0	0	0	0	0
0	0	600	0	0	0	0	0	0	0	0	0
0	0	0	600	0	0	0	0	0	0	0	0
0	0	0	0	600	0	0	0	0	0	0	0
0	0	0	0	0	600	0	0	0	0	0	0
0	0	0	0	0	0	600	0	0	0	0	0
0	0	0	0	0	0	0	600	0	0	0	0
0	0	0	0	0	0	0	0	600	0	0	0
0	0	0	0	0	0	0	0	0	600	0	0
0	0	0	0	0	0	0	0	0	0	600	0
0	0	0	0	0	0	0	0	0	0	0	600

ktt=

1052723.089	0	0	-1041400	0	0	0	0	0	-2356.444444	0	0
0	949156.4444	0	0	-81611.5556	0	0	0	0	0	-81611.5556	0
0	0	1044346	0	0	-2356.444444	0	0	0	0	0	-1041400
-1041400	0	0	1052723.089	0	0	-2356.444444	0	0	0	0	0
0	-81611.55556	0	0	949156.4444	0	0	-81611.5556	0	0	0	0
0	0	-2356.444	0	0	1044345.556	0	0	-1041400	0	0	0
0	0	0	-2356.44444	0	0	1052723.089	0	0	-1041400	0	0
0	0	0	0	-81611.5556	0	0	949156.4444	0	0	-81611.5556	0
0	0	0	0	0	-1041400	0	0	1044346	0	0	-2356.444444
-2356.444444	0	0	0	0	0	-1041400	0	0	1052723.089	0	0
0	-81611.55556	0	0	0	0	0	-81611.5556	0	0	949156.4444	0
0	0	-1041400	0	0	0	0	0	-2356.44	0	0	1044345.556

kt0=

0	-3534666.667	26899933	0	0	0	0	0	0	0	-3534666.67	0
122417333.3	0	1.22E+08	0	0	122417333.3	0	0	0	122417333.3	0	0
-2030000	-3534666.667	0	0	-3534666.67	0	0	0	0	0	0	0
0	0	0	0	-3534666.67	26899933.33	0	-3534666.67	0	0	0	0
0	0	-1.22E+08	122417333.3	0	-122417333.3	122417333.3	0	0	0	0	0
0	3534666.667	0	-2030000	3534666.667	0	0	0	0	0	0	0
0	0	0	0	3534666.667	0	0	3534666.667	2.7E+07	0	0	0
0	0	0	-122417333	0	0	-122417333.3	0	-1.2E+08	0	0	-122417333.3
0	0	0	0	0	0	-2030000	3534666.667	0	0	3534666.667	0
0	3534666.667	0	0	0	0	0	0	0	0	3534666.667	26899933.33
-122417333.3	0	0	0	0	0	0	0	1.2E+08	-122417333.3	0	122417333.3
0	0	0	0	0	0	0	-3534666.67	0	-2030000	-3534666.67	0

k0t=

0	122417333.3	-1767333	0	0	0	0	0	0	0	-122417333	0
-3534666.667	0	-3534667	0	0	3534666.667	0	0	0	3534666.667	0	0
26899933.33	122417333.3	0	0	-122417333	0	0	0	0	0	0	0
0	0	0	0	122417333.3	-1767333.333	0	-122417333	0	0	0	0
0	0	-3534667	-3534666.67	0	3534666.667	3534666.667	0	0	0	0	0
0	122417333.3	0	26899933.33	-122417333	0	0	0	0	0	0	0
0	0	0	0	122417333.3	0	0	-122417333	-1767333	0	0	0
0	0	0	-3534666.67	0	0	3534666.667	0	3534667	0	0	-3534666.667
0	0	0	0	0	0	26899933.33	-122417333	0	0	122417333.3	0
0	122417333.3	0	0	0	0	0	0	0	0	-122417333	-1767333.333
-3534666.667	0	0	0	0	0	0	0	3534667	3534666.667	0	-3534666.667
0	0	0	0	0	0	0	-122417333	0	26899933.33	122417333.3	0

k00=

2.50969E+11	0	0	-44240000	0	0	0	0	0	1.22417E+11	0	0
0	14138666667	0	0	3534666667	0	0	0	0	0	3534666667	0
0	0	3.26E+11	0	0	1.22417E+11	0	0	0	0	0	-44240000
-44240000	0	0	2.50969E+11	0	0	1.22417E+11	0	0	0	0	0
0	3534666667	0	0	14138666667	0	0	3534666667	0	0	0	0
0	0	1.22E+11	0	0	3.25579E+11	0	0	-4.4E+07	0	0	0
0	0	0	1.22417E+11	0	0	2.50969E+11	0	0	-44240000	0	0
0	0	0	0	3534666667	0	0	14138666667	0	0	3534666667	0
0	0	0	0	0	-44240000	0	0	3.3E+11	0	0	1.22417E+11
1.22417E+11	0	0	0	0	0	-44240000	0	0	2.50969E+11	0	0
0	3534666667	0	0	0	0	0	3534666667	0	0	14138666667	0
0	0	-44240000	0	0	0	0	0	1.2E+11	0	0	3.25579E+11

Condensed Stiffness Matrix

Kttc=

1048661.841	-7350.551888	-589.1111	-1040132.18	7350.551888	589.1111111	-294.2475094	0.725873415	589.111	-884.134216	-0.72587342	-589.1111111
-7350.551888	801983.2639	579.4328	-7350.55189	-14718.6729	0.068653058	-0.725873415	16.11741022	0.06865	-0.725873415	-1347.37501	579.4328061
-589.1111111	579.4328061	1042856	-589.111111	0.068653058	-883.6713431	589.1111111	-0.06865306	-294.552	589.1111111	-579.432806	-1041097.478
-1040132.182	-7350.551888	-589.1111	1048661.841	7350.551888	589.1111111	-884.134216	0.725873415	589.111	-294.2475094	-0.72587342	-589.1111111
7350.551888	-14718.67293	0.068653	7350.551888	801983.2639	579.4328061	0.725873415	-1347.37501	579.433	0.725873415	16.11741022	0.068653058
589.1111111	0.068653058	-883.6713	589.1111111	579.4328061	1042856.446	-589.1111111	-579.432806	-1041097	-589.1111111	-0.06865306	-294.5518702
-294.2475094	-0.725873415	589.1111	-884.134216	0.725873415	-589.1111111	1048661.841	7350.551888	-589.111	-1040132.182	-7350.55189	589.1111111
0.725873415	16.11741022	-0.068653	0.725873415	-1347.37501	-579.4328061	7350.551888	801983.2639	-579.433	7350.551888	-14718.6729	-0.068653058
589.1111111	0.068653058	-294.5519	589.1111111	579.4328061	-1041097.478	-589.1111111	-579.432806	1042856	-589.1111111	-0.06865306	-883.6713431
-884.134216	-0.725873415	589.1111	-294.247509	0.725873415	-589.1111111	-1040132.182	7350.551888	-589.111	1048661.841	-7350.55189	589.1111111
-0.725873415	-1347.375006	-579.4328	-0.72587342	16.11741022	-0.068653058	-7350.551888	-14718.6729	-0.06865	-7350.551888	801983.2639	-579.4328061
-589.1111111	579.4328061	-1041097	-589.111111	0.068653058	-294.5518702	589.1111111	-0.06865306	-883.671	589.1111111	-579.432806	1042856.446

RELATION BETWEEN U0 AND Ut

Transfrom Matrix

Tr=

0	-0.000327857	9.24E-06	0	-3.8846E-08	2.64605E-09	0	3.88456E-08	-2.1E-09	0	0.000327857	-4.5074E-06
0.000208333	0	0.000208	-4.1667E-05	0	-0.000208333	4.16667E-05	0	4.2E-05	-0.000208333	0	-4.16667E-05
-9.62259E-05	-0.000273255	0	3.61809E-05	0.000273255	0	1.14516E-08	2.69842E-08	0	-1.73811E-08	-2.6984E-08	0
0	-3.88456E-08	2.65E-09	0	-0.00032786	9.24065E-06	0	0.000327857	-4.5E-06	0	3.88456E-08	-2.08524E-09
-4.16667E-05	0	0.000208	0.000208333	0	-0.000208333	-0.000208333	0	4.2E-05	4.16667E-05	0	-4.16667E-05
3.61809E-05	-0.000273255	0	-9.6226E-05	0.000273255	0	-1.73811E-08	2.69842E-08	0	1.14516E-08	-2.6984E-08	0
0	-3.88456E-08	-2.09E-09	0	-0.00032786	-4.5074E-06	0	0.000327857	9.2E-06	0	3.88456E-08	2.64605E-09
-4.16667E-05	0	-4.17E-05	0.000208333	0	4.16667E-05	-0.000208333	0	-0.00021	4.16667E-05	0	0.000208333
1.14516E-08	-2.69842E-08	0	-1.7381E-08	2.69842E-08	0	-9.62259E-05	0.000273255	0	3.61809E-05	-0.00027326	0
0	-0.000327857	-4.51E-06	0	-3.8846E-08	-2.08524E-09	0	3.88456E-08	2.6E-09	0	0.000327857	9.24065E-06
0.000208333	0	-4.17E-05	-4.1667E-05	0	4.16667E-05	4.16667E-05	0	-0.00021	-0.000208333	0	0.000208333
-1.73811E-08	-2.69842E-08	0	1.14516E-08	2.69842E-08	0	3.61809E-05	0.000273255	0	-9.62259E-05	-0.00027326	0

The structure will yield at 0.18g.

And the structure will Collapse at 0.40g

PLOTS OF DEFORMATION FOR EACH DEGREE OF FREEDOM AT SP-1

For Member 5

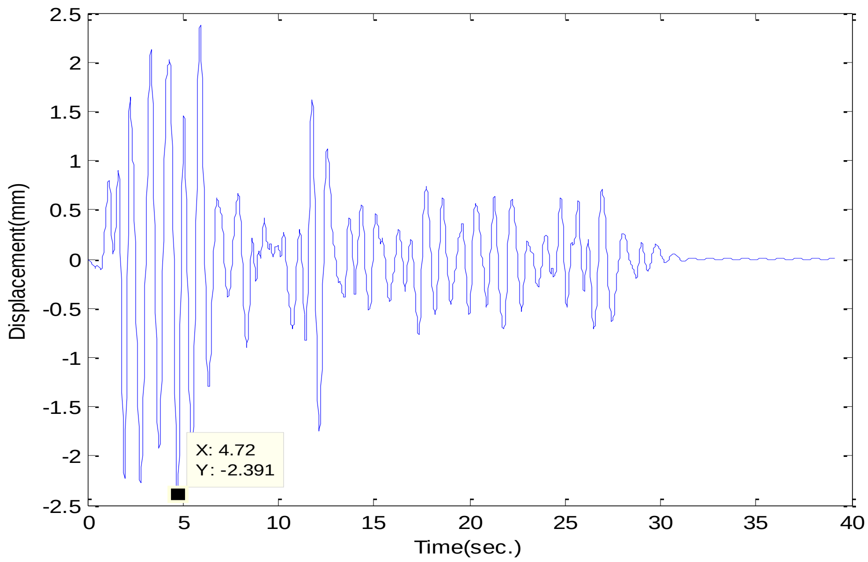


Fig: 8.1 Deformation Response for 1st degree of freedom at SP-1

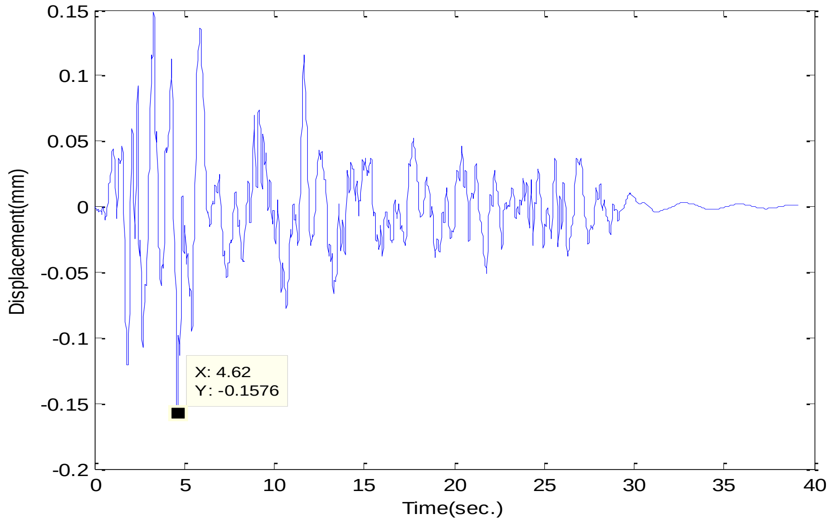


Fig: 8.2 Deformation Response for 2nd degree of freedom at SP-1

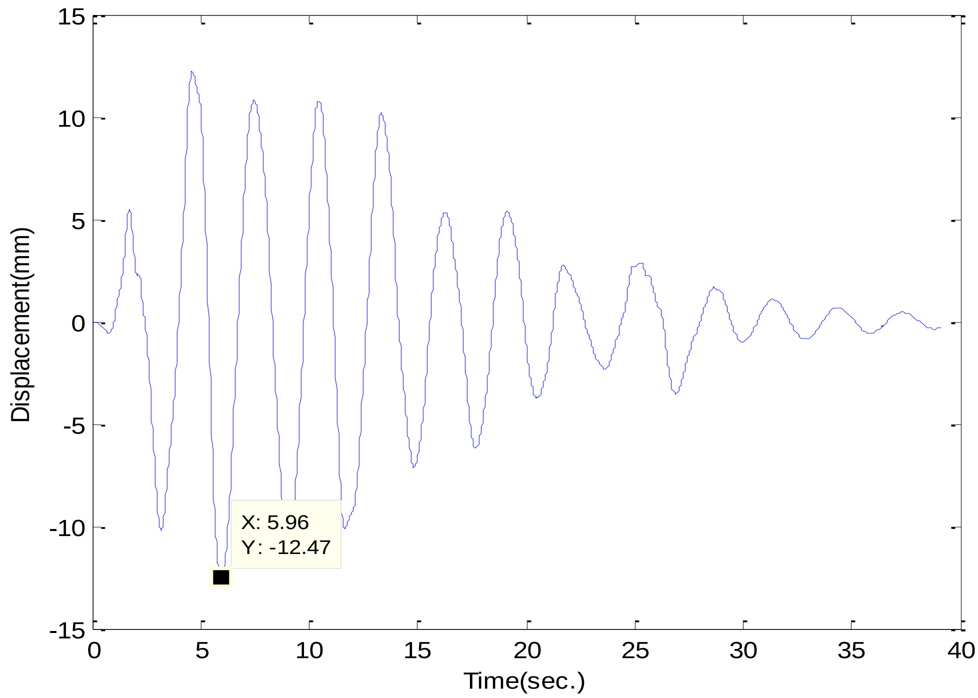


Fig: 8.3 Deformation Response for 3rd degree of freedom at SP-1

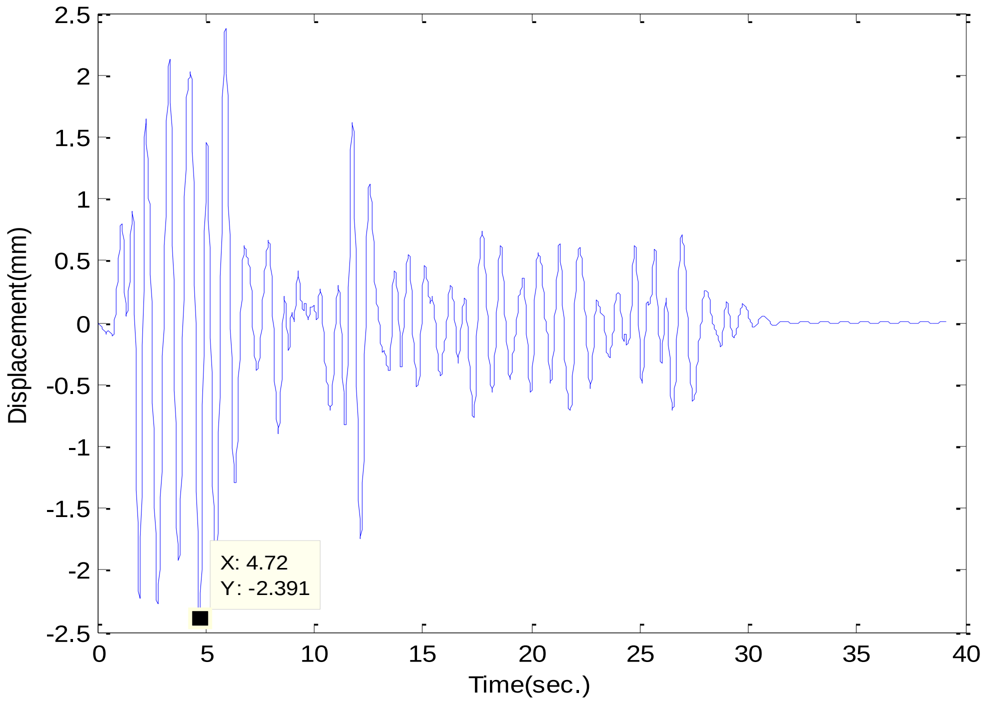


Fig: 8.4 Deformation Response for 4th degree of freedom at SP-1

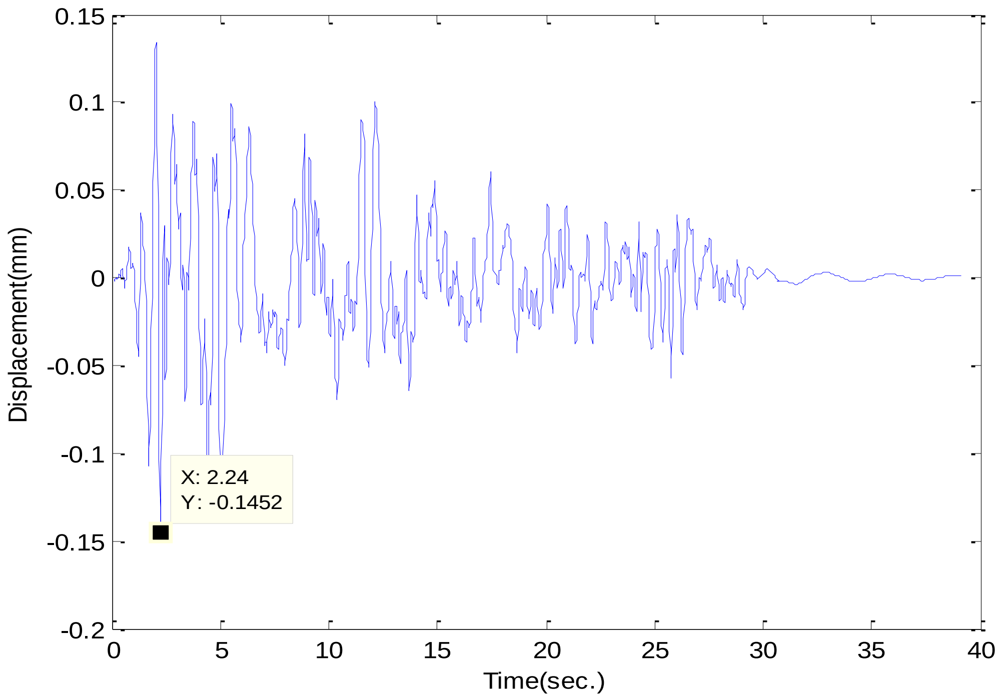


Fig: 8.5 Deformation Response for 5th degree of freedom at SP-1

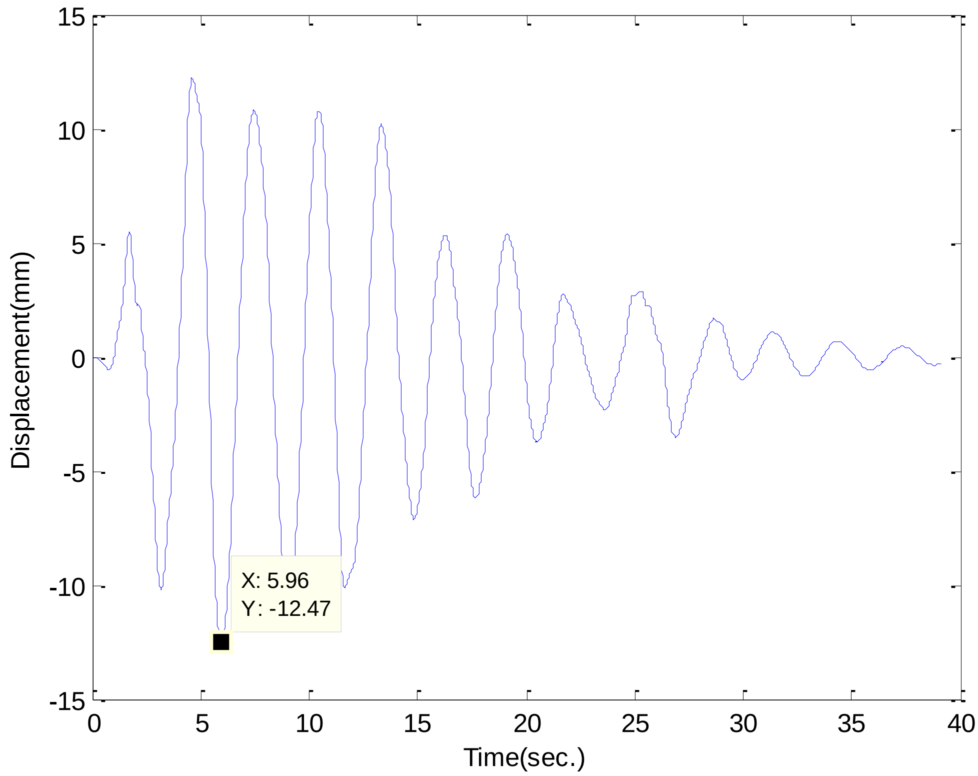


Fig: 8.6 Deformation Response for 6th degree of freedom at SP-1

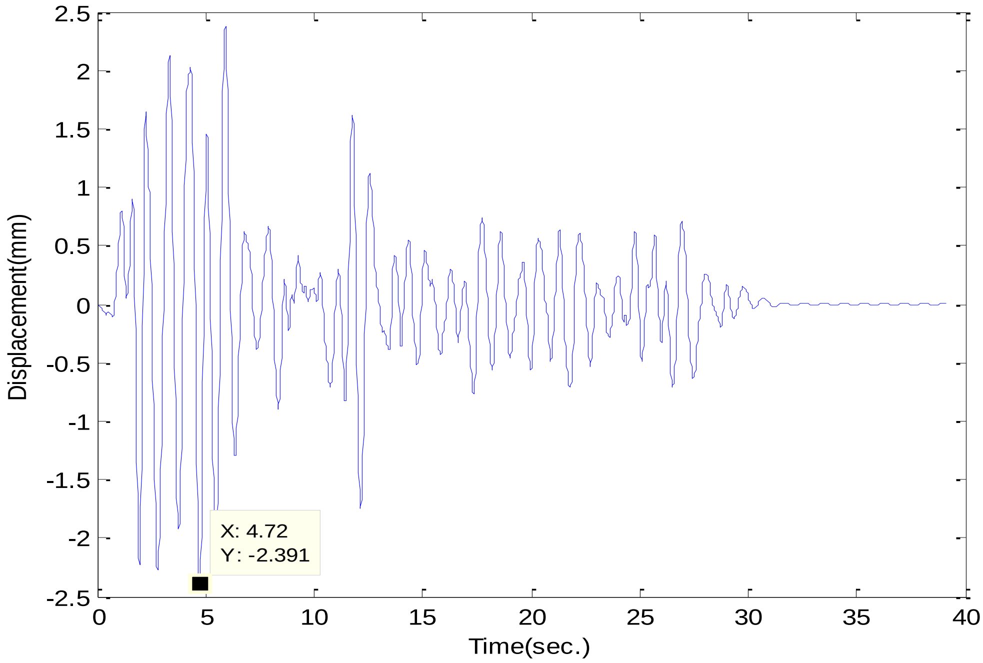


Fig: 8.7 Deformation Response for 7th degree of freedom at SP-1

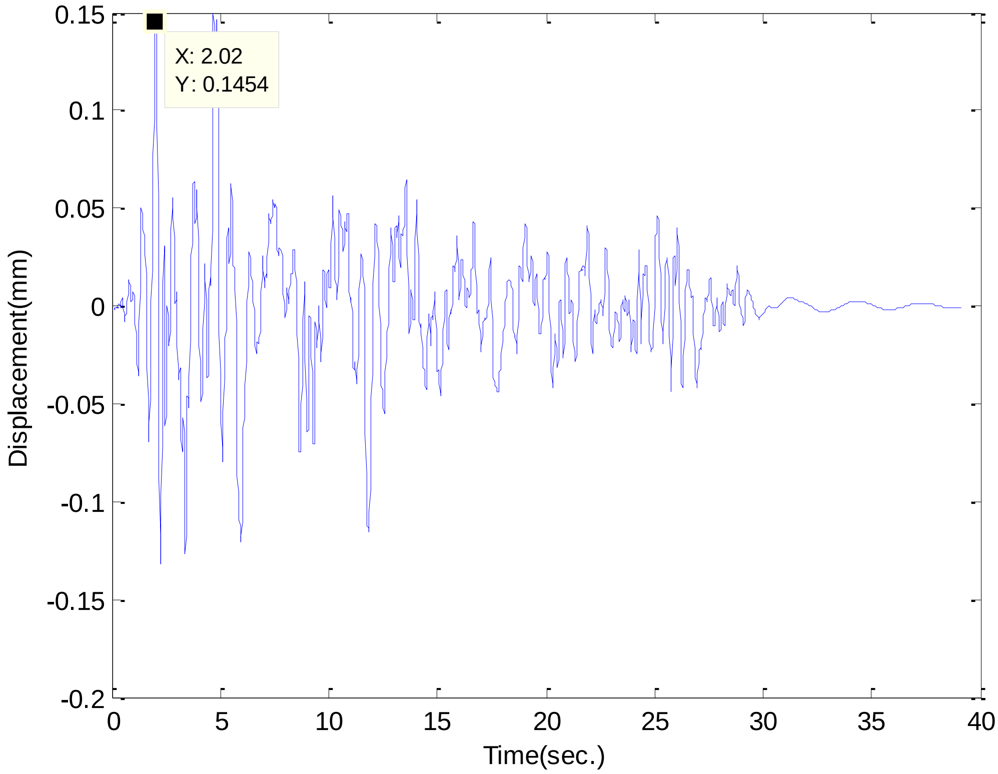


Fig: 8.8 Deformation Response for 8th degree of freedom at SP-1

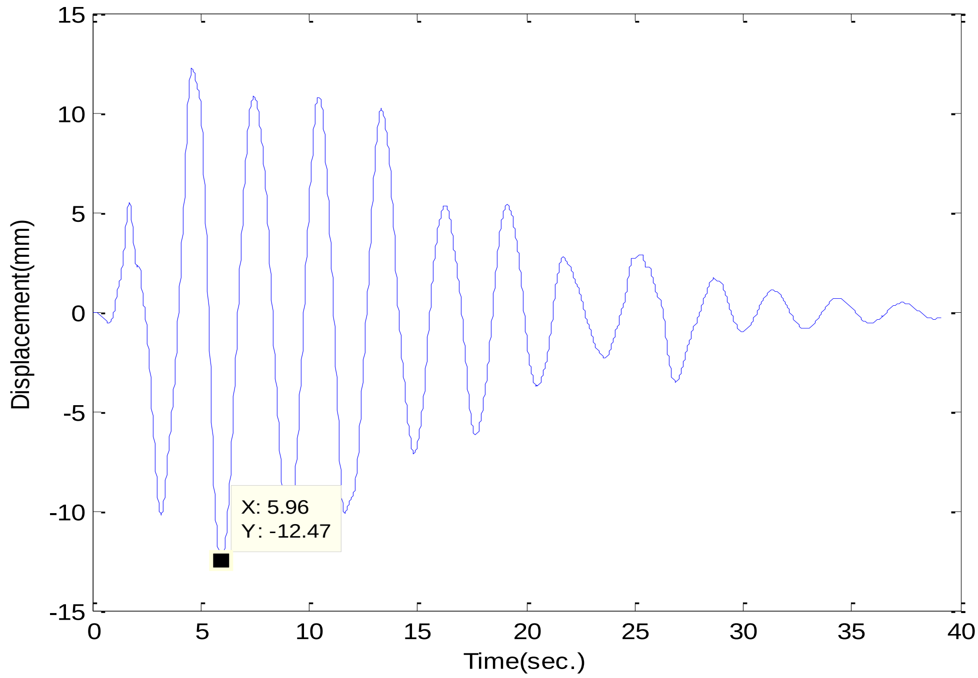


Fig: 8.9 Deformation Response for 9th degree of freedom at SP-1

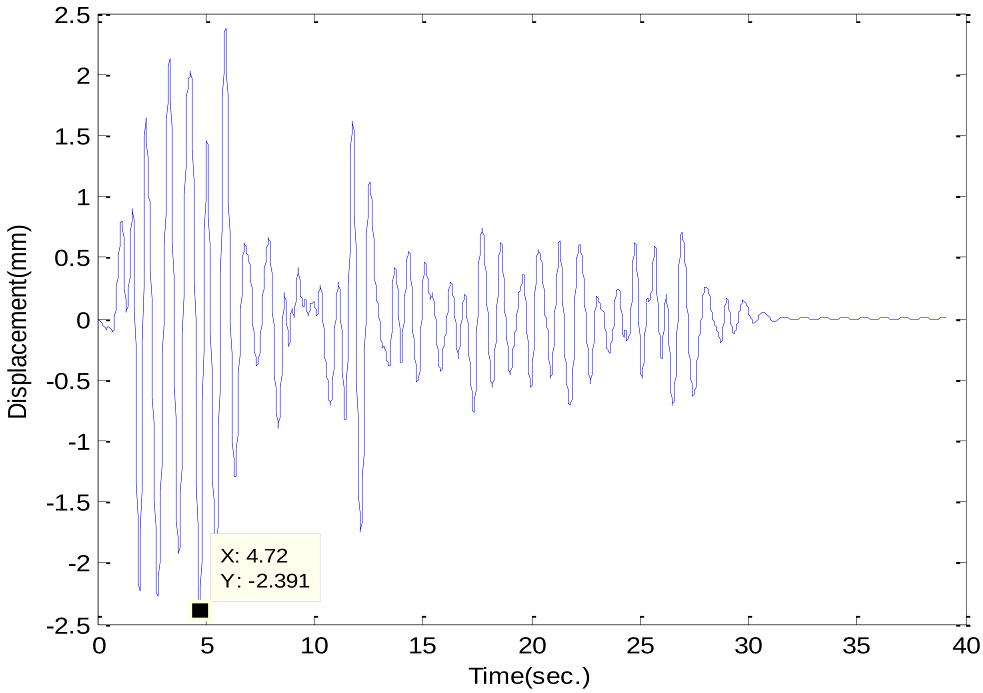


Fig: 8.10 Deformation Response for 10th degree of freedom at SP-1

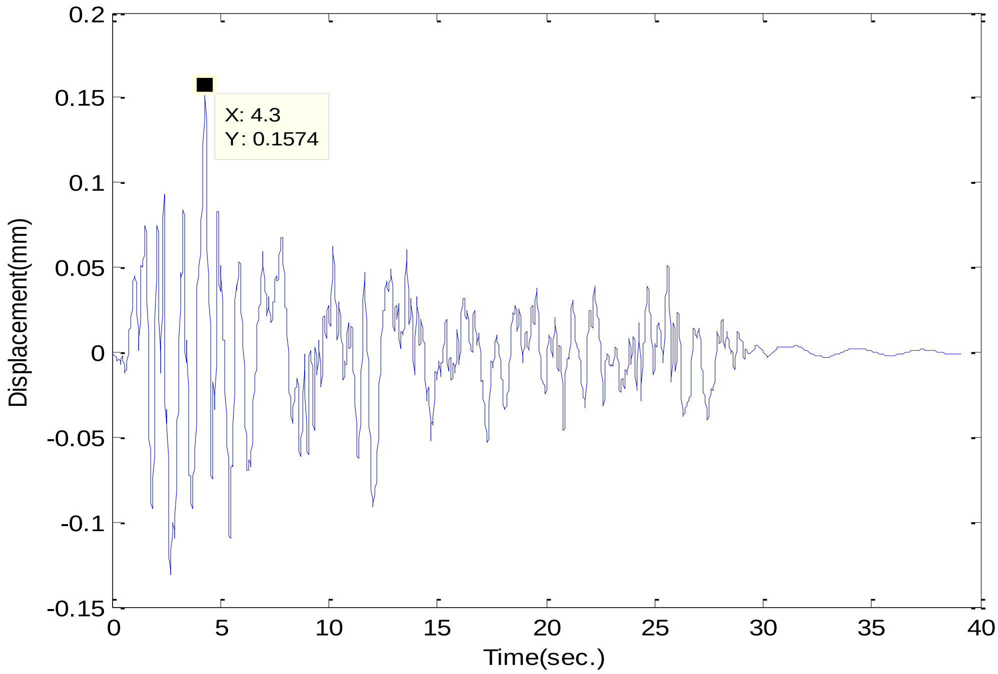


Fig: 8.11 Deformation Response for 11th degree of freedom at SP-1

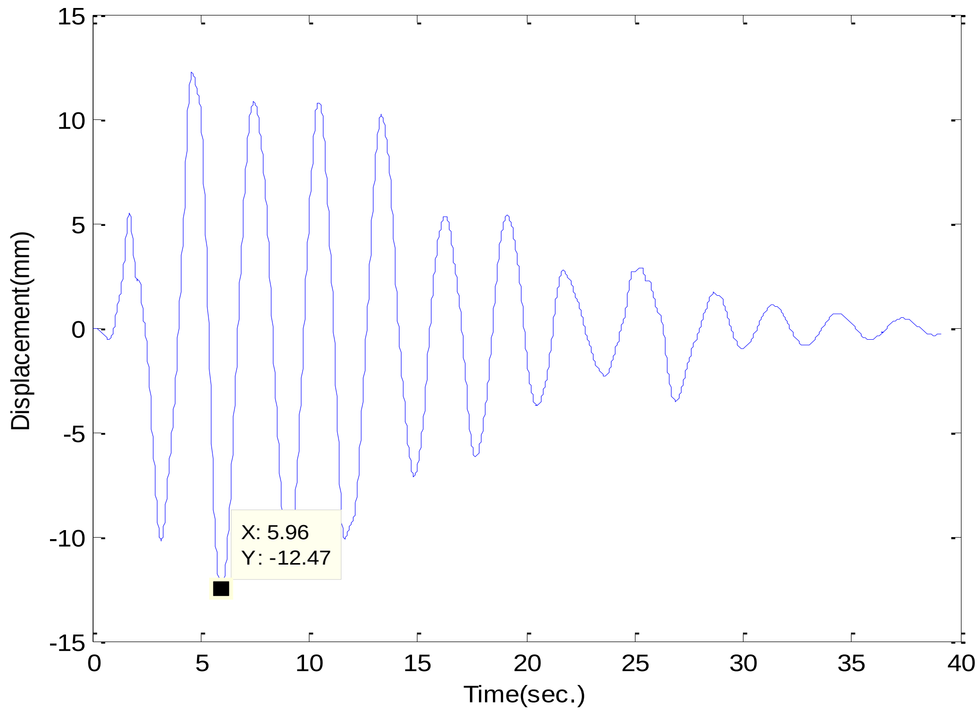


Fig: 8.12 Deformation Response for 12th degree of freedom at SP-1

FORCES IN MEMBER 5

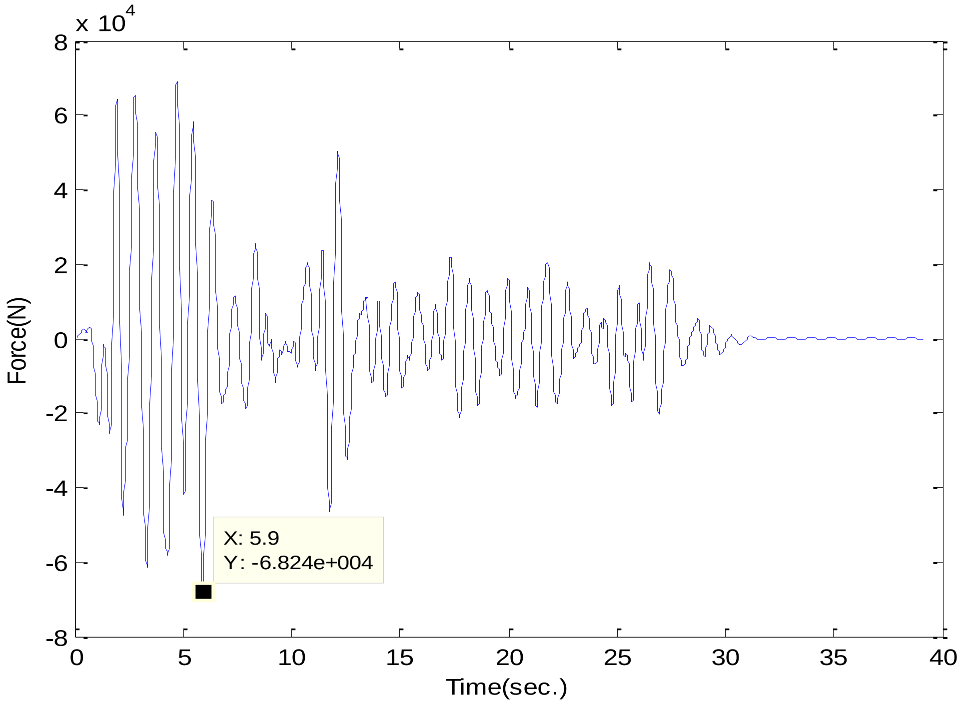


Fig: 8.13 Forces corresponding to 1st degree of freedom at SP-1

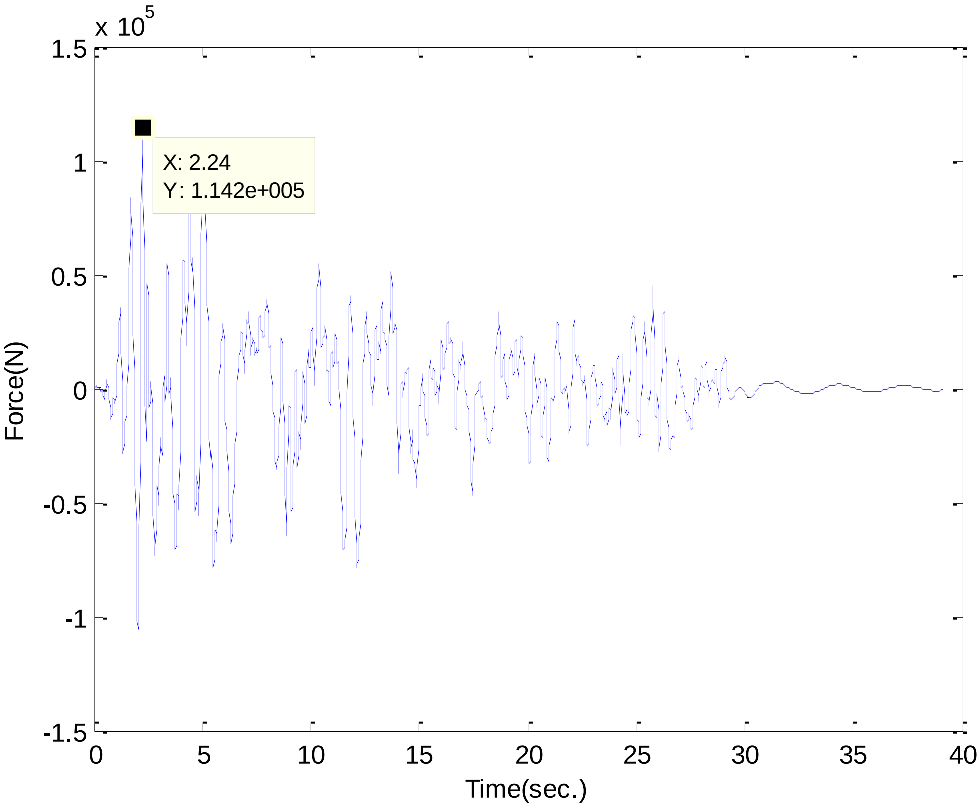


Fig: 8.14 Forces corresponding to 2nd degree of freedom at SP-1

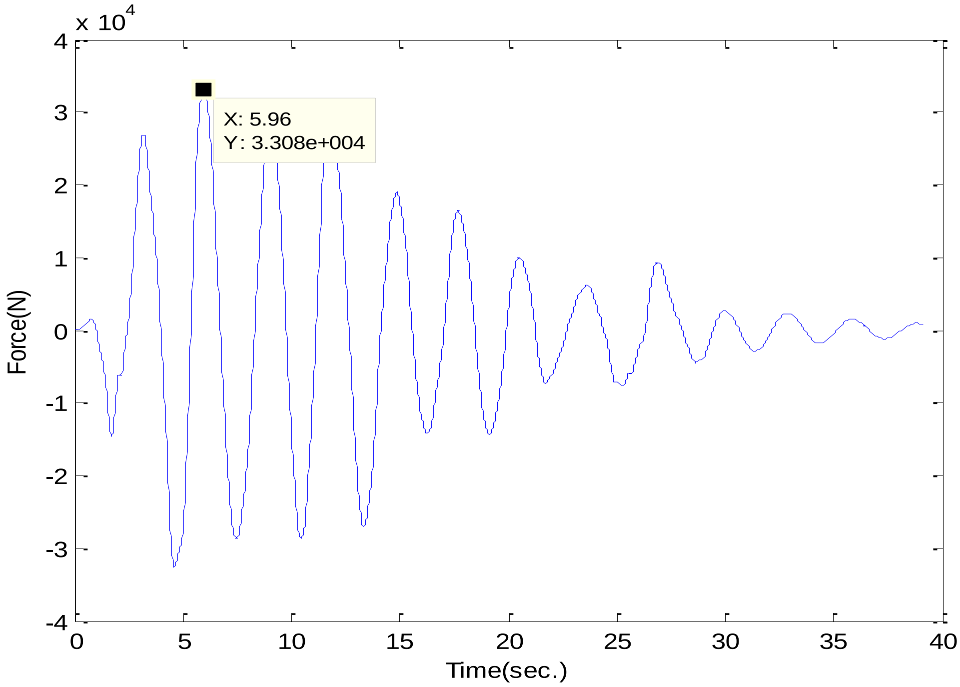


Fig: 8.15 Forces corresponding to 3rd degree of freedom at SP-1

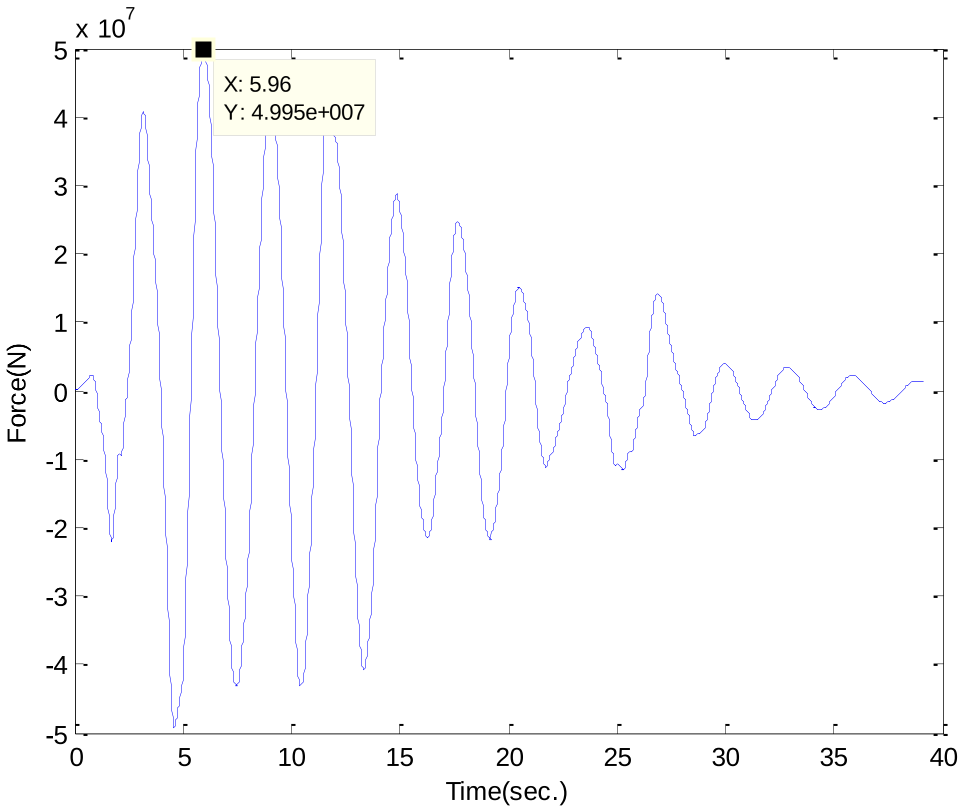


Fig: 8.16 Forces corresponding to 4th degree of freedom at SP-1

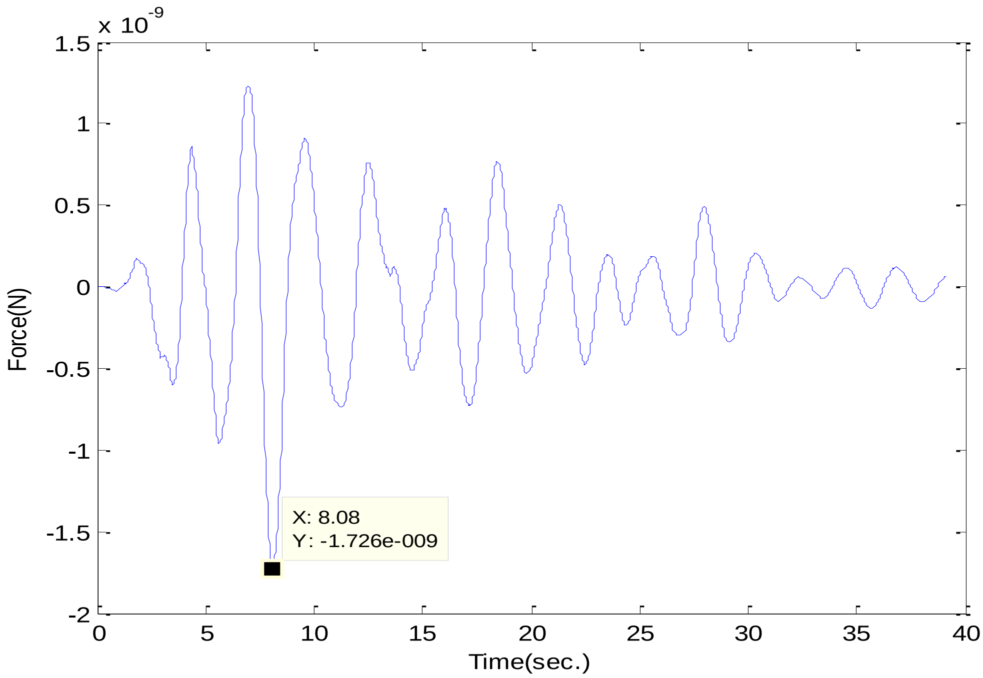


Fig: 8.17 Forces corresponding to 5th degree of freedom at SP-1

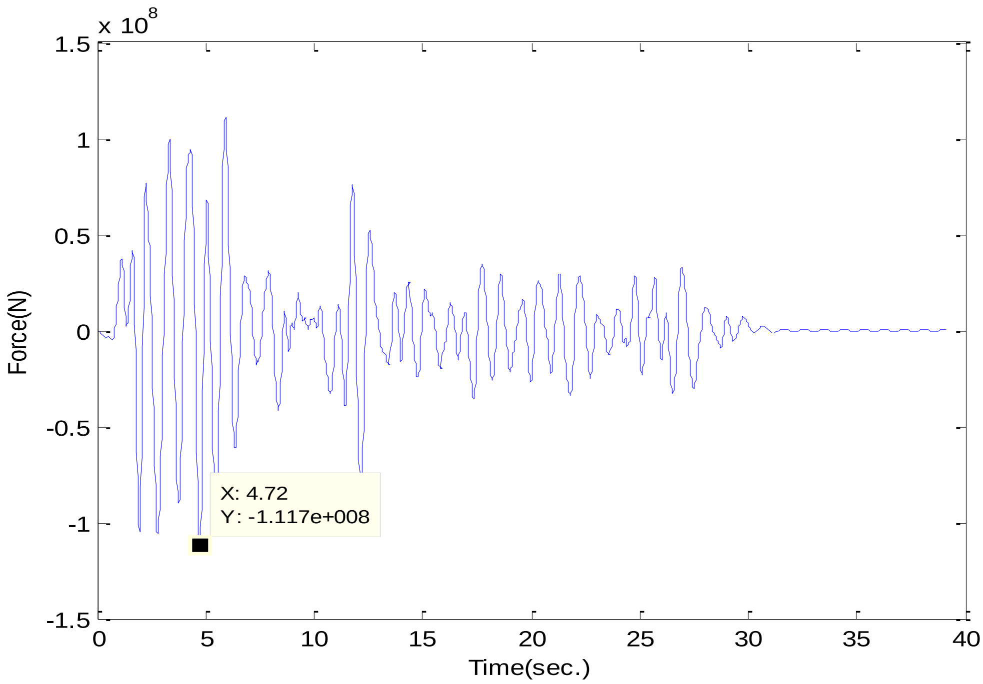


Fig: 8.18 Forces corresponding to 6th degree of freedom at SP-1

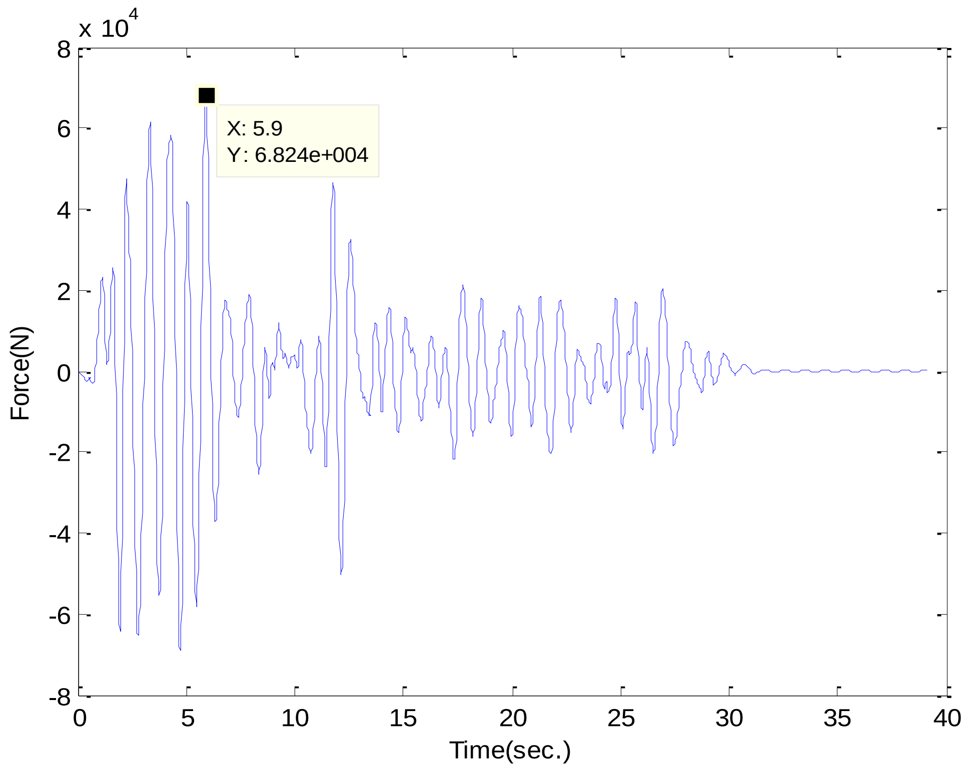


Fig: 8.19 Forces corresponding to 7th degree of freedom at SP-1

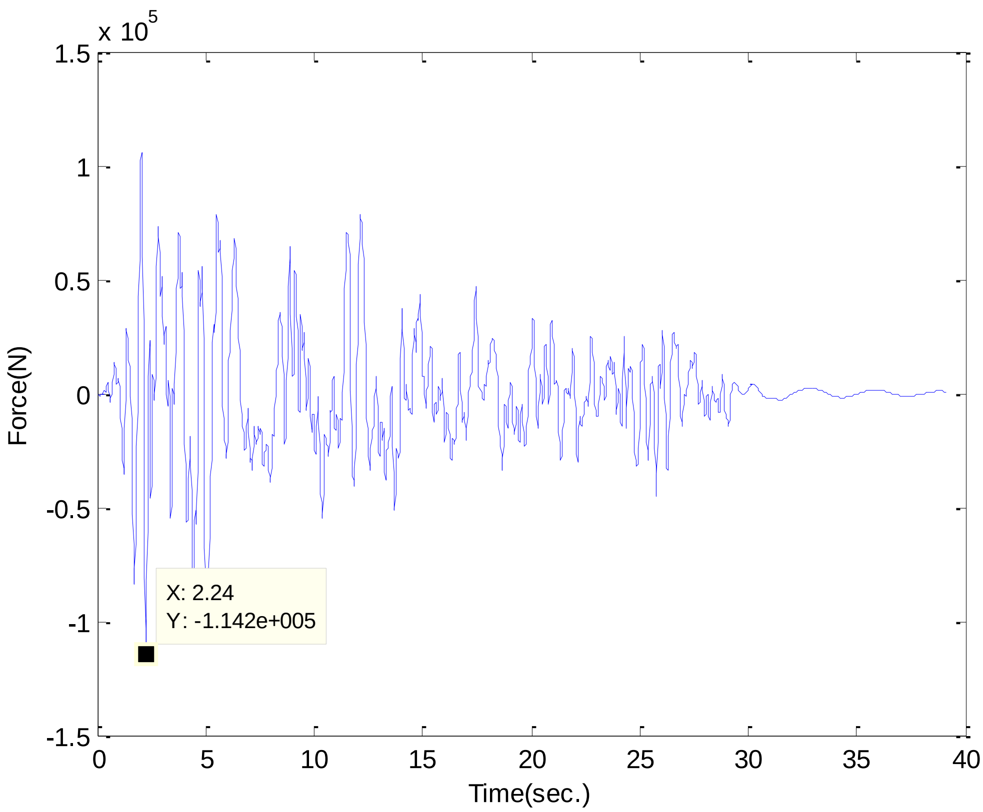


Fig: 8.20 Forces corresponding to 8th degree of freedom at SP-1

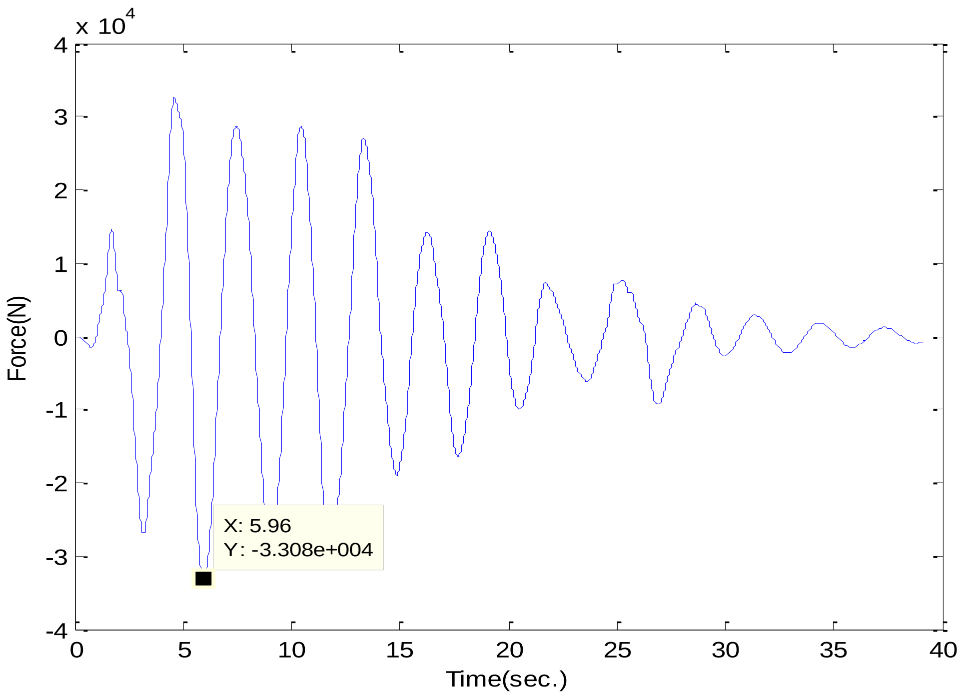


Fig: 8.21 Forces corresponding to 9th degree of freedom at SP-1

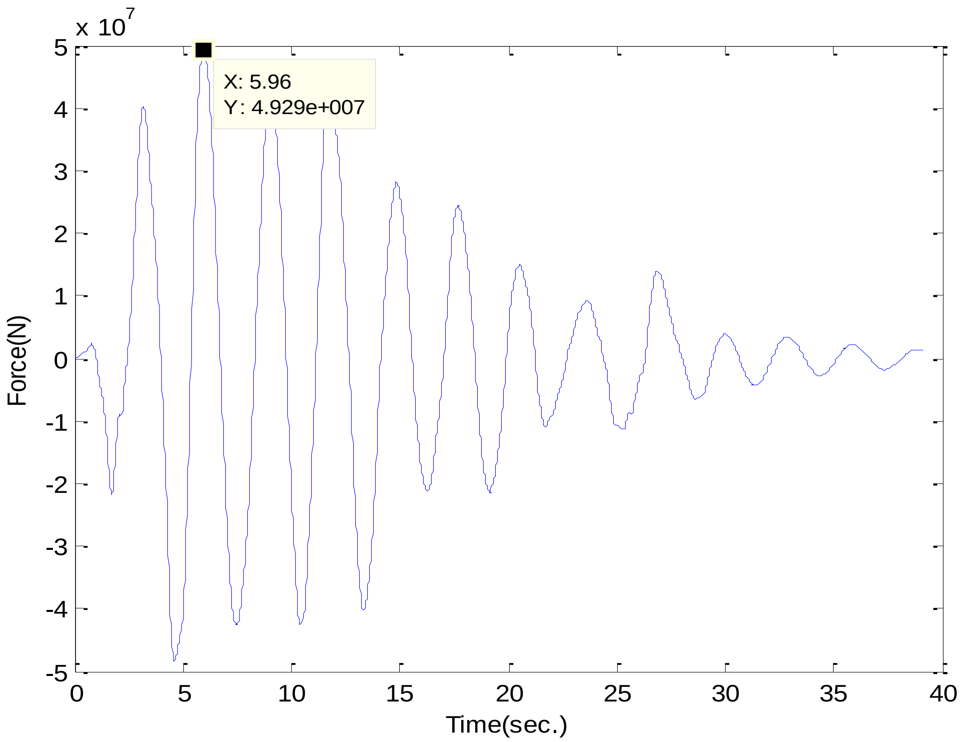


Fig: 8.22 Forces corresponding to 10th degree of freedom at SP-1

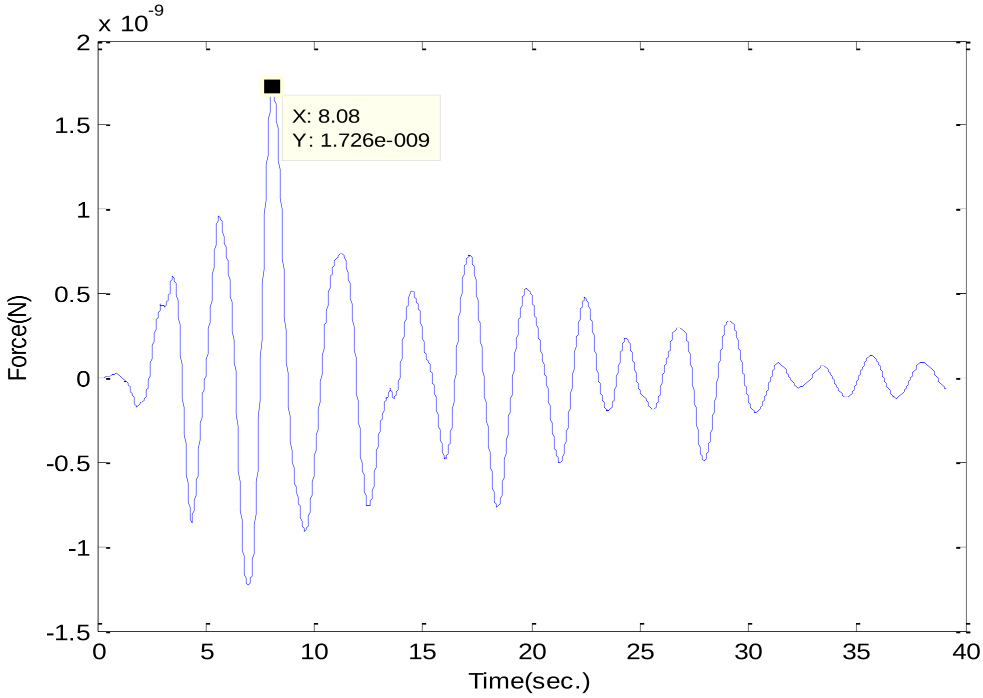


Fig: 8.23 Forces corresponding to 11th degree of freedom at SP-1

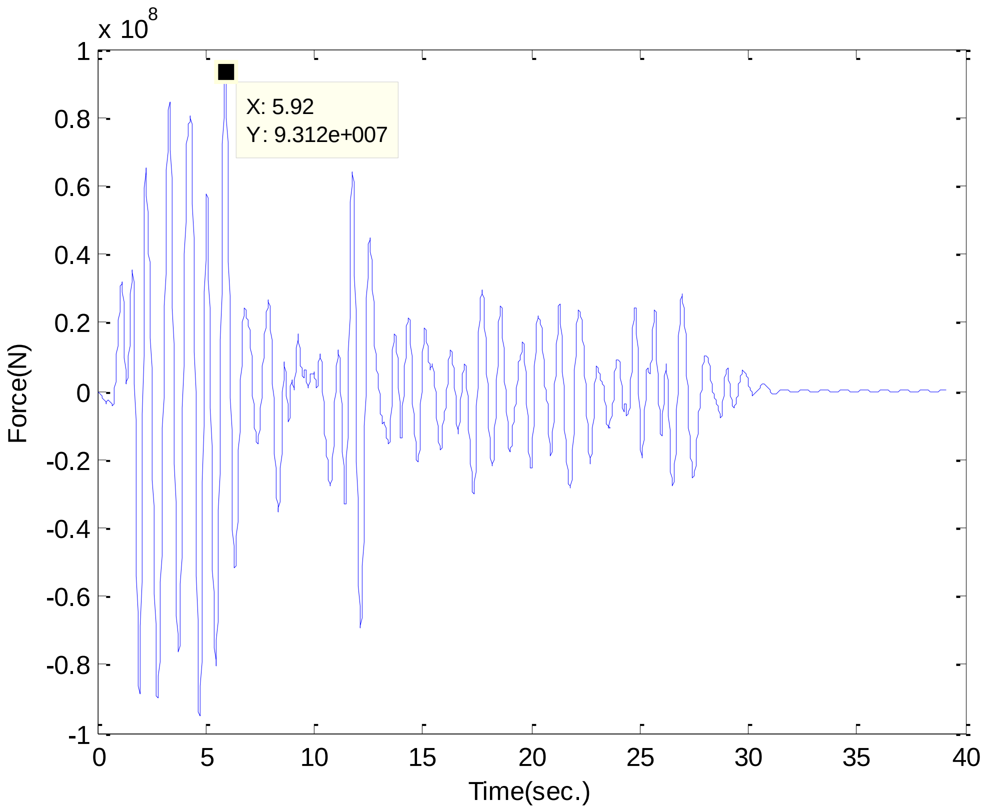


Fig: 8.24 Forces corresponding to 12th degree of freedom at SP-1

Compressive stresses:-

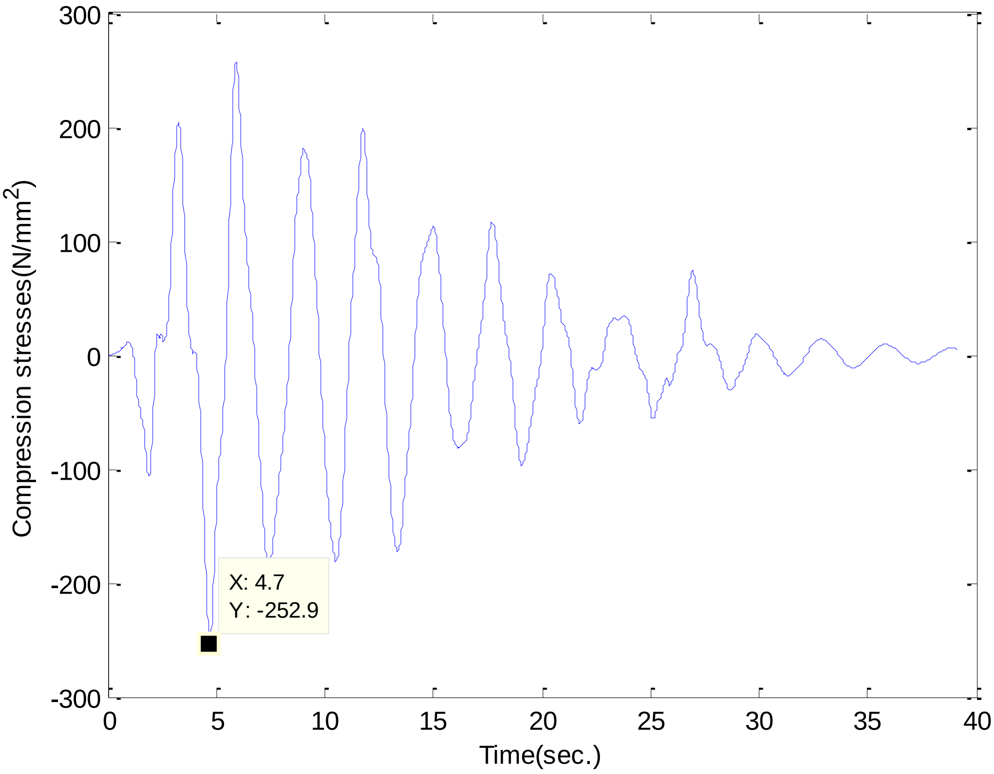


Fig: 8.25 Compressive Stresses in member 5 at joint j

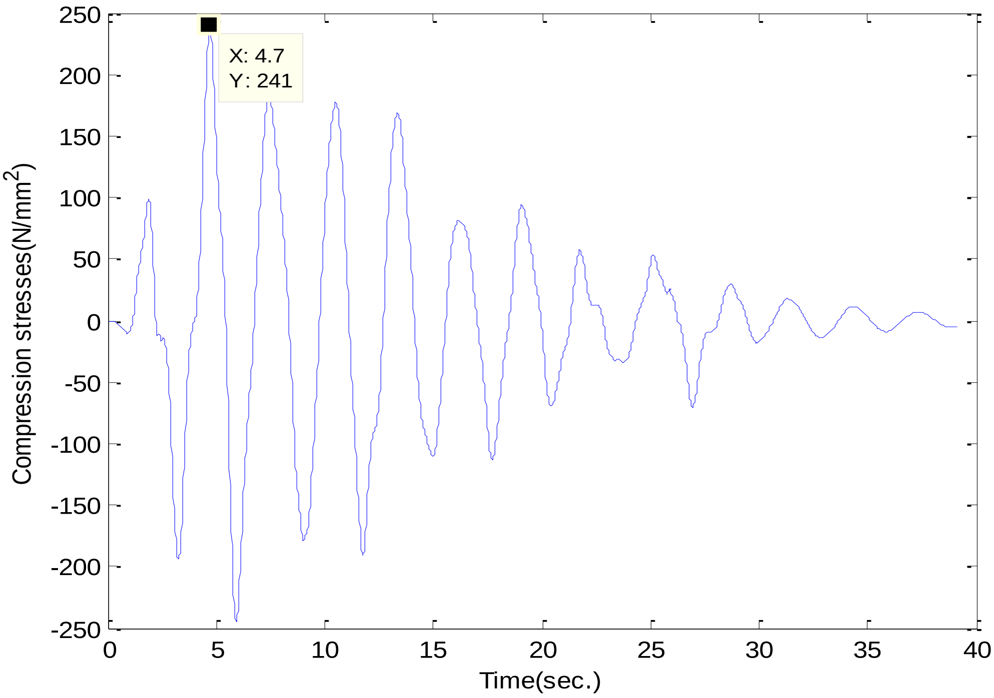


Fig: 8.26 Compressive Stresses in member 5 at joint k

CHAPTER 9. CAPACITY CURVE FOR THE GIVEN PROBLEM STATEMENT

Capacity curve is defined as a plot between Base shear and Displacement. When lateral force is applied to the structure, the structure will displace, due to that deformation stresses will be in picture. Stresses at the support joints will be more so yielding will occur first at support joint.Upto this point the structure will be in Elastic phase. As stresses at support joints reaches 250 N/mm2, the joint will fail and there will be a formation of plastic hinge and the structure will behave as a hinge support structure. Further increasing of lateral load the structure will deform more and when stresses at top joint crosses 250 N/mm2 the whole structure will become collapse.

Deformation =[0.6925 1.385 2.0775 2.77 3.4625 4.155 4.8476 5.5401 6.2326 6.9251 7.6176 8.3101 9.0026 9.6951 10.3876 11.0801 11.7726 12.4651 38.10562 39.19272 40.27982 41.36692 42.45402 43.54112 44.62822 45.71518 46.80228 47.88938 48.97648 50.06358 51.15068 52.23778 53.32488 54.41184 55.49894 56.58604 57.67314 58.76024 59.84734 60.93444];

Baseshear = [1837.8078 3675.6156 5513.4233 7351.2311 9189.0389 11026.8467 12864.6545 14702.4623 16540.27 18378.0778 20215.8856 22053.6934 23891.5012 25729.3089 27567.1167 29404.9245 31242.7323 33080.5401 41623.2566 42072.8732 42522.4899 42972.1065 43421.7232 43871.3399 44320.9565 44770.5732 45220.1898 45669.8065 46119.4231 46569.0398 47018.6565 47468.2731 47917.8898 48367.5064 48817.1231 49266.7397 49716.3564 50165.973 50615.5897 51065.2064];

spectrum deformation =[0.6925 1.385 2.0775 2.77 3.4625 4.155 4.8476 5.5401 6.2326 6.9251 7.6176 8.3101 9.0026 9.6951 10.3876 11.0801 11.7726 12.4651 38.10562 39.19272 40.27982 41.36692 42.45402 43.54112 44.62822 45.71518 46.80228 47.88938 48.97648 50.06358 51.15068 52.23778 53.32488 54.41184 55.49894 56.58604 57.67314 58.76024 59.84734 60.93444];

spectrum acceleration =[113.81037 227.62074 341.430009 455.240379 569.050749 682.861119 796.671489 910.481859 1024.291128 1138.101498 1251.911868 1365.722238 1479.532608 1593.342978 1707.152247 1820.962617 1934.772987 2048.583357 2675.422293 2708.413758 2741.405223 2774.396688 2807.388153 2840.379618 2873.371083 2906.362548 2939.354013 2972.345478 3005.336943 3038.328408 3071.319873 3104.311338 3137.302803 3170.294268 3203.285733 3236.278299 3269.269764 3302.261229 3335.252694 3368.244159];

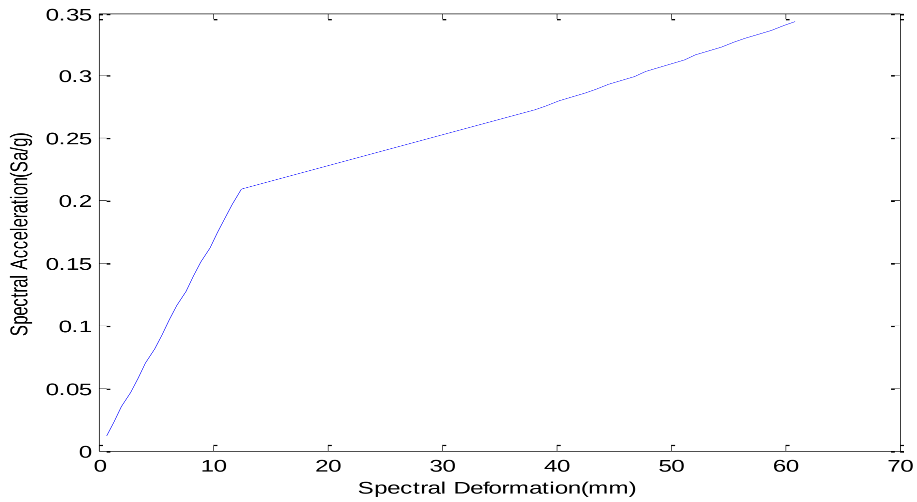


Fig: 9.1 Capacity Spectrum Curve

9.1 CALCULATION OF STRUCTURAL PERFORMANCE LEVELS

As we can see from the above capacity curve, as we apply the incremental base shear to the structure, there will be incremental deformation and the frame will yield first at bottom and the structure will become plastic hinge support structure and the structure goes to nonlinear region. After applying further base shear, there will be more deformation as compare to elastic deformation and the structure will fail, when hinge will form at all the joint.

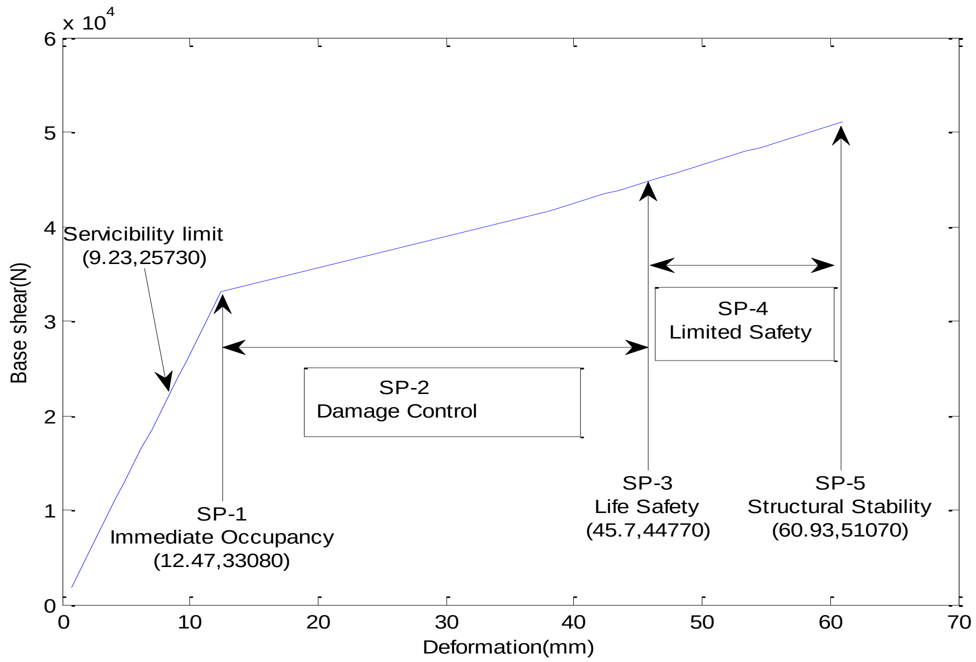


Fig: 9.2 Capacity Curve with structural performance levels.

Serviceability limit = L/ 325 = 3000/325 = 9.23 mm.

At SP-1 Discrete level
Deformation = 12.47 mm.
Base shear = 33080 N

At SP-2 Performance Range
Deformation = 12.47mm – 32.65 mm.
Base shear = 33080N – 44770N

At SP-3 Discrete level
Deformation = 75% of Collapse Prevention Point (Structural Stability) 45.7 mm.
Base shear = 44770 N

At SP-4 Performance Range
Deformation = 60.93mm – 45.7 mm.
Base shear = 44770 N – 51070N

At SP-5 Discrete level
Deformation = 60.93mm.
Base shear = 51070 N

Ductility factor =60.93/12.47= 4.89

CHAPTER 10. PERFORMANCE STUDY AND CONCLUSIONS

The performance of steel frame was investigated using the nonlinear Dynamic analysis.

These are the conclusions drawn from the analyses:

- The nonlinear Dynamic analysis is relatively not a simple but an exact way to explore the non-linear behaviour of buildings.
- The behaviour of steel frame building is adequate as indicated by the intersection of the demand and capacity curves and the distribution of hinges in the beams and the columns. All of the hinges developed in the columns, first at support after that at joints.
- The results obtained in terms of demand, capacity and plastic hinges gave an insight into the real behaviour of structures.

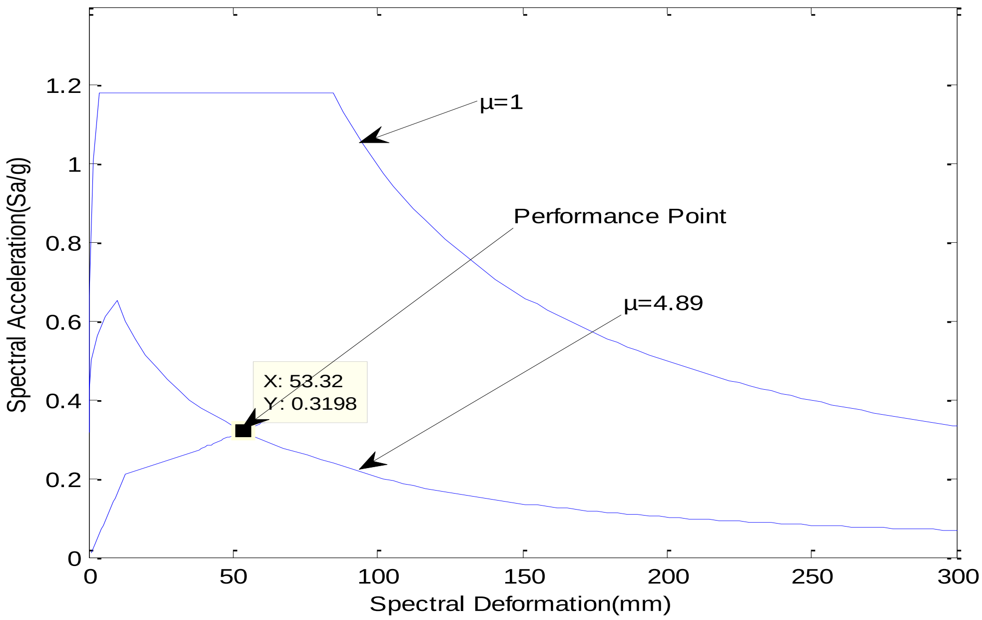


Fig: 10.1 Location of Performance Point

The intersection of the capacity spectrum with the appropriate demand spectrum in the capacity spectrum method (the displacement at the performance point is equivalent to the target displacement in the coefficient method). To have desired performance, every structure has to be designed for this level of forces.

Desired performance with different damping ratios have been shown above.

As can be seen from the above plot, performance point (52.32mm, 0.0.32g) of the structure lies between the life safety and Structural stability performance level (Limited Safety range). This range provide a placeholder for the situation where a retrofit may not meet all the structural requirements of the life safety level, but is better than the level of structural stability. These circumstances include cases when the complete life safety level is not cost effective, or when only some critical structural deficiencies are mitigated.

This performance point represents the maximum structural displacement expected for the demand earthquake ground motion.

If owner want, performance point to come in Damage control range or before SP-1, for that we will have to increase the ductility ratio of the structure. For this increased value of ductility ratio, strength reduction factor will reduce the in-elastic demand curve.

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CHAPTER 12. APPENDIX

12.1 APPENDIX-I: PROGRAM LISTING :- (REF.-32)

PLOTTING OF DEFORMATION, VELOCITY, ACCELERATION, FORCES AND STRESS IN ELASTIC STAGE

```
clear all;
clc;
a=[EL-CENTRO GROUND ACCELERATION DATA];
 for i=1:245
    for j=1:8
        b(8*(i-1)+j)=a(i,j);
    end
end

t=0:0.02:39.18;
% plot(t,b);
 for f=1:40

     g=9811;

factor=(f/100)*b/0.319/11.09;

if f<=18;
    K=[1068813.854    -13868.37674       -588.6204091       -1037862.935
13868.37674  588.6204091  -293.0168761       1.291939754  588.6204091  -
886.7545307    -1.291939754    -588.6204091
-13868.37674      806206.9234  1323.902185  -13868.37674      -18508.17914
0.156011985 -1.291939754      15.28757618  0.156011985 -1.291939754     -
1780.698505    1323.902185
-588.6204091      1323.902185  1044906.858  -588.6204091      0.156011985  -
885.284959  588.6204091  -0.156011985      -293.9238415      588.6204091  -
1323.902185    -1041064.895
-1037862.935     -13868.37674     -588.6204091     1068813.854  13868.37674
588.6204091  -886.7545307     1.291939754  588.6204091  -293.0168761      -
1.291939754    -588.6204091
13868.37674   -18508.17914         0.156011985   13868.37674   806206.9234
1323.902185   1.291939754   -1780.698505         1323.902185   1.291939754
15.28757618  0.156011985
588.6204091  0.156011985  -885.284959  588.6204091  1323.902185  1044906.858
-588.6204091      -1323.902185      -1041064.895      -588.6204091      -
0.156011985    -293.9238415
-293.0168761     -1.291939754     588.6204091  -886.7545307     1.291939754
-588.6204091      1068813.854  13868.37674  -588.6204091      -1037862.935
-13868.37674    588.6204091
1.291939754  15.28757618  -0.156011985     1.291939754  -1780.698505      -
1323.902185      13868.37674  806206.9234  -1323.902185      13868.37674  -
18508.17914    -0.156011985
588.6204091  0.156011985  -293.9238415       588.6204091  1323.902185  -
1041064.895     -588.6204091     -1323.902185     1044906.858  -588.6204091
-0.156011985    -885.284959
-886.7545307     -1.291939754     588.6204091  -293.0168761     1.291939754
-588.6204091     -1037862.935     13868.37674  -588.6204091     1068813.854
-13868.37674    588.6204091
```

```
-1.291939754	-1780.698505	-1323.902185	-1.291939754
15.28757618 -0.156011985	-13868.37674	-18508.17914	-0.156011985
-13868.37674	806206.9234 -1323.902185
-588.6204091	1323.902185 -1041064.895	-588.6204091	0.156011985
-293.9238415	588.6204091 -0.156011985	-885.284959 588.6204091 -
1323.902185	1044906.858];

M=[600	0	0	0	0	0	0	0	0	0	0	0
0	600 0	0	0	0	0	0	0	0	0	0
0	0	600 0	0	0	0	0	0	0	0	0
0	0	0	600 0	0	0	0	0	0	0	0
0	0	0	0	600 0	0	0	0	0	0	0
0	0	0	0	0	600 0	0	0	0	0	0
0	0	0	0	0	0	600 0	0	0	0	0
0	0	0	0	0	0	0	600 0	0	0	0
0	0	0	0	0	0	0	0	600 0	0	0
0	0	0	0	0	0	0	0	0	600 0	0
0	0	0	0	0	0	0	0	0	0	600 0
0	0	0	0	0	0	0	0	0	0	0	600];

T=[0	-0.000326084	2.09532E-05 0	-3.84266E-08	5.90402E-09 0
3.84266E-08 -4.62959E-09	0	0.000326084 -1.01385E-05
0.000208108 0	0.000208108 -4.158E-05	0	-0.000208108	4.158E-05
0	4.158E-05	-0.000208108	0	-4.158E-05
-0.000173569	-0.000257777	0	6.02812E-05	0.000257777	0
1.72075E-08 2.40138E-08 0	-2.77611E-08	-2.40138E-08	0
0	-3.84266E-08	5.90402E-09 0	-0.000326084	2.09532E-05 0
0.000326084 -1.01385E-05	0	3.84266E-08 -4.62959E-09
-4.158E-05	0	0.000208108 0.000208108 0	-0.000208108	-0.000208108
0	4.158E-05	4.158E-05	0	-4.158E-05
6.02812E-05 -0.000257777	0	-0.000173569	0.000257777 0	-
2.77611E-08	2.40138E-08 0	1.72075E-08 -2.40138E-08	0
0	-3.84266E-08	-4.62959E-09	0	-0.000326084	-1.01385E-05
0	0.000326084 2.09532E-05 0	3.84266E-08 5.90402E-09
-4.158E-05	0	-4.158E-05	0.000208108 0	4.158E-05	-0.000208108
0	-0.000208108	4.158E-05	0	0.000208108
1.72075E-08 -2.40138E-08	0	-2.77611E-08	2.40138E-08 0	-
0.000173569	0.000257777 0	6.02812E-05 -0.000257777	0
0	-0.000326084	-1.01385E-05	0	-3.84266E-08	-4.62959E-09
0	3.84266E-08 5.90402E-09 0	0.000326084 2.09532E-05
0.000208108 0	-4.158E-05	-4.158E-05	0	4.158E-05	4.158E-05	0
-0.000208108	-0.000208108	0	0.000208108
-2.77611E-08	-2.40138E-08	0	1.72075E-08	2.40138E-08	0
6.02812E-05 0.000257777 0	-0.000173569	-0.000257777	0];

C=[26993.44913	-349.4830937	-14.83323431	-26154.14597
349.4830937 14.83323431	-7.384025279	0.032556882	14.83323431	-
22.34621417	-0.032556882	-14.83323431
-349.4830937	20375.75447 33.36233505 -349.4830937	-466.4061144
0.003931502 -0.032556882	0.38524692	0.003931502 -0.032556882	-
44.87360232	33.36233505
-14.83323431	33.36233505 26390.99282 -14.83323431	0.003931502 -
22.30918097	14.83323431 -0.003931502	-7.406880806	14.83323431 -
33.36233505	-26234.83535
-26154.14597	-349.4830937	-14.83323431	26993.44913 349.4830937
14.83323431 -22.34621417	0.032556882 14.83323431 -7.384025279	-
0.032556882	-14.83323431
349.4830937	-466.4061144	0.003931502	349.4830937	20375.75447
33.36233505	0.032556882	-44.87360232	33.36233505	0.032556882
0.38524692	0.003931502
```

14.83323431 0.003931502 -22.30918097 14.83323431 33.36233505
26390.99282 -14.83323431 -33.36233505 -26234.83535 -14.83323431
-0.003931502 -7.406880806
-7.384025279 -0.032556882 14.83323431 -22.34621417 0.032556882
-14.83323431 26993.44913 349.4830937 -14.83323431 -26154.14597
-349.4830937 14.83323431
0.032556882 0.38524692 -0.003931502 0.032556882 -44.87360232 -
33.36233505 349.4830937 20375.75447 -33.36233505 349.4830937 -
466.4061144 -0.003931502
14.83323431 0.003931502 -7.406880806 14.83323431 33.36233505 -
26234.83535 -14.83323431 -33.36233505 26390.99282 -14.83323431
-0.003931502 -22.30918097
-22.34621417 -0.032556882 14.83323431 -7.384025279 0.032556882
-14.83323431 -26154.14597 349.4830937 -14.83323431 26993.44913
-349.4830937 14.83323431
-0.032556882 -44.87360232 -33.36233505 -0.032556882
0.38524692 -0.003931502 -349.4830937 -466.4061144 -0.003931502
-349.4830937 20375.75447 -33.36233505
-14.83323431 33.36233505 -26234.83535 -14.83323431 0.003931502
-7.406880806 14.83323431 -0.003931502 -22.30918097 14.83323431
-33.36233505 26390.99282];

Sg1=[1041400 0 0 0 0 0 -1041400 0 0 0 0 0
0 81611.55556 0 0 0 122417333.3 0 -81611.55556 0 0 0
122417333.3
0 0 2356.444444 0 -3534666.667 0 0 0 -2356.444444 0
-3534666.667 0
0 0 0 44240000 0 0 0 0 0 -44240000 0 0
0 0 -3534666.667 0 7069333333 0 0 0 3534666.667 0
3534666667 0
0 122417333.3 0 0 0 2.44835E+11 0 -122417333.3 0 0 0
1.22417E+11
-1041400 0 0 0 0 0 1041400 0 0 0 0 0
0 -81611.55556 0 0 0 -122417333.3 0 81611.55556 0 0
0 -122417333.3
0 0 -2356.444444 0 3534666.667 0 0 0 2356.444444 0
3534666.667 0
0 0 0 -44240000 0 0 0 0 0 44240000 0 0
0 0 -3534666.667 0 3534666667 0 0 0 3534666.667 0
7069333333 0
0 122417333.3 0 0 0 1.22417E+11 0 -122417333.3 0 0 0
2.44835E+11];

Sg2=[2356.444444 0 0 0 3534666.667 0 -2356.444444 0 0
0 3534666.667 0
0 81611.55556 0 -122417333.3 0 0 0 -81611.55556 0 -
122417333.3 0 0
0 0 1041400 0 0 0 0 0 -1041400 0 0 0
0 -122417333.3 0 2.44835E+11 0 0 0 122417333.3 0
1.22417E+11 0 0
3534666.667 0 0 0 7069333333 0 -3534666.667 0 0 0
3534666667 0
0 0 0 0 0 44240000 0 0 0 0 0 -44240000
-2356.444444 0 0 0 -3534666.667 0 2356.444444 0 0 0
-3534666.667 0
0 -81611.55556 0 122417333.3 0 0 0 81611.55556 0
122417333.3 0 0
0 0 -1041400 0 0 0 0 0 1041400 0 0 0
0 -122417333.3 0 1.22417E+11 0 0 0 122417333.3 0
2.44835E+11 0 0

```
3534666.667 0    0    0    3534666667   0    -3534666.667     0    0    0
7069333333  0
0   0   0   0   0   -44240000   0   0   0   0   0   44240000];

Sg3=[1041400    0   0   0   0   0   -1041400    0   0   0   0   0
0   81611.55556 0   0   0   122417333.3 0   -81611.55556    0   0   0
122417333.3
0   0   2356.444444 0   -3534666.667    0   0   0   -2356.444444    0
-3534666.667    0
0   0   0   44240000    0   0   0   0   0   -44240000   0   0
0   0   -3534666.667    0   7069333333  0   0   0   3534666.667 0
3534666667  0
0   122417333.3 0   0   0   2.44835E+11 0   -122417333.3    0   0   0
1.22417E+11
-1041400    0   0   0   0   0   1041400 0   0   0   0   0
0   -81611.55556    0   0   0   -122417333.3    0   81611.55556 0   0
0   -122417333.3
0   0   -2356.444444    0   3534666.667 0   0   0   2356.444444 0
3534666.667 0
0   0   0   -44240000   0   0   0   0   0   44240000    0   0
0   0   -3534666.667    0   3534666667  0   0   0   3534666.667 0
7069333333  0
0   122417333.3 0   0   0   1.22417E+11 0   -122417333.3    0   0   0
2.44835E+11];

Sg4=[2356.444444    0   0   0   3534666.667 0   -2356.444444    0   0
0   3534666.667 0
0   81611.55556 0   -122417333.3    0   0   0   -81611.55556    0   -
122417333.3 0   0
0   0   1041400 0   0   0   0   0   -1041400    0   0   0
0   -122417333.3    0   2.44835E+11 0   0   0   122417333.3 0
1.22417E+11 0   0
3534666.667 0   0   0   7069333333  0   -3534666.667    0   0   0
3534666667  0
0   0   0   0   0   44240000    0   0   0   0   0   -44240000
-2356.444444    0   0   0   -3534666.667    0   2356.444444 0   0   0
-3534666.667    0
0   -81611.55556    0   122417333.3 0   0   0   81611.55556 0
122417333.3 0   0
0   0   -1041400    0   0   0   0   0   1041400 0   0   0
0   -122417333.3    0   1.22417E+11 0   0   0   122417333.3 0
2.44835E+11 0   0
3534666.667 0   0   0   3534666667  0   -3534666.667    0   0   0
7069333333  0
0   0   0   0   0   -44240000   0   0   0   0   0   44240000];

Sg5=[35866.57778    0   0   0   0   -53799866.67    -35866.57778    0
0   0   0   -53799866.67
0   785933.3333 0   0   0   0   -785933.3333    0   0   0   0
0   0   2706.666667 4060000 0   0   0   0   -2706.666667    4060000 0
0
0   0   4060000 8120000000  0   0   0   0   -4060000    4060000000  0
0
0   0   0   0   17680000    0   0   0   0   0   -17680000   0
-53799866.67    0   0   0   0   1.076E+11   53799866.67 0   0   0   0
53799866667
-35866.57778    0   0   0   0   53799866.67 35866.57778 0   0   0   0
53799866.67
0   -785933.3333    0   0   0   0   0   785933.3333 0   0   0   0
```

```
0	0	-2706.666667	-4060000	0	0	0	0	2706.666667	-4060000
0	0
0	0	4060000	4060000000	0	0	0	0	-4060000	8120000000	0
0
0	0	0	0	-17680000	0	0	0	0	0	17680000	0
-53799866.67	0	0	0	0	53799866667	53799866.67	0	0	0	0
1.076E+11];

Sg6=[35866.57778	0	0	0	0	-53799866.67	-35866.57778	0
0	0	0	-53799866.67
0	785933.3333	0	0	0	0	0	-785933.3333	0	0	0	0
0	0	2706.666667	4060000	0	0	0	0	-2706.666667	4060000	0
0
0	0	4060000	8120000000	0	0	0	0	-4060000	4060000000	0
0
0	0	0	0	17680000	0	0	0	0	0	-17680000	0
-53799866.67	0	0	0	0	1.076E+11	53799866.67	0	0	0	0
53799866667
-35866.57778	0	0	0	0	53799866.67	35866.57778	0	0	0	0
53799866.67
0	-785933.3333	0	0	0	0	0	785933.3333	0	0	0	0
0	0	-2706.666667	-4060000	0	0	0	0	2706.666667	-4060000
0	0
0	0	4060000	4060000000	0	0	0	0	-4060000	8120000000	0
0
0	0	0	0	-17680000	0	0	0	0	0	17680000	0
-53799866.67	0	0	0	0	53799866667	53799866.67	0	0	0	0
1.076E+11];

Sg7=[35866.57778	0	0	0	0	-53799866.67	-35866.57778	0
0	0	0	-53799866.67
0	785933.3333	0	0	0	0	0	-785933.3333	0	0	0	0
0	0	2706.666667	4060000	0	0	0	0	-2706.666667	4060000	0
0
0	0	4060000	8120000000	0	0	0	0	-4060000	4060000000	0
0
0	0	0	0	17680000	0	0	0	0	0	-17680000	0
-53799866.67	0	0	0	0	1.076E+11	53799866.67	0	0	0	0
53799866667
-35866.57778	0	0	0	0	53799866.67	35866.57778	0	0	0	0
53799866.67
0	-785933.3333	0	0	0	0	0	785933.3333	0	0	0	0
0	0	-2706.666667	-4060000	0	0	0	0	2706.666667	-4060000
0	0
0	0	4060000	4060000000	0	0	0	0	-4060000	8120000000	0
0
0	0	0	0	-17680000	0	0	0	0	0	17680000	0
-53799866.67	0	0	0	0	53799866667	53799866.67	0	0	0	0
1.076E+11];

Sg8=[35866.57778	0	0	0	0	-53799866.67	-35866.57778	0
0	0	0	-53799866.67
0	785933.3333	0	0	0	0	0	-785933.3333	0	0	0	0
0	0	2706.666667	4060000	0	0	0	0	-2706.666667	4060000	0
0
0	0	4060000	8120000000	0	0	0	0	-4060000	4060000000	0
0
0	0	0	0	17680000	0	0	0	0	0	-17680000	0
-53799866.67	0	0	0	0	1.076E+11	53799866.67	0	0	0	0
53799866667
```

```
-35866.57778     0    0    0    0    53799866.67  35866.57778  0    0    0    0
53799866.67
0    -785933.3333     0    0    0    0    0    785933.3333  0    0    0    0
0    0    -2706.666667     -4060000     0    0    0    0    2706.666667  -4060000
0    0
0    0    4060000  4060000000   0    0    0    0    -4060000     8120000000   0
0
0    0    0    0    -17680000    0    0    0    0    0    17680000     0
-53799866.67     0    0    0    0    53799866667  53799866.67  0    0    0    0
1.076E+11];

else

factor=(f/100)*b/0.319/11.09;
    K=[1048661.841    -7350.551888        -589.1111111        -1040132.182
7350.551888  589.1111111  -294.2475094        0.725873415  589.1111111  -
884.134216 -0.725873415    -589.1111111
-7350.551888      801983.2639  579.4328061  -7350.551888      -14718.67293
0.068653058 -0.725873415      16.11741022  0.068653058  -0.725873415      -
1347.375006    579.4328061
-589.1111111      579.4328061  1042856.446  -589.1111111      0.068653058  -
883.6713431    589.1111111  -0.068653058    -294.5518702    589.1111111  -
579.4328061    -1041097.478
-1040132.182     -7350.551888     -589.1111111     1048661.841  7350.551888
589.1111111  -884.134216  0.725873415  589.1111111  -294.2475094         -
0.725873415    -589.1111111
7350.551888  -14718.67293          0.068653058   7350.551888   801983.2639
579.4328061  0.725873415  -1347.375006          579.4328061  0.725873415
16.11741022 0.068653058
589.1111111  0.068653058  -883.6713431          589.1111111  579.4328061
1042856.446 -589.1111111    -579.4328061    -1041097.478    -589.1111111
-0.068653058    -294.5518702
-294.2475094      -0.725873415      589.1111111  -884.134216  0.725873415  -
589.1111111    1048661.841  7350.551888  -589.1111111    -1040132.182    -
7350.551888    589.1111111
0.725873415  16.11741022  -0.068653058      0.725873415  -1347.375006     -
579.4328061      7350.551888  801983.2639  -579.4328061      7350.551888  -
14718.67293    -0.068653058
589.1111111  0.068653058  -294.5518702        589.1111111  579.4328061  -
1041097.478     -589.1111111     -579.4328061     1042856.446  -589.1111111
-0.068653058    -883.6713431
-884.134216  -0.725873415      589.1111111  -294.2475094      0.725873415  -
589.1111111    -1040132.182    7350.551888  -589.1111111    1048661.841  -
7350.551888    589.1111111
-0.725873415         -1347.375006        -579.4328061        -0.725873415
16.11741022  -0.068653058    -7350.551888    -14718.67293    -0.068653058
-7350.551888    801983.2639  -579.4328061
-589.1111111     579.4328061  -1041097.478     -589.1111111     0.068653058
-294.5518702     589.1111111  -0.068653058     -883.6713431     589.1111111
-579.4328061    1042856.446];

M=[600  0    0    0    0    0    0    0    0    0    0    0
0    600  0    0    0    0    0    0    0    0    0    0
0    0    600  0    0    0    0    0    0    0    0    0
0    0    0    600  0    0    0    0    0    0    0    0
0    0    0    0    600  0    0    0    0    0    0    0
0    0    0    0    0    600  0    0    0    0    0    0
0    0    0    0    0    0    600  0    0    0    0    0
0    0    0    0    0    0    0    600  0    0    0    0
0    0    0    0    0    0    0    0    600  0    0    0
0    0    0    0    0    0    0    0    0    600  0    0
```

```
0	0	0	0	0	0	0	0	0	0	600 0
0	0	0	0	0	0	0	0	0	0	600];

T=[0	-0.000327857	9.24065E-06 0	-3.88456E-08	2.64605E-09 0
3.88456E-08 -2.08524E-09	0	0.000327857 -4.5074E-06
0.000208333 0	0.000208333	-4.16667E-05	0	-0.000208333
4.16667E-05 0	4.16667E-05 -0.000208333	0	-4.16667E-05
-9.62259E-05	-0.000273255	0	3.61809E-05 0.000273255 0
1.14516E-08 2.69842E-08 0	-1.73811E-08	-2.69842E-08	0
0	-3.88456E-08	2.64605E-09 0	-0.000327857	9.24065E-06 0
0.000327857 -4.5074E-06 0	3.88456E-08 -2.08524E-09
-4.16667E-05	0	0.000208333 0.000208333 0	-0.000208333	-
0.000208333	0	4.16667E-05 4.16667E-05 0	-4.16667E-05
3.61809E-05 -0.000273255	0	-9.62259E-05	0.000273255 0	-
1.73811E-08	2.69842E-08 0	1.14516E-08 -2.69842E-08	0
0	-3.88456E-08	-2.08524E-09	0	-0.000327857	-4.5074E-06 0
0.000327857 9.24065E-06 0	3.88456E-08 2.64605E-09
-4.16667E-05	0	-4.16667E-05	0.000208333 0	4.16667E-05 -
0.000208333	0	-0.000208333	4.16667E-05 0	0.000208333
1.14516E-08 -2.69842E-08	0	-1.73811E-08	2.69842E-08 0	-
9.62259E-05	0.000273255 0	3.61809E-05 -0.000273255	0
0	-0.000327857	-4.5074E-06 0	-3.88456E-08	-2.08524E-09	0
3.88456E-08 2.64605E-09 0	0.000327857 9.24065E-06
0.000208333 0	-4.16667E-05	-4.16667E-05	0	4.16667E-05
4.16667E-05 0	-0.000208333	-0.000208333	0	0.000208333
-1.73811E-08	-2.69842E-08	0	1.14516E-08 2.69842E-08 0
3.61809E-05 0.000273255 0	-9.62259E-05	-0.000273255	0];

C=[26485.6184	-185.2339076	-14.8456	-26211.33098	185.2339076
14.8456 -7.415037236	0.01829201	14.8456 -22.28018224	-0.01829201
-14.8456
-185.2339076	20269.31825 14.60170671 -185.2339076	-370.9105579
0.001730057 -0.01829201 0.406158738 0.001730057 -0.01829201 -33.95385015
14.60170671
-14.8456	14.60170671 26339.32245 -14.8456	0.001730057 -22.26851785
14.8456 -0.001730057	-7.42270713 14.8456 -14.60170671	-26235.65645
-26211.33098	-185.2339076	-14.8456	26485.6184	185.2339076
14.8456 -22.28018224	0.01829201	14.8456 -7.415037236	-0.01829201
-14.8456
185.2339076	-370.9105579	0.001730057	185.2339076	20269.31825
14.60170671	0.01829201	-33.95385015	14.60170671	0.01829201
0.406158738 0.001730057
14.8456 0.001730057 -22.26851785	14.8456 14.60170671 26339.32245 -
14.8456	-14.60170671	-26235.65645	-14.8456	-0.001730057	-
7.42270713
-7.415037236	-0.01829201 14.8456 -22.28018224	0.01829201	-14.8456
26485.6184	185.2339076 -14.8456	-26211.33098	-185.2339076
14.8456
0.01829201	0.406158738 -0.001730057	0.01829201	-33.95385015	-
14.60170671	185.2339076 20269.31825 -14.60170671	185.2339076 -
370.9105579	-0.001730057
14.8456 0.001730057 -7.42270713 14.8456 14.60170671 -26235.65645	-
14.8456	-14.60170671	26339.32245 -14.8456	-0.001730057	-
22.26851785
-22.28018224	-0.01829201 14.8456 -7.415037236	0.01829201	-14.8456
-26211.33098	185.2339076 -14.8456	26485.6184	-185.2339076
14.8456
-0.01829201 -33.95385015	-14.60170671	-0.01829201 0.406158738 -
0.001730057	-185.2339076	-370.9105579	-0.001730057	-
185.2339076	20269.31825 -14.60170671
```

```
-14.8456	14.60170671	-26235.65645	-14.8456	0.001730057	-
7.42270713	14.8456	-0.001730057	-22.26851785	14.8456	-14.60170671
26339.32245];

Sg1=[1041400	0	0	0	0	0	-1041400	0	0	0	0	0
0	81611.55556	0	0	0	122417333.3	0	-81611.55556	0	0	0
122417333.3
0	0	2356.444444	0	-3534666.667	0	0	0	-2356.444444	0
-3534666.667	0
0	0	0	44240000	0	0	0	0	0	-44240000	0	0
0	0	-3534666.667	0	7069333333	0	0	0	3534666.667	0
3534666667	0
0	122417333.3	0	0	0	2.44835E+11	0	-122417333.3	0	0	0
1.22417E+11
-1041400	0	0	0	0	0	1041400	0	0	0	0	0
0	-81611.55556	0	0	0	-122417333.3	0	81611.55556	0	0
0	-122417333.3
0	0	-2356.444444	0	3534666.667	0	0	0	2356.444444	0
3534666.667	0
0	0	0	-44240000	0	0	0	0	0	44240000	0	0
0	0	-3534666.667	0	3534666667	0	0	0	3534666.667	0
7069333333	0
0	122417333.3	0	0	0	1.22417E+11	0	-122417333.3	0	0	0
2.44835E+11];

Sg2=[2356.444444	0	0	0	3534666.667	0	-2356.444444	0	0
0	3534666.667	0
0	81611.55556	0	-122417333.3	0	0	0	-81611.55556	0	-
122417333.3	0	0
0	0	1041400	0	0	0	0	0	-1041400	0	0	0
0	-122417333.3	0	2.44835E+11	0	0	0	122417333.3	0
1.22417E+11	0	0
3534666.667	0	0	0	7069333333	0	-3534666.667	0	0	0
3534666667	0
0	0	0	0	0	44240000	0	0	0	0	0	-44240000
-2356.444444	0	0	0	-3534666.667	0	2356.444444	0	0	0
-3534666.667	0
0	-81611.55556	0	122417333.3	0	0	0	81611.55556	0
122417333.3	0	0
0	0	-1041400	0	0	0	0	0	1041400	0	0	0
0	-122417333.3	0	1.22417E+11	0	0	0	122417333.3	0
2.44835E+11	0	0
3534666.667	0	0	0	3534666667	0	-3534666.667	0	0	0
7069333333	0
0	0	0	0	0	-44240000	0	0	0	0	0	44240000];

Sg3=[1041400	0	0	0	0	0	-1041400	0	0	0	0	0
0	81611.55556	0	0	0	122417333.3	0	-81611.55556	0	0	0
122417333.3
0	0	2356.444444	0	-3534666.667	0	0	0	-2356.444444	0
-3534666.667	0
0	0	0	44240000	0	0	0	0	0	-44240000	0	0
0	0	-3534666.667	0	7069333333	0	0	0	3534666.667	0
3534666667	0
0	122417333.3	0	0	0	2.44835E+11	0	-122417333.3	0	0	0
1.22417E+11
-1041400	0	0	0	0	0	1041400	0	0	0	0	0
0	-81611.55556	0	0	0	-122417333.3	0	81611.55556	0	0
0	-122417333.3
```

0 0 -2356.444444 0 3534666.667 0 0 0 2356.444444 0
3534666.667 0
0 0 0 -44240000 0 0 0 0 0 44240000 0 0
0 0 -3534666.667 0 3534666667 0 0 0 3534666.667 0
7069333333 0
0 122417333.3 0 0 0 1.22417E+11 0 -122417333.3 0 0 0
2.44835E+11];

Sg4=[2356.444444 0 0 0 3534666.667 0 -2356.444444 0 0
0 3534666.667 0
0 81611.55556 0 -122417333.3 0 0 0 -81611.55556 0 -
122417333.3 0 0
0 0 1041400 0 0 0 0 0 -1041400 0 0 0
0 -122417333.3 0 2.44835E+11 0 0 0 122417333.3 0
1.22417E+11 0 0
3534666.667 0 0 0 7069333333 0 -3534666.667 0 0 0
3534666667 0
0 0 0 0 0 44240000 0 0 0 0 0 -44240000
-2356.444444 0 0 0 -3534666.667 0 2356.444444 0 0 0
-3534666.667 0
0 -81611.55556 0 122417333.3 0 0 0 81611.55556 0
122417333.3 0 0
0 0 -1041400 0 0 0 0 0 1041400 0 0 0
0 -122417333.3 0 1.22417E+11 0 0 0 122417333.3 0
2.44835E+11 0 0
3534666.667 0 0 0 3534666667 0 -3534666.667 0 0 0
7069333333 0
0 0 0 0 0 -44240000 0 0 0 0 0 44240000];

Sg5=[8966.644444 0 0 0 0 -26899933.33 -20402.88889 0
0 0 0 -26899933.33
0 785933.3333 0 0 0 0 0 -785933.3333 0 0 0 0
0 0 676.6666667 2030000 0 0 0 0 -676.6666667 2030000 0
0
0 0 2030000 6090000000 0 0 0 0 -1767333.333 0 0 0
0 0 0 0 0 0 0 0 0 0 0 0
-26899933.33 0 0 0 0 80699800000 26899933.33 0 0 0 0
0
-8966.644444 0 0 0 0 26899933.33 8966.644444 0 0 0 0
26899933.33
0 -785933.3333 0 0 0 0 0 785933.3333 0 0 0 0
0 0 -676.6666667 -2030000 0 0 0 0 589.1111111 -2030000
0 0
0 0 2030000 0 0 0 0 0 -1767333.333 6090000000 0 0
0 0 0 0 0 0 0 0 0 0 0 0
-26899933.33 0 0 0 0 0 26899933.33 0 0 0 0
80699800000];

Sg6=[8966.644444 0 0 0 0 -26899933.33 -20402.88889 0
0 0 0 -26899933.33
0 785933.3333 0 0 0 0 0 -785933.3333 0 0 0 0
0 0 676.6666667 2030000 0 0 0 0 -676.6666667 2030000 0
0
0 0 2030000 6090000000 0 0 0 0 -1767333.333 0 0 0
0 0 0 0 0 0 0 0 0 0 0 0
-26899933.33 0 0 0 0 80699800000 26899933.33 0 0 0 0
0
-8966.644444 0 0 0 0 26899933.33 8966.644444 0 0 0 0
26899933.33
0 -785933.3333 0 0 0 0 0 785933.3333 0 0 0 0

```
0    0    -676.6666667     -2030000     0    0    0    0    589.1111111 -2030000
0    0
0    0    2030000 0    0    0    0    0    -1767333.333     6090000000   0    0
0    0    0    0    0    0    0    0    0    0    0    0
-26899933.33      0     0     0     0     0     26899933.33  0     0     0     0
80699800000];

Sg7=[8966.644444     0    0    0    0    -26899933.33     -20402.88889     0
0    0    0    -26899933.33
0    785933.3333 0    0    0    0    0    -785933.3333     0    0    0    0
0     0    676.6666667 2030000 0    0    0    0    -676.6666667     2030000 0
0
0    0    2030000 6090000000  0    0    0    0    -1767333.333     0    0    0
0    0    0    0    0    0    0    0    0    0    0    0
-26899933.33     0    0    0    0    80699800000 26899933.33 0    0    0    0
0
-8966.644444     0    0    0    0    26899933.33 8966.644444 0    0    0    0
26899933.33
0    -785933.3333     0    0    0    0    0    785933.3333 0    0    0    0
0    0    -676.6666667     -2030000     0    0    0    0    589.1111111 -2030000
0    0
0    0    2030000 0    0    0    0    0    -1767333.333     6090000000   0    0
0    0    0    0    0    0    0    0    0    0    0    0
-26899933.33      0     0     0     0     0     26899933.33  0     0     0     0
80699800000];

Sg8=[8966.644444     0    0    0    0    -26899933.33     -20402.88889     0
0    0    0    -26899933.33
0    785933.3333 0    0    0    0    0    -785933.3333     0    0    0    0
0     0    676.6666667 2030000 0    0    0    0    -676.6666667     2030000 0
0
0    0    2030000 6090000000  0    0    0    0    -1767333.333     0    0    0
0    0    0    0    0    0    0    0    0    0    0    0
-26899933.33     0    0    0    0    80699800000 26899933.33 0    0    0    0
0
-8966.644444     0    0    0    0    26899933.33 8966.644444 0    0    0    0
26899933.33
0    -785933.3333     0    0    0    0    0    785933.3333 0    0    0    0
0    0    -676.6666667     -2030000     0    0    0    0    589.1111111 -2030000
0    0
0    0    2030000 0    0    0    0    0    -1767333.333     6090000000   0    0
0    0    0    0    0    0    0    0    0    0    0    0
-26899933.33      0     0     0     0     0     26899933.33  0     0     0     0
80699800000];

end

u0=[0;0;0;0;0;0;0;0;0;0;0;0];
u10=[0;0;0;0;0;0;0;0;0;0;0;0];
u=u0;
u1=u10;

f0=-g*factor(1)*M*[1;1;1;1;1;1;1;1;1;1;1;1];

u20=inv(M)*(f0-C*u10-K*u0);

u2=u20;

dt=0.02;
```

```
th=1.4;
to=dt*th;

A=(6/to)*M+3*C;
B=3*M+to/2*C;

for p=1:1958

    f0=-M*g*factor(p)*[1;1;1;1;1;1;1;1;1;1;1;1];
    f1=-M*g*factor(p+1)*[1;1;1;1;1;1;1;1;1;1;1;1];
    f2=-M*g*factor(p+2)*[1;1;1;1;1;1;1;1;1;1;1;1];
     for v=1:12
        u(v,p)=u0(v);
        u1(v,p)=u10(v);
        u2(v,p)=u20(v);
     end
 Keff=K+3/to*C+6/to^2*M;
     df=f1+(f2-f1)*(th-1)-f0;

    dfs=df+A*u10+B*u20;
    Du=[0;0;0;0;0;0;0;0;0;0;0;0];

    for m=1:7
     Duu=inv(Keff)*dfs;
     Du=Du+Duu;
     dfj=(Keff-K)*Du;
     dfs=dfs-dfj;
    end

    Du2=6/to^2*Du-6/to*u10-3*u20;

    du2=Du2/th;

    du1=u20*dt+0.5*du2*dt;
    du=u10*dt+0.5*u20*dt^2+1/6*du2*dt^2;

    u0=u0+du;
    u10=u10+du1;

    u20=inv(M)*(f1-C*u10-K*u0);
end
 for i=1:1958
     deformation(f,i)=u(3,i);
 end
 for i=1:1958
     acceleration(f,i)=u2(3,i);
 end
    baseshear=K*u;
  for i=1:1958
  linearbaseshear(f,i)=baseshear(3,i);
  end

  t=0:0.02:39.14;
for n=1:12                                                              % to

   for k=1:1958
     temp(k)=u(n,k);
   end
```

```
% figure,plot(t,temp);
% xlabel('Time(sec.)')
% ylabel('Displacement(mm)')
c=max(temp);
end

depddis=T*u;
    for q=1:12

        for j=1:1958
            s(j)=depddis(q,j);
        end
%         figure,plot(t,s);
%         xlabel('Time(sec.)')
%         ylabel('Rotation(rad/s)')
    end

  for r=1:1958
           A1(1,r)=u(1,r);
           B1(1,r)=u(2,r);
           C1(1,r)=u(3,r);
           D1(1,r)=depddis(1,r);
           E1(1,r)=depddis(2,r);
           F1(1,r)=depddis(3,r);
           G1(1,r)=u(4,r);
           H1(1,r)=u(5,r);
           I1(1,r)=u(6,r);
           J1(1,r)=depddis(4,r);
           K1(1,r)=depddis(5,r);
           L1(1,r)=depddis(6,r);
  end
     Ds1=[A1;B1;C1;D1;E1;F1;G1;H1;I1;J1;K1;L1];

       Am1=Sg1*Ds1;

%   for i=1:12
%
%         for j=1:1958
%             A(j)=Am1(i,j);
%         end
%         figure,plot(t,A);
%         xlabel('Time(sec.)')
%         ylabel('Forces')
%    end



  for s=1:1958
           A2(1,s)=u(7,s);
           B2(1,s)=u(8,s);
           C2(1,s)=u(9,s);
           D2(1,s)=depddis(7,s);
           E2(1,s)=depddis(8,s);
           F2(1,s)=depddis(9,s);
           G2(1,s)=u(4,s);
           H2(1,s)=u(5,s);
           I2(1,s)=u(6,s);
           J2(1,s)=depddis(4,s);
```

```
            K2(1,s)=depddis(5,s);
            L2(1,s)=depddis(6,s);
   end
     Ds2=[A2;B2;C2;D2;E2;F2;G2;H2;I2;J2;K2;L2];

        Am2=Sg2*Ds2;

%     for i=1:12
%
%          for j=1:1958
%              B(j)=Am2(i,j);
%          end
%          figure,plot(t,B);
%          xlabel('Time(sec.)')
%          ylabel('Forces')
%      end


    for i=1:1958
            A3(1,i)=u(10,i);
            B3(1,i)=u(11,i);
            C3(1,i)=u(12,i);
            D3(1,i)=depddis(10,i);
            E3(1,i)=depddis(11,i);
            F3(1,i)=depddis(12,i);
            G3(1,i)=u(7,i);
            H3(1,i)=u(8,i);
            I3(1,i)=u(9,i);
            J3(1,i)=depddis(7,i);
            K3(1,i)=depddis(8,i);
            L3(1,i)=depddis(9,i);
    end
     Ds3=[A3;B3;C3;D3;E3;F3;G3;H3;I3;J3;K3;L3];

        Am3=Sg3*Ds3;

%        for i=1:12
%
%          for j=1:1958
%              D(j)=Am3(i,j);
%          end
%          figure,plot(t,D);
%          xlabel('Time(sec.)')
%          ylabel('Forces')
%        end


       for i=1:1958
            A4(1,i)=u(10,i);
            B4(1,i)=u(11,i);
            C4(1,i)=u(12,i);
            D4(1,i)=depddis(10,i);
            E4(1,i)=depddis(11,i);
            F4(1,i)=depddis(12,i);
            G4(1,i)=u(1,i);
            H4(1,i)=u(2,i);
            I4(1,i)=u(3,i);
            J4(1,i)=depddis(1,i);
            K4(1,i)=depddis(2,i);
```

```
          L4(1,i)=depddis(3,i);
      end
    Ds4=[A4;B4;C4;D4;E4;F4;G4;H4;I4;J4;K4;L4];

       Am4=Sg4*Ds4;


%     for i=1:12
%
%          for j=1:1958
%              E(j)=Am4(i,j);
%          end
%          figure,plot(t,E);
%          xlabel('Time(sec.)')
%          ylabel('Forces')
%      end

 for i=1:1958
          A5(1,i)=[0];
          B5(1,i)=[0];
          C5(1,i)=[0];
          D5(1,i)=[0];
          E5(1,i)=[0];
          F5(1,i)=[0];
          G5(1,i)=u(1,i);
          H5(1,i)=u(2,i);
          I5(1,i)=u(3,i);
          J5(1,i)=depddis(1,i);
          K5(1,i)=depddis(2,i);
          L5(1,i)=depddis(3,i);
 end
    Ds5=[A5;B5;C5;D5;E5;F5;G5;H5;I5;J5;K5;L5];

       Am5=Sg5*Ds5;

%     for i=1:12
%
%          for j=1:1958
%              F(j)=Am5(i,j);
%          end
%          figure,plot(t,F);
%          xlabel('Time(sec.)')
%          ylabel('Forces')
%     end
%
   for i=1:1958
          A6(1,i)=[0];
          B6(1,i)=[0];
          C6(1,i)=[0];
          D6(1,i)=[0];
          E6(1,i)=[0];
          F6(1,i)=[0];
          G6(1,i)=u(4,i);
          H6(1,i)=u(5,i);
          I6(1,i)=u(6,i);
          J6(1,i)=depddis(4,i);
          K6(1,i)=depddis(5,i);
          L6(1,i)=depddis(6,i);
   end
    Ds6=[A6;B6;C6;D6;E6;F6;G6;H6;I6;J6;K6;L6];
```

```
      Am6=Sg6*Ds6;


%   for i=1:12
%
%         for j=1:1958
%             G(j)=Am6(i,j);
%         end
%         figure,plot(t,G);
%         xlabel('Time(sec.)')
%         ylabel('Forces')
%   end
%
  for i=1:1958
          A7(1,i)=[0];
          B7(1,i)=[0];
          C7(1,i)=[0];
          D7(1,i)=[0];
          E7(1,i)=[0];
          F7(1,i)=[0];
          G7(1,i)=u(7,i);
          H7(1,i)=u(8,i);
          I7(1,i)=u(9,i);
          J7(1,i)=depddis(7,i);
          K7(1,i)=depddis(8,i);
          L7(1,i)=depddis(9,i);
  end
    Ds7=[A7;B7;C7;D7;E7;F7;G7;H7;I7;J7;K7;L7];

      Am7=Sg7*Ds7;


%    for i=1:12
%
%         for j=1:1958
%             H(j)=Am7(i,j);
%         end
%         figure,plot(t,H);
%         xlabel('Time(sec.)')
%         ylabel('Forces')
%     end

   for i=1:1958
          A8(1,i)=[0];
          B8(1,i)=[0];
          C8(1,i)=[0];
          D8(1,i)=[0];
          E8(1,i)=[0];
          F8(1,i)=[0];
          G8(1,i)=u(10,i);
          H8(1,i)=u(11,i);
          I8(1,i)=u(12,i);
          J8(1,i)=depddis(10,i);
          K8(1,i)=depddis(11,i);
          L8(1,i)=depddis(12,i);
   end
    Ds8=[A8;B8;C8;D8;E8;F8;G8;H8;I8;J8;K8;L8];
```

```
       Am8=Sg8*Ds8;

%   for i=1:12
%
%         for j=1:1958
%             I(j)=Am8(i,j);
%         end
%         figure,plot(t,I);
%         xlabel('Time(sec.)')
%         ylabel('Forces')
%   end

%                                  STRESSES FOR BEAMS


Zy=252476.1905;
Zz=3060433.333;
Zyc=243600;
Zzc=1793328.889;
Ac=11789;

for i=1:1958
    Comsigma1(f,i)= Am1(5,i)/Zy+Am1(6,i)/Zz;
      Tensigma1(f,i)= -Am1(5,i)/Zy-Am1(6,i)/Zz;
end
%       figure,plot(t,Comsigma1);
%         xlabel('Time(sec.)')
%         ylabel('Compression stresses(N/mm^2)')
%         maxComsigma1=max(Comsigma1);
%       figure,plot(t,Tensigma1);
%         xlabel('Time(sec.)')
%          ylabel('Tension stresses(N/mm^2)')
%          maxTensigma1=max(Tensigma1);
for i=1:1958
    Comsigma2(f,i)=Am2(4,i)/Zy+Am2(5,i)/Zz;
      Tensigma2(f,i)=-Am2(4,i)/Zy-Am2(5,i)/Zz;
end
%        figure,plot(t,Comsigma2);
%         xlabel('Time(sec.)')
%         ylabel('Compression stresses(N/mm^2)')
%         maxComsigma2=max(Comsigma2);
%       figure,plot(t,Tensigma2);
%         xlabel('Time(sec.)')
%          ylabel('Tension stresses(N/mm^2)')
%          maxTensigma2=max(Tensigma2);

for i=1:1958
    Comsigma3(f,i)=Am3(5,i)/Zy+Am3(6,i)/Zz;
      Tensigma3(f,i)=-Am3(5,i)/Zy-Am3(6,i)/Zz;
end
%        figure,plot(t,Comsigma3);
%         xlabel('Time(sec.)')
%         ylabel('Compression stresses(N/mm^2)')
%         maxComsigma3=max(Comsigma3);
%       figure,plot(t,Tensigma3);
%         xlabel('Time(sec.)')
%          ylabel('Tension stresses(N/mm^2)')
%          maxTensigma3=max(Tensigma3);
for i=1:1958
    Comsigma4(f,i)=Am4(4,i)/Zy+Am4(5,i)/Zz;
```

```
      Tensigma4(f,i)=-Am4(4,i)/Zy-Am4(5,i)/Zz;
end
%        figure,plot(t,Comsigma4);
%         xlabel('Time(sec.)')
%         ylabel('Compression stresses(N/mm^2)')
%         maxComsigma4=max(Comsigma4);
%       figure,plot(t,Tensigma4);
%         xlabel('Time(sec.)')
%          ylabel('Tension stresses(N/mm^2)')
%           maxTensigma4=max(Tensigma4);

% %                          STRESSES FOR COLUMNS_

for i=1:1958
    Comsigma5j(f,i)=Am5(2,i)/Ac+Am5(4,i)/Zyc+Am5(6,i)/Zzc;
    Tensigma5j(f,i)=Am5(2,i)/Ac-Am5(4,i)/Zyc-Am5(6,i)/Zzc;
    Comsigma5k(f,i)=Am5(8,i)/Ac-Am5(10,i)/Zyc-Am5(12,i)/Zzc;
    Tensigma5k(f,i)=Am5(8,i)/Ac+Am5(10,i)/Zyc+Am5(12,i)/Zzc;
end
%        figure,plot(t,Comsigma5j);
%         xlabel('Time(sec.)')
%         ylabel('Compression stresses(N/mm^2)')
%         maxComsigma5j=max(Comsigma5j);
%       figure,plot(t,Tensigma5j);
%         xlabel('Time(sec.)')
%          ylabel('Tension stresses(N/mm^2)')
%          maxTensigma5j=max(Tensigma5j);
%       figure,plot(t,Comsigma5k);
%         xlabel('Time(sec.)')
%         ylabel('Compression stresses(N/mm^2)')
%         maxComsigma5k=max(Comsigma5k);
%       figure,plot(t,Tensigma5k);
%         xlabel('Time(sec.)')
%          ylabel('Tension stresses(N/mm^2)')
%          maxTensigma5k=max(Tensigma5k);

for i=1:1958
    Comsigma6j(f,i)=Am6(2,i)/Ac+Am6(4,i)/Zyc+Am6(6,i)/Zzc;
    Tensigma6j(f,i)=Am6(2,i)/Ac-Am6(4,i)/Zyc-Am6(6,i)/Zzc;
    Comsigma6k(f,i)=Am6(8,i)/Ac-Am6(10,i)/Zyc-Am6(12,i)/Zzc;
    Tensigma6k(f,i)=Am6(8,i)/Ac+Am6(10,i)/Zyc+Am6(12,i)/Zzc;
end

%       figure,plot(t,Comsigma6j);
%         xlabel('Time(sec.)')
%         ylabel('Compression stresses(N/mm^2)')
%         maxComsigma6j=max(Comsigma6j);
%       figure,plot(t,Tensigma6j);
%         xlabel('Time(sec.)')
%          ylabel('Tension stresses(N/mm^2)')
%          maxTensigma6j=max(Tensigma6j);
%       figure,plot(t,Comsigma6k);
%         xlabel('Time(sec.)')
%         ylabel('Compression stresses(N/mm^2)')
%         maxComsigma6k=max(Comsigma6k);
%       figure,plot(t,Tensigma6k);
%         xlabel('Time(sec.)')
%          ylabel('Tension stresses(N/mm^2)')
%          maxTensigma6k=max(Tensigma6k);
```

```
for i=1:1958
    Comsigma7j(f,i)=Am7(2,i)/Ac+Am7(4,i)/Zyc+Am7(6,i)/Zzc;
    Tensigma7j(f,i)=Am7(2,i)/Ac-Am7(4,i)/Zyc-Am7(6,i)/Zzc;
    Comsigma7k(f,i)=Am7(8,i)/Ac-Am7(10,i)/Zyc-Am7(12,i)/Zzc;
    Tensigma7k(f,i)=Am7(8,i)/Ac+Am7(10,i)/Zyc+Am7(12,i)/Zzc;
end
%       figure,plot(t,Comsigma7j);
%         xlabel('Time(sec.)')
%         ylabel('Compression stresses(N/mm^2)')
%         maxComsigma7j=max(Comsigma7j);
%       figure,plot(t,Tensigma7j);
%         xlabel('Time(sec.)')
%          ylabel('Tension stresses(N/mm^2)')
%          maxTensigma7j=max(Tensigma7j);
%       figure,plot(t,Comsigma7k);
%         xlabel('Time(sec.)')
%         ylabel('Compression stresses(N/mm^2)')
%         maxComsigma7k=max(Comsigma7k);
%       figure,plot(t,Tensigma7k);
%         xlabel('Time(sec.)')
%          ylabel('Tension stresses(N/mm^2)')
%          maxTensigma7k=max(Tensigma7k);

for i=1:1958
    Comsigma8j(f,i)=Am8(2,i)/Ac+Am8(4,i)/Zyc+Am8(6,i)/Zzc;
    Tensigma8j(f,i)=Am8(2,i)/Ac-Am8(4,i)/Zyc-Am8(6,i)/Zzc;
    Comsigma8k(f,i)=Am8(8,i)/Ac-Am8(10,i)/Zyc-Am8(12,i)/Zzc;
    Tensigma8k(f,i)=Am8(8,i)/Ac+Am8(10,i)/Zyc+Am8(12,i)/Zzc;
end

%       figure,plot(t,Comsigma8j);
%         xlabel('Time(sec.)')
%         ylabel('Compression stresses(N/mm^2)')
%         maxComsigma8j=max(Comsigma8j);
%       figure,plot(t,Tensigma8j);
%         xlabel('Time(sec.)')
%          ylabel('Tension stresses(N/mm^2)')
%           maxTensigma8j=max(Tensigma8j);
%       figure,plot(t,Comsigma8k);
%         xlabel('Time(sec.)')
%         ylabel('Compression stresses(N/mm^2)')
%         maxComsigma8k=max(Comsigma8k);
%       figure,plot(t,Tensigma8k);
%         xlabel('Time(sec.)')
%          ylabel('Tension stresses(N/mm^2)')
%          maxTensigma8k=max(Tensigma8k);


 end
```

PROGRAM OF CAPACITY CURVE

```
clear all
clc

deformation=[0.6925  1.385     2.0775   2.77      3.4625   4.155     4.8476
5.5401   6.2326   6.9251   7.6176   8.3101   9.0026   9.6951   10.3876 11.0801
11.7726 12.4651 38.10562     39.19272     40.27982     41.36692     42.45402
43.54112      44.62822      45.71518      46.80228      47.88938      48.97648
50.06358      51.15068      52.23778      53.32488      54.41184      55.49894
56.58604     57.67314     58.76024     59.84734     60.93444];

baseshear2=[1837.8078    3675.6156    5513.4233    7351.2311    9189.0389
11026.8467   12864.6545   14702.4623   16540.27     18378.0778   20215.8856
22053.6934   23891.5012   25729.3089   27567.1167   29404.9245   31242.7323
33080.5401   41623.2566   42072.8732   42522.4899   42972.1065   43421.7232
43871.3399   44320.9565   44770.5732   45220.1898   45669.8065   46119.4231
46569.0398   47018.6565   47468.2731   47917.8898   48367.5064   48817.1231
49266.7397   49716.3564   50165.973    50615.5897   51065.2064];

figure,plot(defor2,base2);
xlabel('Deformation(mm)')
ylabel('Base Shear(N)')
xlim([0 50]);
ylim([0 60000]);
```

PROGRAM OF CAPACITY SPECTRUM CURVE

```
deformation=[0.6925  1.385     2.0775   2.77      3.4625   4.155     4.8476
5.5401   6.2326   6.9251   7.6176   8.3101   9.0026   9.6951   10.3876 11.0801
11.7726 12.4651 38.10562     39.19272     40.27982     41.36692     42.45402
43.54112      44.62822      45.71518      46.80228      47.88938      48.97648
50.06358      51.15068      52.23778      53.32488      54.41184      55.49894
56.58604     57.67314     58.76024     59.84734     60.93444];

PeakPseudoAcceleration=[113.81037  227.62074      341.430009    455.240379
569.050749   682.861119   796.671489   910.481859   1024.291128 1138.101498
1251.911868 1365.722238 1479.532608 1593.342978 1707.152247 1820.962617
1934.772987 2048.583357 2675.422293 2708.413758 2741.405223 2774.396688
2807.388153 2840.379618 2873.371083 2906.362548 2939.354013 2972.345478
3005.336943 3038.328408 3071.319873 3104.311338 3137.302803 3170.294268
3203.285733     3236.278299     3269.269764     3302.261229     3335.252694
3368.244159];
 figure,plot(Deformation,PeakpseudoAcceleration);
 xlabel('Deformation(mm))')
 ylabel('Peak Pseudo-Acceleration(mm/sec^2)')
 title('Capacity Spectrum Curve')
```

PROGROM OF DEFORMATION RESPONSE SPECTRUM,VELOCITY RESPONSE SPECTRUM, ACCELERATION RESPONSE SPECTRUM AND SPECTRAL ACCELERATION- SPECTRAL DEFORMATION .

```
clear all;
clc;

a=[EL-CENTRO GROUND ACCELERATION DATA];


for i=1:245
    for j=1:8
        b(8*(i-1)+j)=a(i,j);
    end
end

% t=0:0.02:39.18;
% plot(t,b);


     g=9811;
     c=-g*b;


  for n=1:60


     u0=0;
     u10=0;
     dt=0.02;
     th=1.4;
     to=th*dt;
     xi=0.02;

     T=n/20;

     omega=(2*pi)/T;
     a1=6/to;
     a2=6*xi*omega;
     a3=(to*xi*omega+3);
     a4=6/to^2;
     ug20=c(1);
     u20=ug20-(2*omega *xi)*u10-omega^2*u0;

     Keff=(omega^2+a4+a2/to);

       for i=1:1958
          ug20=c(i);
          ug21=c(i+1);
          ug22=c(i+2);
          u(i)=u0;
          u1(i)=u10;
          u2(i)=u20;
```

```
          dug2f=ug21+(ug22-ug21)*(th-1)-ug20;

          dug2s=dug2f+(a1+a2)*u10+a3*u20;
          Du=inv(Keff)*dug2s;
          Du2=a4*Du-a1*u10-3*u20;

          du2=Du2/th;

          du1=u20*dt+0.5*du2*dt;
          du=u10*dt+0.5*u20*(dt^2)+(1/6)*du2*(dt^2);

          u0=u0+du;
          u10=u10+du1;

          u20=ug21-(2*omega *xi)*u10-omega^2*u0;

       end
%    t=0:0.02:39.14;
%    figure,plot(t,u);
%    maxaccel(n)=max(u);
          mindefor=min(u);
          minvelocity=min(u1);
          minaccel=min(u2);
          absolutedefor(n)=abs(mindefor);
          absolutevelocity(n)=abs(minvelocity);
          absoluteaccel(n)=abs(minaccel);



 end

 for n=1:59

    T=(n+1)/20;

    velocityResponse(n)=absolutevelocity(n+1);
    accelResponse(n)=absoluteaccel(n+1);

    deformationResponse(n)=absolutedefor(n+1);
    PseudovelocityResponse(n)=2*pi*inv(T)*absolutedefor(n+1);
    PseudoaccelResponse(n)=(2*pi*inv(T))^2*absolutedefor(n+1);

 end
  for n=1:59

    T1(n)=n/20;
  end

%   figure,plot(T1,velocityResponse);
%  xlabel('T(sec.)')
%  ylabel('velocityResponse(mm/sec)')
%  title('velocity Response Spectrum')
%
%  figure,plot(T1,accelResponse);
%  xlabel('T(sec.)')
%  ylabel('accelResponse(mm/sec^2)')
%  title('accel Response Spectrum')
%
```

```
figure,plot(T1,deformationResponse);
xlabel('T(sec.)')
ylabel('Peak Deformation(mm)')


figure,plot(T1,PseudovelocityResponse);
xlabel('T(sec.)')
ylabel('Peak Pseudo-Velocity(mm/sec)')

figure,plot(T1,PseudoaccelResponse/g);
xlabel('T(sec.)')
ylabel('Peak Pseudo-Acceleration(Sa/g)')


figure,plot(deformationResponse,PseudoaccelResponse);
xlabel('Deformation(mm))')
ylabel('Peak Pseudo-Acceleration(mm/sec^2)')
```

PROGRAM OF ACTUAL DEMAND CURVE

```
clear all;
clc;

a=[EL-CENTRO GROUND ACCELERATION DATA];

for i=1:245
    for j=1:8
        b(8*(i-1)+j)=a(i,j);
    end
end

 t=0:0.02:39.18;
 plot(t,b);

     m=2400;
     g=9811;
     c=-g*b;

 Mu=[1 1.5 2 3 4.89];

 for r=1:5
     mu=Mu(r);


   for n=1:60

      if(n<=3)

         Rmu=1;

        elseif(3>n<=12)

         Rmu=sqrt(2*mu-1);
```

```
    elseif(n>12)

     Rmu=mu;

  end

 u0=0;
 u10=0;
 dt=0.02;
 th=1.4;
 to=th*dt;
 xi=0.02;

 T=n/20;

 omega=(2*pi)/T;
 a1=6/to;
 a2=6*xi*omega;
 a3=(to*xi*omega+3);
 a4=6/to^2;
 ug20=c(1);
 u20=ug20-(2*omega *xi)*u10-omega^2*u0;

 Keff=(omega^2+a4+a2/to);

   for i=1:1958
      ug20=c(i);
      ug21=c(i+1);
      ug22=c(i+2);
      u(i)=u0;
      u1(i)=u10;
      u2(i)=u20;

      dug2f=ug21+(ug22-ug21)*(th-1)-ug20;

      dug2s=dug2f+(a1+a2)*u10+a3*u20;
      Du=inv(Keff)*dug2s;
      Du2=a4*Du-a1*u10-3*u20;

      du2=Du2/th;

      du1=u20*dt+0.5*du2*dt;
      du=u10*dt+0.5*u20*(dt^2)+(1/6)*du2*(dt^2);

      u0=u0+du;
      u10=u10+du1;

      u20=ug21-(2*omega *xi)*u10-omega^2*u0;

   end
t=0:0.02:39.14;
figure,plot(t,u);
maxaccel(n)=max(u);
      mindefor=min(u);
      minvelocity=min(u1);
      minaccel=min(u2);
      absolutedefor(n)=abs(mindefor);
      absolutevelocity(n)=abs(minvelocity);
```

```
        absoluteaccel(n)=abs(minaccel);


  end


   for n=1:59

       T1=(n+1)/20;

  velocityResponse(n)=absolutevelocity(n+1);
  accelResponse(n)=absoluteaccel(n+1);


   deformationResponse(n)=(mu/Rmu)*absolutedefor(n+1);

   pseudoaccelResponse(n)=(2*pi/T1)^2*(1/Rmu)*deformationResponse(n);
   end

    for n=1:59
        T1(n)=(n+1)/20;
    end



 plot(T1,deformationResponse);
 xlabel('T(sec.)')
 ylabel('Peak Deformation(mm)')
 title('Deformation Response Spectrum')
 hold on

 plot(T1,pseudoaccelResponse);
 xlabel('T(sec.)')
 ylabel('Peak Pseudo-Acceleration(mm/sec^2)')
 title('Pseudo-Acceleration Response Spectrum')
 hold on


plot(deformationResponse,PseudoaccelResponse);
xlabel('Deformation(mm))')
ylabel('Peak Pseudo-Acceleration(mm/sec^2)')
title('Demand Curve')
hold on
end
```

PROGRAM OF DEMAND CURVE

```
clear all
clc

 g=9811;

Tc=0.6;

  for n=1:121
    T=(n-1)/40;

   if T<=0.125;
       Sag(n)=(1+21.6*T);
       Sd(n)=(T/(2*pi))^2*Sag(n);
   elseif T<=Tc;
       Sag(n)=3.7;
       Sd(n)=(T/(2*pi))^2*Sag(n);
   else
        Sag(n)=(2.22/T);
        Sd(n)=(T/(2*pi))^2*Sag(n);
   end

 end
T1=0:.025:3;

 figure,plot(T1,Sd*g);
 xlabel('Period(sec))')
 ylabel('Deformation(mm)')



 figure,plot(T1,Sag);
 xlabel('Period(sec))')
 ylabel('Spectral Acceleration Coefficient(Sa/g)')

 xlim([0 3]);
 ylim([0 4.5]);
```

DEMAND CURVE FOR VARIOUS VALUES OF μ AND PERFORMANCE POINT

```
clear all
clc
 factor=0.319;
 g=9811;


Tc=0.6;
%  Mu=[1 1.5 2 3 4.89];
 Mu=[1 4.89];

   for r=1:2
    mu=Mu(r);
  for n=1:121
    T=(n-1)/40;
```

```
   if T<=0.125;
       Sag(n)=(1/((mu-1)*(T/Tc)+1))*(1+21.6*T)*factor*g;
       Sd(n)=mu*(T/(2*pi))^2*Sag(n);
   elseif T<=Tc;
       Sag(n)=(1/((mu-1)*(T/Tc)+1))*3.7*factor*g;
       Sd(n)=mu*(T/(2*pi))^2*Sag(n);
   else
        Sag(n)=(1/mu)*(2.22/T)*factor*g;
        Sd(n)=mu*(T/(2*pi))^2*Sag(n);
   end

 end
T1=0:.025:3;

% plot(T1,Sag/g);
%  xlabel('Period(sec))')
%  ylabel('Spectral Acceleration Coefficient(Sa/g)')
%
%  xlim([0 3]);
%  ylim([0 1.5]);
%   xlim([0 1]);
%   ylim([0 1]);
%   hold on

 plot(Sd/1.25,Sag/g);
 xlabel('Spectral Deformation(mm)')
 ylabel('Spectral Acceleration(Sa/g)')

 xlim([0 300]);
 ylim([0 1.4]);
  hold on

    end
  hold on
deformation=[0.6925 1.385   2.0775  2.77    3.4625  4.155   4.8476
5.5401  6.2326  6.9251  7.6176  8.3101  9.0026  9.6951  10.3876 11.0801
11.7726 12.4651 38.10562    39.19272    40.27982    41.36692    42.45402
43.54112    44.62822    45.71518    46.80228    47.88938    48.97648
50.06358    51.15068    52.23778    53.32488    54.41184    55.49894
56.58604    57.67314    58.76024    59.84734    60.93444];

acceleration=[113.81037 227.62074   341.430009  455.240379  569.050749
682.861119  796.671489  910.481859  1024.291128 1138.101498 1251.911868
1365.722238 1479.532608 1593.342978 1707.152247 1820.962617 1934.772987
2048.583357 2675.422293 2708.413758 2741.405223 2774.396688 2807.388153
2840.379618 2873.371083 2906.362548 2939.354013 2972.345478 3005.336943
3038.328408 3071.319873 3104.311338 3137.302803 3170.294268 3203.285733
3236.278299 3269.269764 3302.261229 3335.252694 3368.244159];
plot(deformation,acceleration/g);
```

12.2 APPENDIX-II

EL-CENTRO GROUND ACCELERATION DATA AT 0.02 SEC. INTERVAL FOR 30 SECOND

```
[0.00630 0.00364 0.00099 0.00428 0.00758 0.01087 0.00682 0.00277
   -0.00128 0.00368 0.00864 0.01360 0.00727 0.00094 0.00420 0.00221
   0.00021 0.00444   0.00867   0.01290   0.01713  -0.00343  -0.02400  -
0.00992
   0.00416 0.00528   0.01653   0.02779   0.03904   0.02449   0.00995
0.00961
   0.00926 0.00892  -0.00486  -0.01864  -0.03242  -0.03365  -0.05723  -
0.04534
  -0.03346 -0.03201  -0.03056  -0.02911  -0.02766  -0.04116  -0.05466  -
0.06816
  -0.08166 -0.06846  -0.05527  -0.04208  -0.04259  -0.04311  -0.02428  -
0.00545
   0.01338 0.03221   0.05104   0.06987   0.08870   0.04524   0.00179  -
0.04167
  -0.08513 -0.12858  -0.17204  -0.12908  -0.08613  -0.08902  -0.09192  -
0.09482
  -0.09324 -0.09166  -0.09478  -0.09789  -0.12902  -0.07652  -0.02401
0.02849
   0.08099 0.13350   0.18600   0.23850   0.21993   0.20135   0.18277
0.16420
   0.14562 0.16143   0.17725   0.13215   0.08705   0.04196  -0.00314  -
0.04824
  -0.09334 -0.13843  -0.18353  -0.22863  -0.27372  -0.31882  -0.25024  -
0.18166
  -0.11309 -0.04451   0.02407   0.09265   0.16123   0.22981   0.29839
0.23197
   0.16554 0.09912   0.03270  -0.03372  -0.10014  -0.16656  -0.23299  -
0.29941
  -0.00421 0.29099   0.22380   0.15662   0.08943   0.02224  -0.04495
0.01834
   0.08163 0.14491   0.20820   0.18973   0.17125   0.13759   0.10393
0.07027
   0.03661 0.00295  -0.03071  -0.00561   0.01948   0.04458   0.06468
0.08478
   0.10487 0.05895   0.01303  -0.03289  -0.07882  -0.03556   0.00771
0.05097
   0.01013 -0.03071  -0.07156  -0.11240  -0.15324  -0.11314  -0.07304  -
0.03294
   0.00715 -0.06350  -0.13415  -0.20480  -0.12482  -0.04485   0.03513
0.11510
   0.19508 0.12301   0.05094  -0.02113  -0.09320  -0.02663   0.03995
0.10653
   0.17311 0.11283   0.05255  -0.00772   0.01064   0.02900   0.04737
0.06573
   0.02021 -0.02530  -0.07081  -0.04107  -0.01133   0.00288   0.01709
0.03131
  -0.02278 -0.07686  -0.13095  -0.18504  -0.14347  -0.10190  -0.06034  -
0.01877
   0.02280 -0.00996  -0.04272  -0.02147  -0.00021   0.02104  -0.01459  -
0.05022
  -0.08585  -0.12148  -0.15711  -0.19274  -0.22837  -0.18145  -0.13453
-0.08761
  -0.04069   0.00623   0.05316   0.10008   0.14700   0.09754   0.04808
-0.00138
```

0.05141 0.10420 0.15699 0.20979 0.26258 0.16996 0.07734
-0.01527
-0.10789 -0.20051 -0.06786 0.06479 0.01671 -0.03137 -0.07945
-0.12753
-0.17561 -0.22369 -0.27177 -0.15851 -0.04525 0.06802 0.18128
0.14464
0.10800 0.07137 0.03473 0.09666 0.15860 0.22053 0.18296
0.14538
0.10780 0.07023 0.03265 0.06649 0.10033 0.13417 0.10337
0.07257
0.04177 0.01097 -0.01983 0.04438 0.10860 0.17281 0.10416
0.03551
-0.03315 -0.10180 -0.07262 -0.04344 -0.01426 0.01492 -0.02025
-0.05543
-0.09060 -0.12578 -0.16095 -0.19613 -0.14784 -0.09955 -0.05127
-0.00298
-0.01952 -0.03605 -0.05259 -0.04182 -0.03106 -0.02903 -0.02699
0.02515
0.01770 0.02213 0.02656 0.00419 -0.01819 -0.04057 -0.06294
-0.02417
0.01460 0.05337 0.02428 -0.00480 -0.03389 -0.00557 0.02274
0.00679
-0.00915 -0.02509 -0.04103 -0.05698 -0.01826 0.02046 0.00454
-0.01138
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