NEAR AND FAR FAULT GROUND MOTION EFFECT ON CONCRETE GRAVITY DAM

A DISSERTATION

Submitted in partial fulfilment of the requirement for the award of the degree
of
MASTER OF TECHNOLOGY
in
EARTHQUAKE ENGINEERING

(With specialization in structural Dynamics)
by

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CANDIDATE'S DECLARATION

I hereby declare that the work which is being presented in this dissertation report entitled, "**Near and Far Fault Ground Motion Effect on Concrete Gravity Dams**" in partial fulfillment of requirement for the award of the degree of **Master of Technology** in **Earthquake Engineering** with **specialization in Structural Dynamics**, submitted in the **Department of Earthquake Engineering**, **Indian Institute of Technology Roorkee**, is an authentic record of my own work carried out under the supervision of **Dr. Pankaj Agarwal**, Professor and Head, Department of Earthquake Engineering, IIT Roorkee, Roorkee.

I also declare that I have not submitted the matter embodied in this report for award of any other degree.

Place: Roorkee **Prince Pundrik**

Date: **17526013**

CERTIFICATE

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

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Thank you,

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ABSTRACT

This thesis tends to investigate the effect of near fault and far fault ground motions on concrete gravity dam. Forward directivity and long duration high energy velocity pulse, causes response of the dam under near fault earthquake to be significantly different in comparison to the far fault earthquake. A linear time history analysis is carried out an ABAQUS software for an arbitrary dam using the concept of finite element, which is based on the cumulative inelastic duration of the stress history produced by performing a linear time history analysis of the model concrete gravity dam. The acceptance levels of DCR are examined by the performance exhibited by the concrete gravity dam during the dynamic analysis. For the above analysis, total of 16 earthquakes of both near and far fault categories are selected and applied on full reservoir condition. The result of the analysis indicates that near fault ground motions which significantly effects the response of dam-reservoir-foundation system, causes more damage to the dam in comparison to far fault ground motion.

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Chapter-1

Introduction

1.1 General

A gravity dam made of concrete is a solid structure, made normal to a stream flow to accumulate a reservoir on its upstream. It is the most important Civil Engineering structures related to hydro energy. Non-overflow and spillway are the two major bifurcations in gravity dam. Height of gravity dams depend a lot on the soil conditions of the place where foundation is to be laid.

Worries about the seismic security of concrete dams have been developing during ongoing years, halfway, on the grounds that the populace in danger in areas downstream of real dams keeps on growing and furthermore in light of the fact that it is progressively apparent that the seismic structure ideas being used at the time most existing dams were manufactured were lacking. Since the Northridge and Hyogoken-Nanbu (Kobe) earthquakes, there has been much talk about the ampleness of design practice with regards to concrete dams. Such an examination happens after each harming tremor, and, truth be told, the seismic arrangements of structures depend to a great extent on involvement from actual quakes.

The peril presented by large dams has been shown since 1928 by the disappointment of numerous dams of various kinds and in numerous places of the world. In any case, no disappointment of a concrete dam has come about because of seismic tremor excitation; in actuality the main complete damage of concrete dams has been because of disappointments in the ground rock supporting the dams. Then again, two critical cases of tremors harm to concrete dams happened during the 1960s: Hsinfengkiang in China and Koyna in India. The harm was serious enough in the two cases to require significant fixes and reinforcing, however the reservoirs were not discharged, so there was no flooding harm. This phenomenal record, in any case, isn't adequate reason to justify about the seismic security of concrete dams, in light of the fact that no such dam has yet been exposed to most extreme possible earthquake while at the same time holding a full reservoir level. Consequently, it is fundamental that all current concrete dams in seismic areas, just as new dams made arrangements for such locales, be checked to verify that they will perform safely during the worst earthquake to which they may be oppressed particularly in the near - field regions.

A plenty of dams around the global are built in high seismic zones. An improved understanding of the response of the dams to earthquake motion is necessary for the evaluation of risk associated with the already existing dams as well as for the successful design. The safe performance of such structures is essential for economic as well as environmental consideration. In the large number of cases, the failure of the dams has resulted in disastrous consequences.

The structures those vibrating in the air and those surrounded by water are two different systems having different dynamics characteristics. This is due to the interaction between structure and water which results hydrodynamics pressure and makes determination of dynamic forces very complicated. These oscillations result in impulsive and convective pressures. The convective pressures generally have insignificant magnitude so convective pressures are neglected. The impulsive pressure is generally experienced by the dam as hydrodynamics pressures. From this fact it can be said that the reservoir of the dam also interacts with the dam when it is subjected to dynamic loading due to phenomena such as earthquake.

This thesis deals with concrete dam response during considerably large earthquakes and forces on near-field and far-field effects in full reservoir and rigid base condition. An arbitrary dam is considered for analysis. The initial sections consist of current information about characteristics of near-field and far-field ground motions. Then by using a set of ground motions recorded in both the near-field and far-field of recent earthquakes, a detailed study of the plausible damage that can occur to the dam due to both of them are made to compare using the linear time history analysis in terms of stress Demand Capacity Ratio (DCR) and Cumulative Inelastic Duration/ Cumulative Overstress Duration (COD) analyses, which are carried on ABAQUS software.

1.2 Limit state concepts of concrete gravity dam

Two limit state concepts to be considered in earthquake design of dams namely serviceability limit condition and limiting condition of ultimate load. Serviceability limit condition is used for Design Basis Earthquake (DBE) which has the 50% chances of not being exceeded in next 100 years (ICOLD, 1989). According to the serviceability limit state method the design is based on linear-elastic dynamic analysis and allowable stress concept and the design being acceptable only if the principal stresses are within the biaxial strength curve of concrete. But in this limit state method for concrete dams no cracks are accepted in the structure. In limit state of ultimate load, the criteria simply tell us that cracking of concrete is permitted. The extent of cracking gives us the performance criteria for the structure. Due to formation of cracks the dam concrete is divided into concrete blocks. These concrete blocks are to be analyzed for the dynamic stability. Wieland (Wieland, 1996) in his paper said, “In ductile structures it is assumed that the ductility of the structure will prevent the failure during an earthquake exceeding the DBE, whereas in concrete dams the MCE is the upper limit which cannot be exceeded. Therefore, if the dam is stable in the MCE, it is safe in deterministic sense.” Performance criteria of dams are to be dealt case by case, there are no universally applicable criterion to evaluate the safety of the dams. Performance of concrete gravity dams under seismic loading is generally assessed by checking the stresses developed combined with the engineering judgments. Wieland (Wieland, 1996) defined two performance criteria to be applicable in accordance with the limit state design concept.

1) Design Level (DBE): No structural cracks are accepted and both the static and dynamic stresses together should lie within the allowable biaxial stress curve.

2) Safety Level (MCE): Structural cracks are accepted, dynamic stability of the dam is not compromised, and equipment and installations to bring down the reservoir level and spillways must be functional after an earthquake.

Dam deformations are not to be worried in DBE level, but for MCE, the rocking and sliding deformations of the blocks are to be kept in limit. These criteria differ from dam to dam and should be analyzed dam to dam basis. All the existing criteria have been utilizing the static tensile strength of the concrete although there is enough evidence that under dynamic loading the tensile strength of concrete will increase to certain extent.

1.3 Objective

- To assess the behavior of concrete gravity dam under seismic forces.
- To study and compare the damage that can occur to the dam due to both near and far field earthquakes using the linear time history analysis.

Chapter-2

Near and Far Fault Earthquakes

2.1 General

Ground movement records that got in late, some major earthquakes, uncovered interesting qualities of ground movements in a near fault region. There are lot of evident differences between earthquakes originating near to the fault and far away from the fault according to the recordings available (Chopra and Chintanapakdee, 2001). The most obvious attributes of near fault ground movements are forward directivity and excursion impacts as recognized by the researchers in this field (Mavroeidis and Papageorgiou, 2003). Velocity pulses and large displacements are quite frequently observed in the normal segments of the fault. Far field ground motions are not found with the articulated pulses that are characterized with the near field earthquakes. In view of the attributes of near fault ground movements, extensive harm and huge response are the possibilities of structure, which is resulting because of the high input energy at start (WI et al., 2004). In this manner, basic reaction to these ground movements has gotten much consideration as of late. The impacts of near-fault ground movements on numerous structural designing structures, for example, dams and towers, and so on., have been explored in numerous ongoing studies (Dicleli and Buddaram, 2007). Recently done investigations and studies uncovered that there are increasing demand on displacement and stresses when the dam is exposed to near fault ground movements. Even though, past examinations gave some data about the impacts of near fault ground movements on the response of dams, concrete gravity dams are still not very well researched regarding the harm that can be caused due to the near fault motions.

2.2 Different approaches for dam modelling

In the present era, a lot of interest is being given to the dynamic response of fluid-solid interaction system in various fields of engineering as mechanical, civil, aerospace. To take the fluid effect in analysis of dam for strong ground motions, lots of methods are available which have been grouped into three categories, the Lagrangian method, the added mass or virtual mass method, and the Eulerian method, Fluid present on the upstream face of the dam is replaced by lumped mass in added mass approach, calculation of which is done using Westergaard (Westergaard, 1933) or IS code method. The ignorance of damping effect and stiffness is a drawback as only portion of fluid was fluid mass added.

Displacement is the degree of freedom for both reservoir and structure when Lagrangian is considered also irrespective of the shear stiffness Lagrangian equation is the one that governs and acts same as solid. Since the degree of freedom for both structure and reservoir is the same there is no need to provide special interface equations as compatibility is automatically satisfied at nodes. This method offers an advantage of adding the fluid elements directly into the software being used for structural analysis.

With degree of freedom being different, Navier-Strokes equations for fluid and Lagrangian equation for solids govern the motion of fluid in Eulerian method. These equations are reduced to wave equations for inviscid fluid and for small amplitude motions. While solving with the above method, every system is solved uniquely and using an iteration method, the interaction between them should be considered.

2.2.1 Added Mass approach

When a body is decelerated or accelerated, system is added with inertia which is the added mass in the fluid system. Simultaneous occupation by the fluid and object is a general problem whenever added mass is considered. Although in reality every fluid mass will accelerate to different degrees, modelling can be done as though object is moving and volume of fluid mass is moving with it, which is much simpler comparatively.

2.2.2 Lagrangian approach

The assumptions of small displacement and inviscid fluid is used in Lagrangian approach for modelling the fluid. In the approach the displacement is the degree of freedom for both structure as well as fluid. Displacement based finite elements are used for discretizing the fluid domain. The penalty method is used for imposing the irrotational condition for inviscid fluids.

2.2.3 Eulerian approach

The degree of freedom in Eulerian approach is displacement and pressure for structure and fluid respectively i.e., different terms are used for modelling the fluid and solid. The assumptions of small displacement and in-viscid fluid is used for modelling the fluid. As compared to Lagrangian approach, the Eulerian will involve large numerical computations as the system will be a coupled system.

Special workstations are required for solving the coupled system because the variables are different in this method.

2.3 Previous Study

Scientists have given much thought on the investigation of characterization of 'pulse like' movement in the near field area and the impact of near field ground movements on different structures. In a progression of research, Bayrakter (Bayrakter et al., 2008) and his collaborators explored the impact of near field and far field ground movements on seismic response of different kind of dams, for example, concrete gravity dams, concrete faced rock fill and clay core rock fill dams. Zhang and Wang (Wang et al., 2014) did an examination on the effect of near field and far field ground movements on seismic harm of concrete gravity dams.

Yazdani and Alembagheri (Yazdani et al., 2017) found the proportional pulse in forward directivity (FD) and non-forward directivity (NFD) near field ground movements and examined the impact of pulse period on the response of concrete gravity dams.

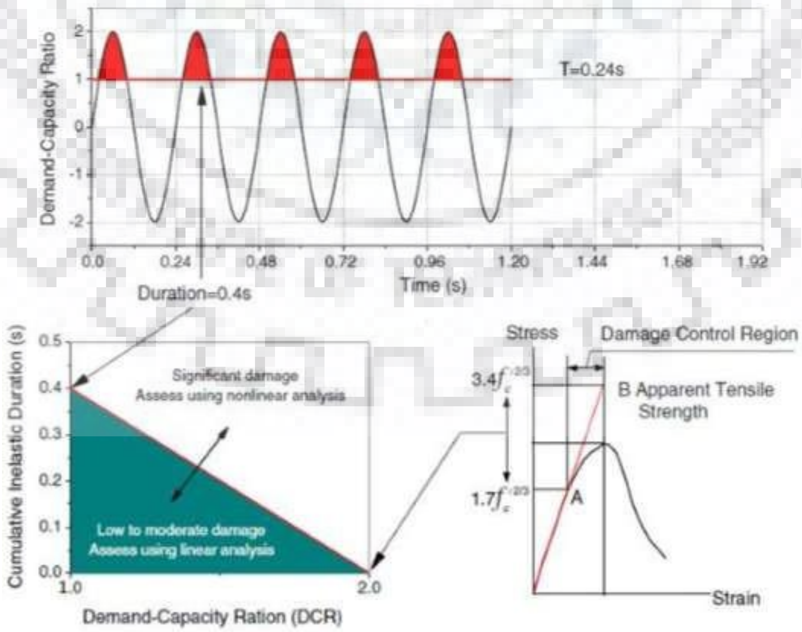

Figure 2.1*:* Diagram of seismic performance evaluation, (after Ghanaat, 2004: Wang et al., 2014)

The nonlinear examination of dams demands more computational time than linear investigation. Ghanaat (Ghanaat, 2004; Ghanaat, 2002) proposed a standard and rational technique for performance assessment of concrete dams from linear time history investigation as stress Demand Capacity Ratio (DCR), Cumulative Inelastic Duration/Cumulative Overstress Duration (COD) and level of overstressed area (Fig 2.1). The Demand Capacity Ratio is characterized (Ghanaat, 2004; Ghanaat, 2002) as "the proportion of the resulted principle stress to tensile strength of concrete".

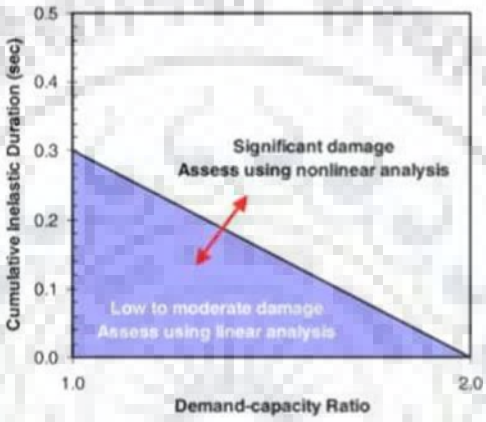

Figure 2.2: Damage provision for dam, (after Ghanaat, 2004)

The most extreme permitted DCR for linear analysis of concrete dams is 2 relating for a pressure demand double the tensile strength of concrete. The all-out time of stress exceedance over a stress level related with a specific DCR is named as Cumulative Overstress Duration (Wang et al., 2014). Fig 2.2 shows the performance/damage criteria for concrete gravity dams as proposed by (Ghanaat, 2004; Ghanaat, 2002). The criteria and portrayal of three damage levels can be found in the investigation of Ghanaat (Ghanaat, 2004; Ghanaat, 2002) and Wang (Wang et al., 2014).

2.4 Near fault and Far fault ground motion attributes

Mechanism of the source, route through which it propagates, condition of the soil at the particular site etc. greatly affect the seismic ground motion which is caused by fault rupture followed by huge energy discharge and because of the involvement of lot of variables we can’t predict the phenomenon. The ground movement from far locale is unique in its attributes in contrast to the one which is near to the fault because of the fact that these waves vary with source, size, distance and direction from rupture area and attributes of the neighborhood soil condition. Near field ground movements are not just linked with solid shaking but also with geometry of fault and direction in which seismic wave is moving. The near field zone is commonly thought to be limited around a separation of 20 km (Davoodi, 2013) from the rupture fault. The fundamental recognized characteristics for the near field ground movements is the presence of unique, high intensity large pulse Fig 2.3 towards the start of the ground movement and which is obvious at velocity time history (Yazdani et al., 2017). The estimation of Peak Ground Velocity (PGV) of near field ground movements is higher than normal ground movements, to a great extent, and the proportion of PGV to PGA (Peak Ground Acceleration) is more than 0.1 sec (Akköse and Şimşek, 2010). The pulse like movements are brought about by the directivity effects.

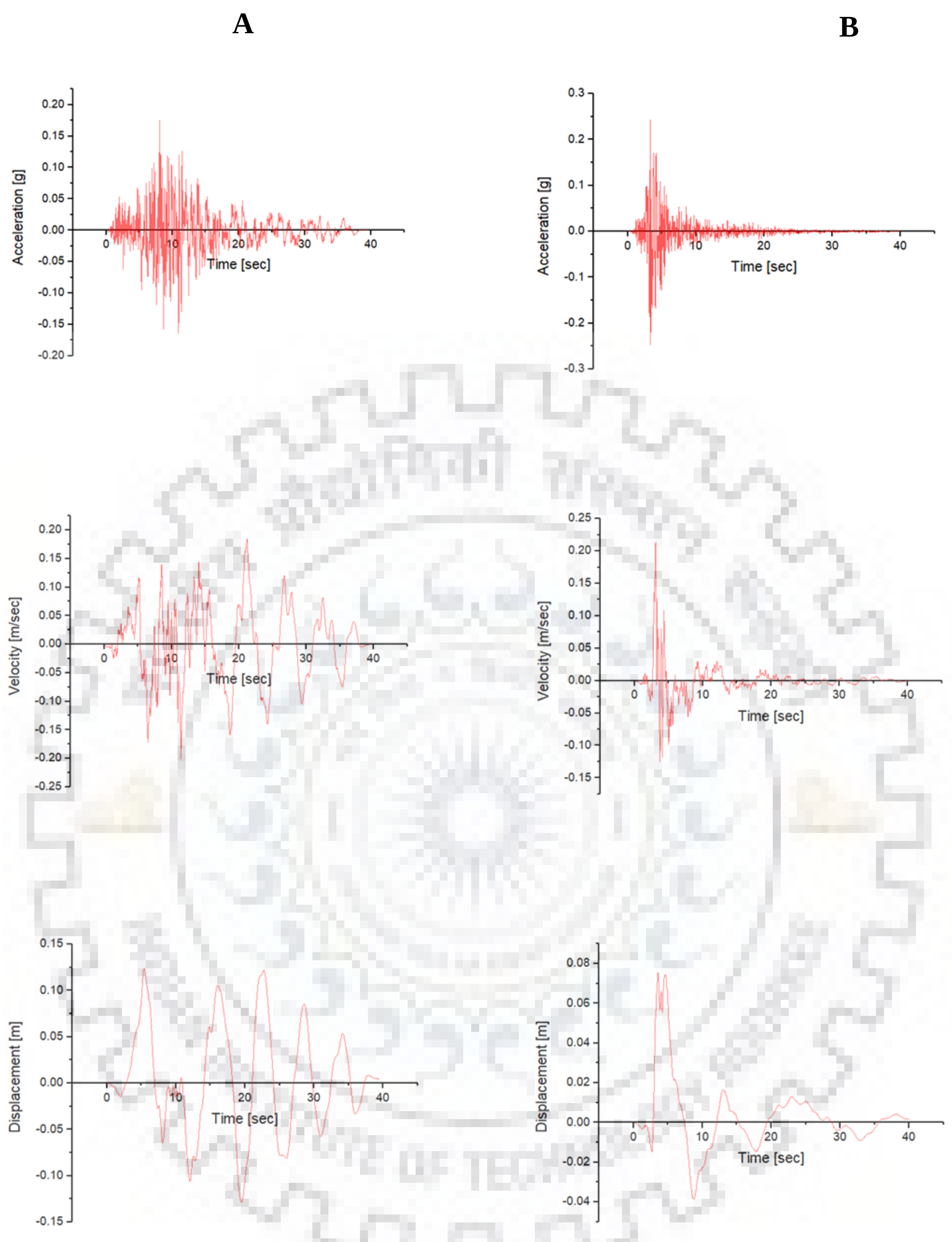

Figure 2.3: Time history example of acceleration, velocity and displacement for (A) Far Field (B) Near Field

Recent earthquakes have shown the serious harm which occurred due to fling step effect and forward directivity both of which are essential attributes of near field ground movements (Mavroeidis and Papageorgiou, 2003). The figure below defines the three types of directivity effects that are neutral, reverse and forward. If hypocenter is normal to the fault, neutral directivity happens. Longer duration and lower amplitude are attributes of reverse directivity which is exhibited by rupture moving away from site. When site and fault are lined up in direction of the slip and movement of rupture front is towards the site, impacts of forward directivity can be seen. Strike-slip and dip-slip both have forward directivity impacts.

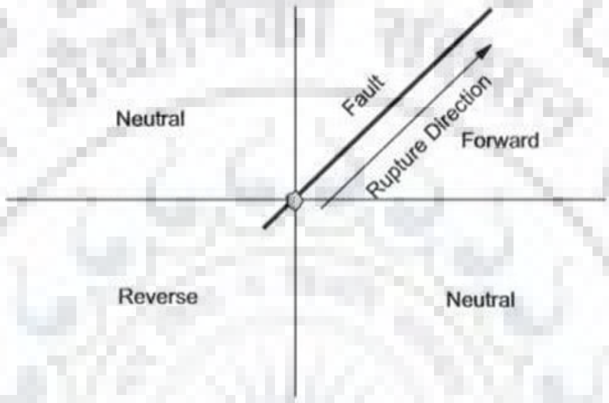

Figure 2.4*:* Defining Directivity

Chapter-3

Methods of Analyses

3.1 Introduction

The magnitude and the distribution of stresses in different locations of dam section can be determined thoroughly for different static and dynamic loading conditions. The main objective of the analyses is to investigate the adequacy of the structure and the interaction of it with the foundation. The type of the structure and its configuration are the only responsible issues for the selection of the method of analyses. Generally, approximate simplified methods or the finite element methods are used for the analyses of dam depending upon the refinement needed. Concrete dams being brittle in nature, most of the designs are based on the conventional methods like allowable stress methods. But, in the recent days the design philosophy of dams has changed concerned with the earthquake safety of dams. Gravity dams should be able to survive Maximum Credible Earthquake (MCE) without any catastrophic failure resulting in loss of life and significant damage to property. MCE is the largest earthquake associated with a specific seismotectonic structure or source area within the regions of low seismicity. Linear analysis carried out with MCE results in tensile stresses exceeding greatly the tensile strength of the concrete calculated and hence the design philosophy is no longer valid since cracking is expected to occur in the dam sections. The problem worsens if the elastic dam is bonded perfectly to the foundation rock. The tensile stresses at the heel of the foundation under full reservoir condition exceed the tensile strength of the concrete and with earthquake load acting on the dam system the stresses will be still higher. To eliminate the undesirable stresses, the uneconomical remedial measures such as post tensioning, dam thickening, and reinforcement would be necessary. And the engineers still adopting the conventional methods for design high tensile strength of mass concrete or small ground accelerations gives us no option but to come up with the performance criteria at both Design Basis Earthquake (DBE) and MCE levels. The following are few of the methodologies which are being used worldwide for analyses, stability, performance of a concrete gravity dam.

Different analyses for the concrete gravity dam can be carried out using following steps, they are,

1) Static Analysis
2) Frequency Analysis (Modal Analysis)

3)	Linear dynamic analysis

3.2 Equivalent Static analysis

Static analysis is the most simplified analysis of a structure where the effects of a sudden change in structure are calculated without any long-term response due to that change on the structure. In case of static analysis, the forces used in the dam sections are,

1) Self weight of dam
2) Hydrostatic forces due to reservoir acting on the upstream face of the dam
3) Uplift force acting at the base of the dam
4) Silt load acting on the upstream side of the dam section
5) Weight of water acting on the foundation in upstream side

The force profiles acting on the dam section are shown in Fig-3.1

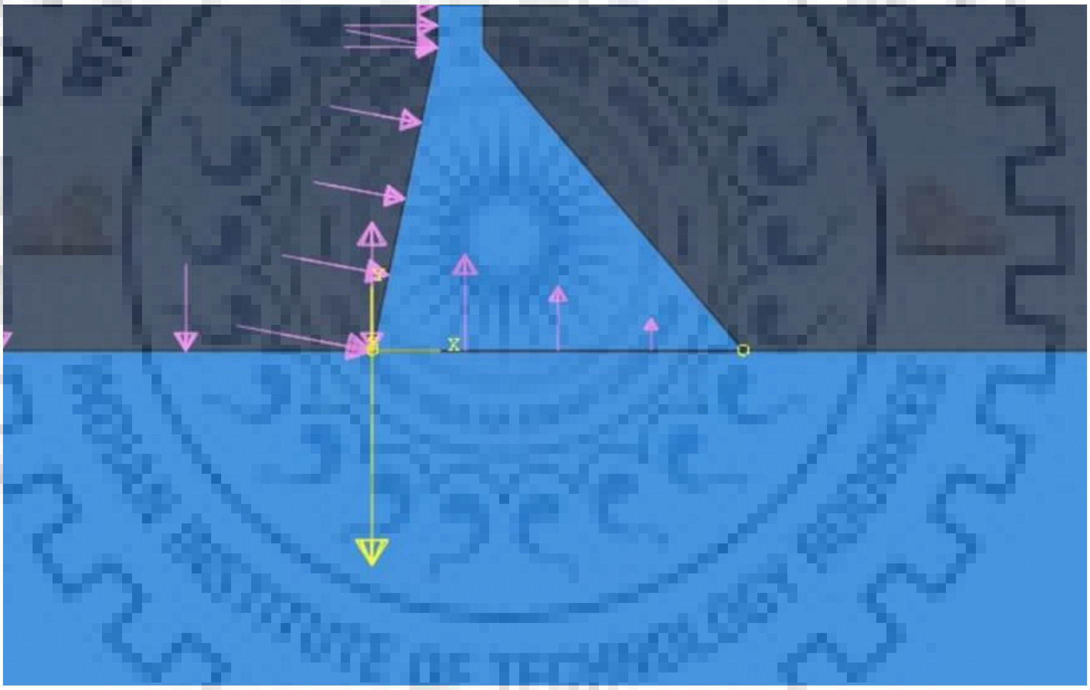

Figure 3.1: Static Loading in dam model

For the static analysis, inertia and hydrodynamic loads are not considered. The boundary conditions are taken as fixed at the foundation base and as roller on the foundation sides.

3.3 Frequency analysis (Modal analysis)

Free vibration analysis of the dam is performed. Lancoz solver is used to determine the frequency of the dam section for different modes. This method is useful to find the mode shapes with their natural frequencies. The response of each mode can be calculated separately from this analysis. Mode superposition is carried out using the responses of different modes obtained from this analysis. Another important parameter, mass participation factors of different modes can be calculated using this method of analysis which can tell us whether the multimode analysis is needed or not over the single mode analysis.

3.4 Time History analysis

Time history analysis is a step-by-step analysis of the dynamic response of a structure to a specified loading (earthquake) that varies with time. Time history analysis is used to obtain a more accurate seismic response of the structure under dynamic loading of a representative earthquake. In this type of analysis, the response of the structure at each time step specified can be obtained, which aids in properly studying the behavior of the structure. Another advantage of time history analysis is the ground motions/loading can be applied in multiple directions simultaneously.

Linear time history analysis was performed so as to get an idea as to the behavior of the structure when subjected to the earthquake. The performance of the building in the linear range can be compared to the results obtained from other linear analysis like that of equivalent static analysis. Although linear time history analysis does not include the nonlinear behavior of the structure, it will give some insight about our study regarding near field and far field earthquakes.

In this thesis the model of dam is subjected to accelerations from earthquake records that represent the expected earthquake at base of the foundation. The dam section given in the problem is analyzed for Lumped mass with elementary boundary condition. The deconvoluted time history is applied at the bottom of the soil or rock foundation. The acceleration is applied at the base of the rock foundation.

3.4.1 Ground Motion

Dynamic assessment of gravity dams involves an important part, selection of ground motion. In this investigation, 16 earthquake records are chosen as the ground motions having both near field and far field data.

Selection of data has been carried out using provisions of ASCE/SEI 7-16. The ground motion selected are to be scaled either by amplitude scaling method or by spectral matching for a particular period range of the structure under consideration. This period range has an upper bound greater than or equal to 2 times the largest first-mode period and a lower bound equal to the period at which at least 90% mass participation is achieved. The ground motions selected are then deconvoluted and applied at the base of the foundation rock.

3.4.2 Added mass approach as per IS 1893:1984

According to IS 1893: 1984, "if the height of the vertical portion of the upstream face of the dam is equal to or greater than one-half the total height of the dam, analyze it as if vertical throughout. If the height of the vertical portion of the upstream face of the dam is less than one-half the total height of the dam, use the pressure on the sloping line connecting the point of intersection of the upstream face of the dam and the reservoir surface with the point of intersection of the upstream face of the dam with the foundation". In the present dam geometry, the height of the vertical portion of the upstream face of the darn is less than one-half height of the dam and hence it is not analyzed as vertical throughout for frequency extraction. Due to horizontal acceleration of ground motion at the base of the dam there is an instantaneous hydrodynamic pressure or suction exerted on the dam. Based on the assumption that the water is incompressible, the hydrodynamic pressure at depth 'y' below the reservoir surface shall be determined as follow:

$$P = C_S \, \alpha_h \, wh \tag{3.1}$$

where,

P = Hydrodynamic pressure in kg/m^2 at depth y,

Cs= Coefficient which varies with shape and depth

α_h= Design horizontal seismic coefficient

w = Unit weight of water in kg/ m^3, and

h = Depth of reservoir in m.

The variation of coefficient Cs with shapes and depths is illustrated in Appendix G of IS: 1893- 1984 code

$$Cs = \frac{Cm}{2}\left\{\frac{y}{h}\left(2-\frac{y}{h}\right)+\sqrt{\frac{y}{h}\left(2-\frac{y}{h}\right)}\right. \qquad (3.2)$$

where,

C_m = Maximum value of C_s

y = Depth below the water surface, and

h = Depth of the reservoir

The mathematical model really consists of a lumped mass representation of actual structure as shown in figures below:

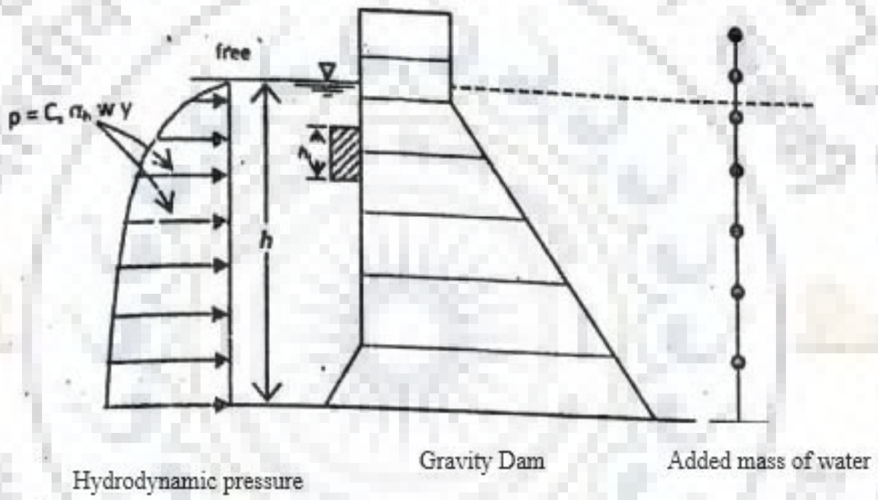

Figure 3.2*:* Hydrodynamic pressure

In the lumped mass system, the distributed mass of structure is lumped at discrete points and these masses are connected with each other by massless elastic segments. As per IS Code, "for dams up to '100 in height, the seismic coefficient method shall be used for the design of the dams, while for dams over 100 m height the response spectrum method shall be used. Both the seismic coefficient method (for dams up to 100 m height) and response spectrum method (for dams greater than 100 m height) are meant only for preliminary design of dams". The design value of horizontal seismic coefficient is calculated using following expression given in (IS 1893:1984).

$$\alpha_h = \beta\, IF_0 S_a / g \qquad (3.3)$$

Where,

β = a seismic coefficient depending upon the soil-foundation system

I= Importance Factor (for dam, I=3)

F_0 = seismic zone factor (avg. acceleration spectra)

Sa / g = average spectral coefficient for appropriate natural period and damping of structure

The fundamental period of vibration as per IS 1893:1984 is given as –

$$T = 5.55\, H^2 / B\, (\gamma_m / (gE_m))^{0.5} \qquad (3.4)$$

Where, H = height of the dam in m,

B = base width of the dam in m,

γ_m= unit weight of the material of dam in N/m^3

g = acceleration due to gravity in m/s^2, and

E_m = modulus of elasticity of the material in N/m^2

3.5 Tensile strength of mass concrete

The tensile strength of concrete is a very ghostly parameter which was under investigation for very long periods of time. There is no direct method available to determine the tensile strength of concrete. It is derived using indirect methods like flexural tensile strength tests and splitting tensile tests under static loading. Many authors have proved that the tensile strength of concrete varies with strain rates. The material properties of concrete including modulus of elasticity, and poisson's ratio. The following are the discussions on the tensile strength of mass concrete.

3.5.1 IS code provisions

IS 456:2000 provides the tensile strength of concrete based on splitting tensile tests and flexural tensile tests. Tensile strength is estimated as a part of compressive strength and the following formula may be used for flexural strength,

$$f_t = 0.7\sqrt[2]{f_{ck}} \qquad (3.5)$$

where, f_{ck} is the characteristic cube compressive strength of concrete in N/mm^2

IS 6512-2013 provides the permissible tensile stresses in dams against each type of load combination.

3.5.2 Raphael criteria

Raphael (Raphael, 1984) found a theoretical relationship between tensile strength and modulus of rupture especially for dam concrete under static and seismic loading conditions. The static tensile strengths were derived conducting series of splitting tensile tests and flexural tensile tests. Testing a huge number of samples (nearly 20,000) extracted from dam core under static loading conditions the relation between tensile strength, (f_t) and compressive strength, (f_c) is obtained

$$f_t = 1.7 f_c^{2/3} \quad \text{(psi)} \tag{3.6}$$

and

$$f_t = 0.7 f_c^{2/3} \quad \text{(kg/cm}^2\text{)} \tag{3.7}$$

Under dynamic loading it has been shown that the compressive strength and tensile strength have been found to have increased tremendously. The tests were conducted under varying speeds ranging from one hundredths of a second to hundreds of a second. In all the conditions it is observed that increasing the rate of loading has resulted in the increasing of both strength and modulus of elasticity of concrete. The following conclusions on tensile strengths under static and seismic loading were determined from the series of experiments conducted on mass concrete. Use of the above tensile strength has also been mentioned in NCSDT guidelines.

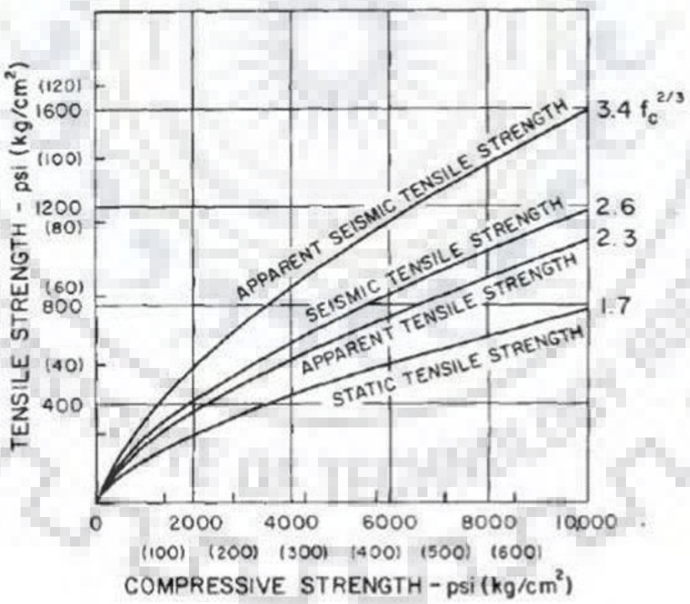

Figure 3.3*:* Design chart for tensile strength, (after Raphael, 1985)

Chapter-4

Modeling of Dam System

4.1 Introduction

Three-dimensional model of concrete gravity dam reservoir system is assumed to be two dimensional after taking the assumption that geometry and properties of material vary very slowly along the axis. Seismic analysis of buildings and other engineering structures is often based on the assumption that the foundation is rigid, which is subjected to unidirectional horizontal ground acceleration. The type of interaction between dam and foundation as well as dam and reservoir have a huge impact on the response achieved by the concrete gravity dam after subjecting to seismic ground motion. The simplified techniques generally neglect the interaction between the dam-reservoir-foundation system, whereas the various rigorous methods proposed by researchers, range from finite element modelling to boundary integral equations.

4.2 Geometrical Description

The gravity dam geometry considered for the present study is shown in figure 3.1. A 2-D FEM analysis of the dam foundation system has been carried out using the finite element package, ABAQUS.

A concrete gravity dam with the height of 114m, a base width of 112m, a downstream slope of 1:0.8, and an upstream slope of 1:0.2, is chosen as a numerical example.

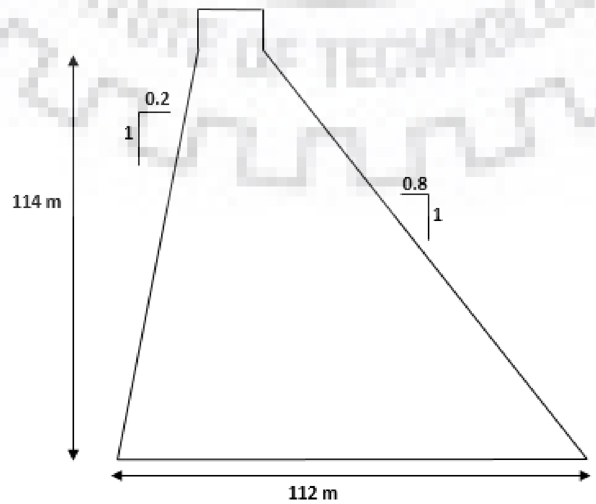

Figure 4.1: Physical dimension of the dam model

4.3 Material properties of the Dam and Foundation

The material properties of the dam used in the analysis are shown in table 4.1.

Table 4.1: Material properties of dam

	Concrete	Foundation Rock
Elastic Modulus (*GPa*)	30	16.5
Poisson's Ratio (v)	0.2	0.33
Density (Kg/m^3)	2500	2600

4.4 Dam-Reservoir-Foundation system

The foundation is considered to be a simplified massless body of 240 m in length along both the directions and 180 m in depth. The estimate of foundation extent is calculated by convergence studies with varying sizes of width of the foundation which is necessary to produce accurate results which must be included in the finite element models to accurately produce stresses in the dam.

The foundation width of 2 times the width of dam on each side and depth of 1.5 times the width of dam was finalized for finite element modeling of foundation. This is also supported by the manual by U.S. army corps. All the vertical grids are restrained from horizontal displacement and all the horizontal grids are restrained from vertical displacement. The foundation is modelled using 2D plane strain elements. As there is no medium to aid the wave propagation through the foundation model, the massless foundation model allows us to apply earthquake ground motion on the fixed boundaries of the foundation which are transmitted to the dam base with much changes.

In accordance with the properties of reservoir, bulk modulus and density of water are taken as 2.07GPa and 1000kg/m^3, respectively.

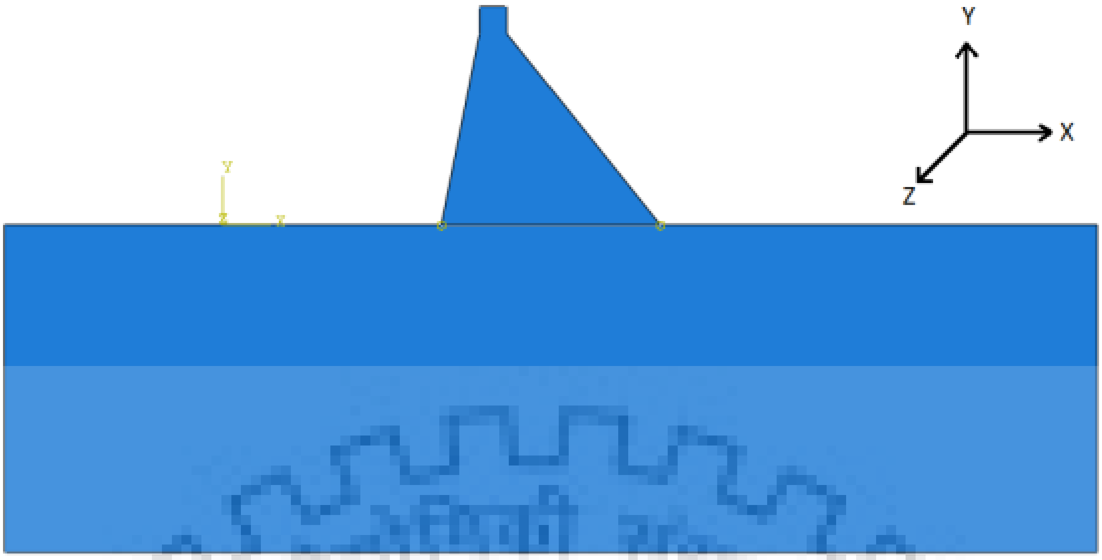

Figure 4.2: Dam chosen for analysis

4.5 Boundary Condition

The dam is resting on the rock foundation. The two sides of foundation are supported by roller with movement allowed in vertical direction in case of static analysis. In case of dynamic analysis, the ground motion is applied at the base of foundation.

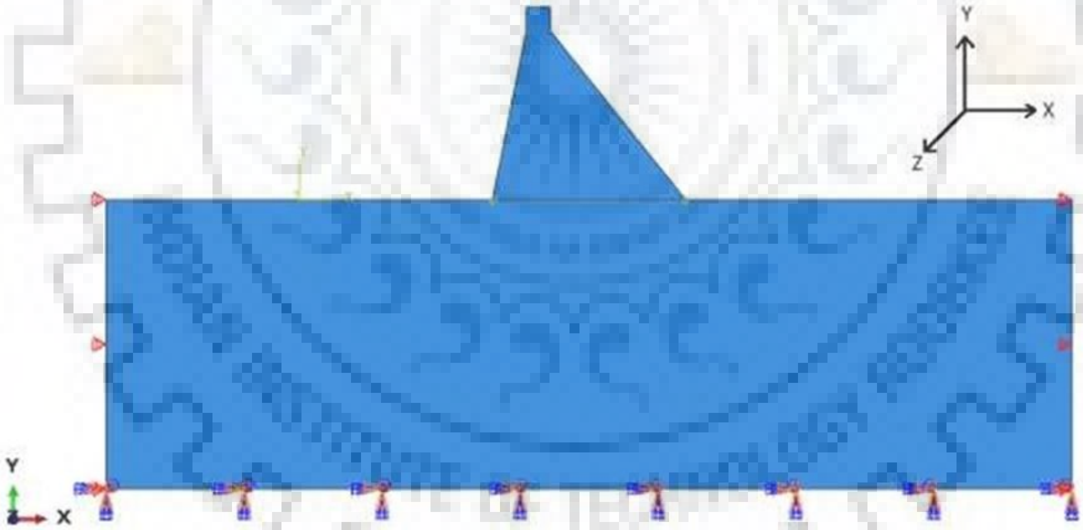

Figure 4.3: Foundation Boundaries

Waves are reflected back from the truncated boundary. So to create the non-reflecting boundary condition convergence analysis was carried out with varying width of the foundation extent and by applying a time history earthquake. The foundation extent so considered accounts for the variation that could have been caused due to reflecting back of waves.

4.5.1 Ground Response Analysis

Ground Response Analysis (GRA) was carried for further study related to reflecting back of waves from truncated boundary. Loma Preita earthquake was considered for analysis (earthquake data in table 6.3) and analysis was carried out in both ABAQUS and DEEPSOIL v6.1. Deconvoluted earthquake was applied at the base of foundation in our model on ABAQUS software considering the elementary boundary condition and response was calculated. The same earthquake was then applied on DEEPSOIL v6.1, which considers boundary to be of infinite extent, and again response was calculated. Above two responses were then compared and result shown in the graph below indicated that reflecting back of wave was accounted for.

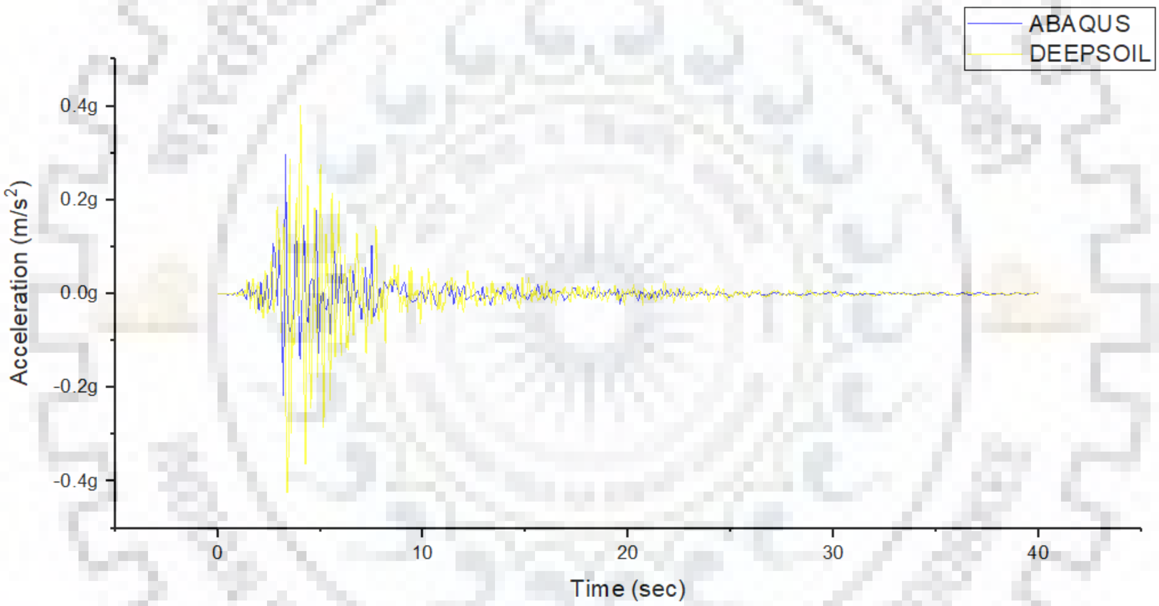

Figure 4.4: Comparing response of ABAQUS and DEEPSOIL

The response from ABAQUS is a bit delayed than that of DEEPSOIL because of the fact that analysis was carried out in 2D and 1D respectively.

4.6 Finite element used in modeling

In 2D model of dam foundation system CPE8R (8-node biquadratic, reduced integration) element is used to model both dam and foundation elements. ABAQUS 2D plane strain element CPE8R is shown in the Figure 4.5.

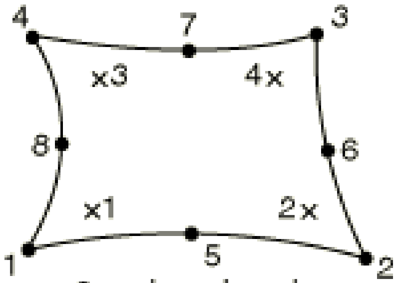

Figure 4.5: Geometry of
CPE8R element

This element is defined by 8 nodes having two degrees of freedom at each node, translation in the nodal x and y directions.

4.7 Element sizes for Dam and Foundation

As there are no particular design criteria for the fixing of mesh density, it is done by a trial and error method optimizing the acceptable accuracy and computation time both. Convergence study has been carried out to optimize the element size for dam body and foundation. Element size of 2.5 for both Dam and Foundation has been finalized for the present finite element analysis of concrete gravity dam.

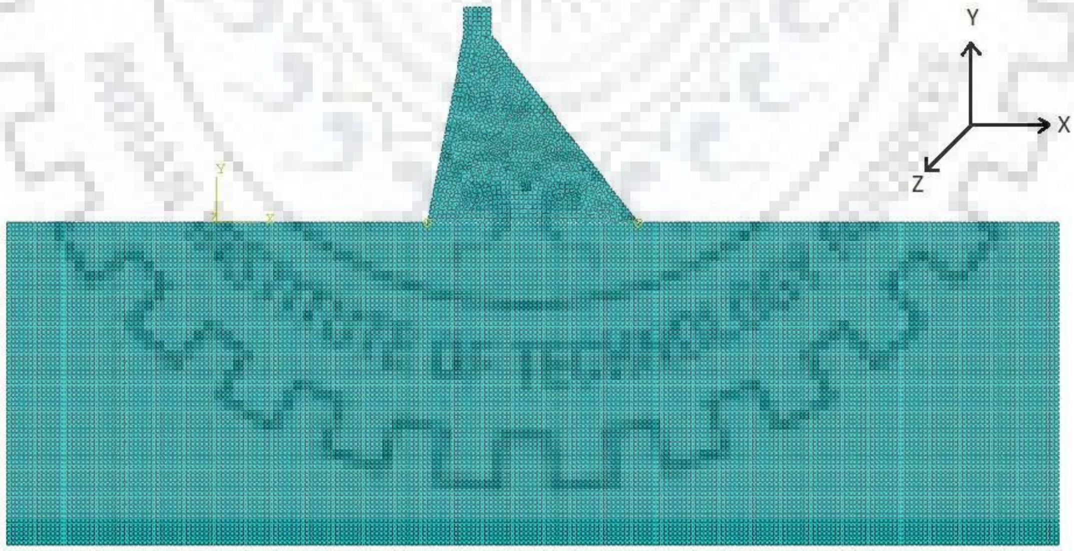

Figure 4.6: Finite element plot of Dam Foundation System

4.7.1 Convergence study

Convergence analysis is carried out for different mesh sizes using time history analysis. Stress in vertical direction (S22) at heel has been considered for study and Loma Preita at near fault has been considered for time history (data is provided in table 6.3). Mesh sizes considered are 8, 6, 5, 4, 3, 2.5 and 2 of which data can be seen converging around mesh size of 2.5 and 2 in the graph shown below. Considering both computation time and optimization of result, mesh size of 2.5 is considered for analysis.

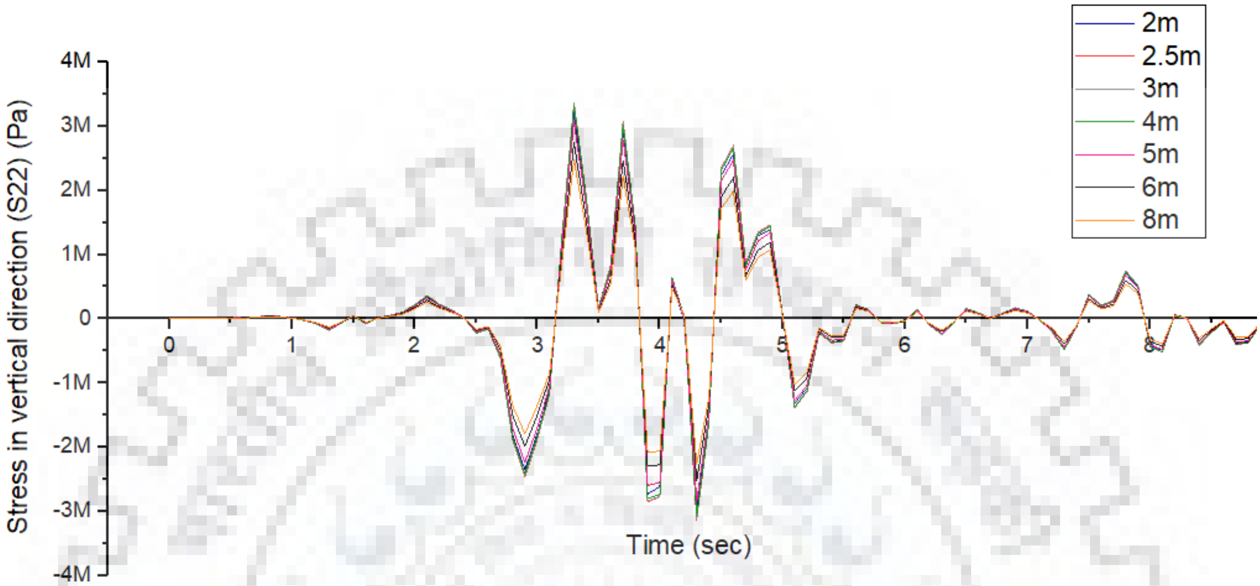

Figure 4.7: Stress plot for different mesh sizes

4.8 Lumped Mass

In case of seismic analysis, the dynamic effect of reservoir is undertaken by adding mass on the upstream face of dam using IS code approach.

In the present dam geometry, the height of the vertical portion of the upstream face of the dam is less than one-half height of the dam and hence it is analyzed using the pressure on the sloping line connecting the point of intersection of the upstream face of the dam and the reservoir surface with the point of intersection of the upstream face of the dam with foundation. There are enough lumped masses in the model in order to represent the dominant frequencies of the gravity dam. Hence the value of Cm, in accordance to slope, which is maximum value of pressure coefficient for the sloping faces used for calculation of Cs which is coefficient varying with shape and depth is taken as 0.65 from Fig. 10 of IS 1893- 1984. Water is assumed to be incompressible. The depth of reservoir id taken as 114m and density of water as 1000Kg/m3. Inertial mass is added at upstream side of Model as accordance to IS: 1893-1984.

The lumped mass model is shown in the figure below.

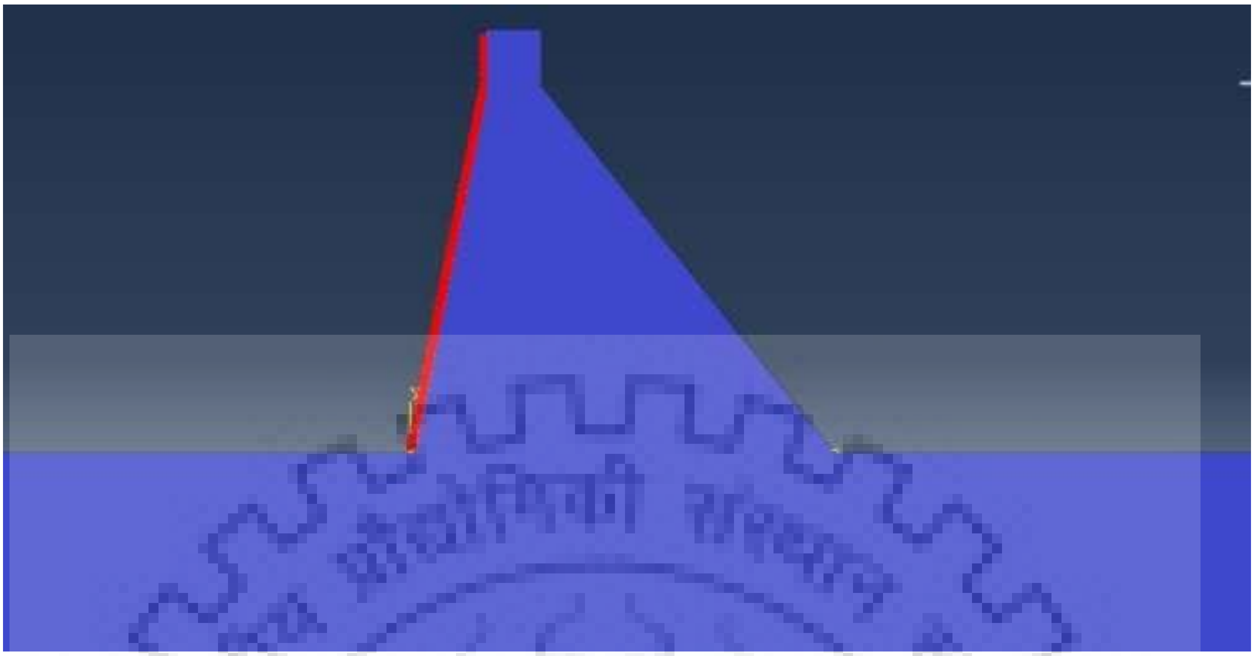

Figure 4.8: Lumped Mass Model

4.9 Free Vibration Characteristics

A modal analysis is carried out in ABAQUS of the dam body to understand the free vibration characteristics and also the predominant frequencies of the structure. First 4 typical mode shapes are shown below:

Table 4.2: Natural frequencies and natural time periods of first 4 modes

Mode	Frequency (Hz)	Time Period (sec)
1	2.0939	0.47757
2	3.5827	0.27911
3	4.0737	0.24547
4	5.0119	0.199525

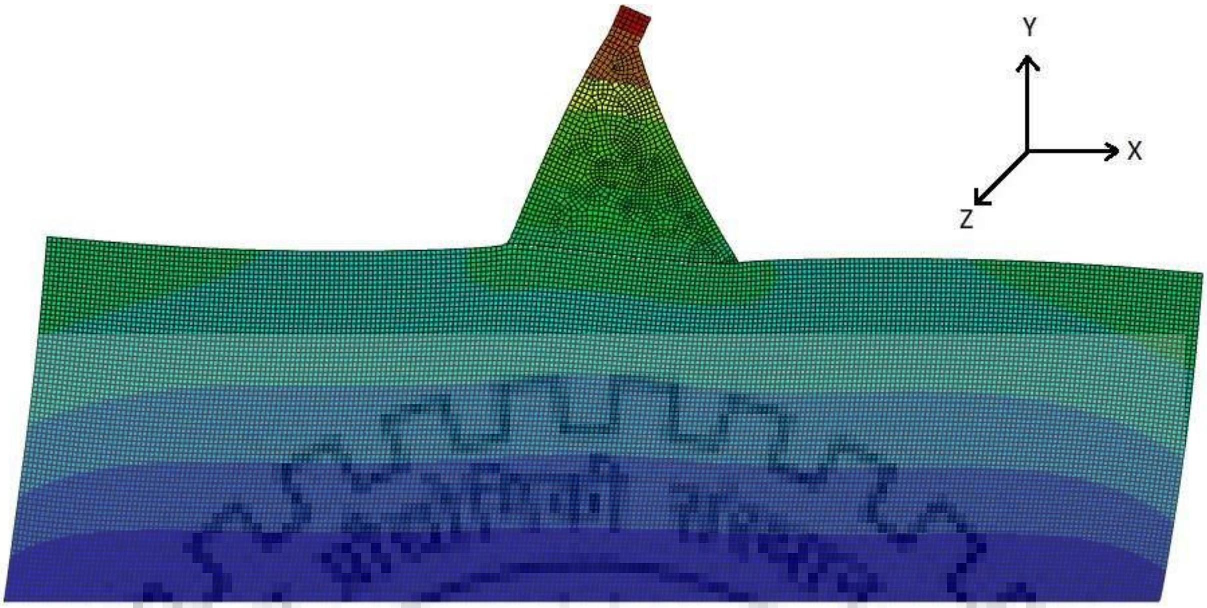

Figure 4.9: First mode shape

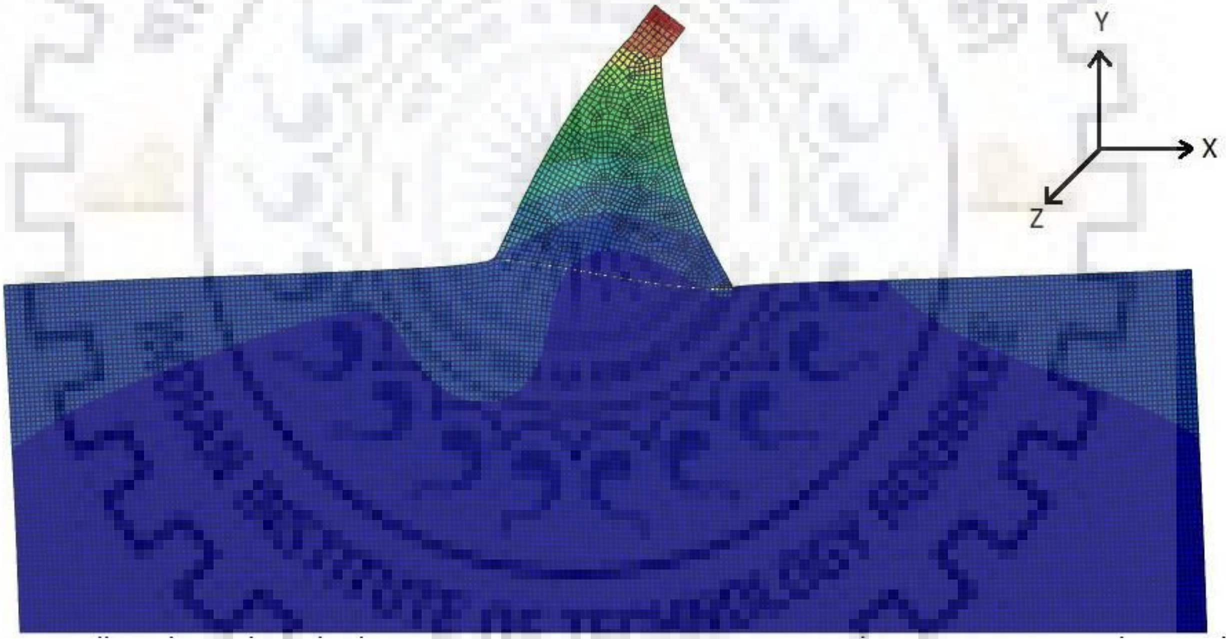

Figure 4.10: Second mode shape

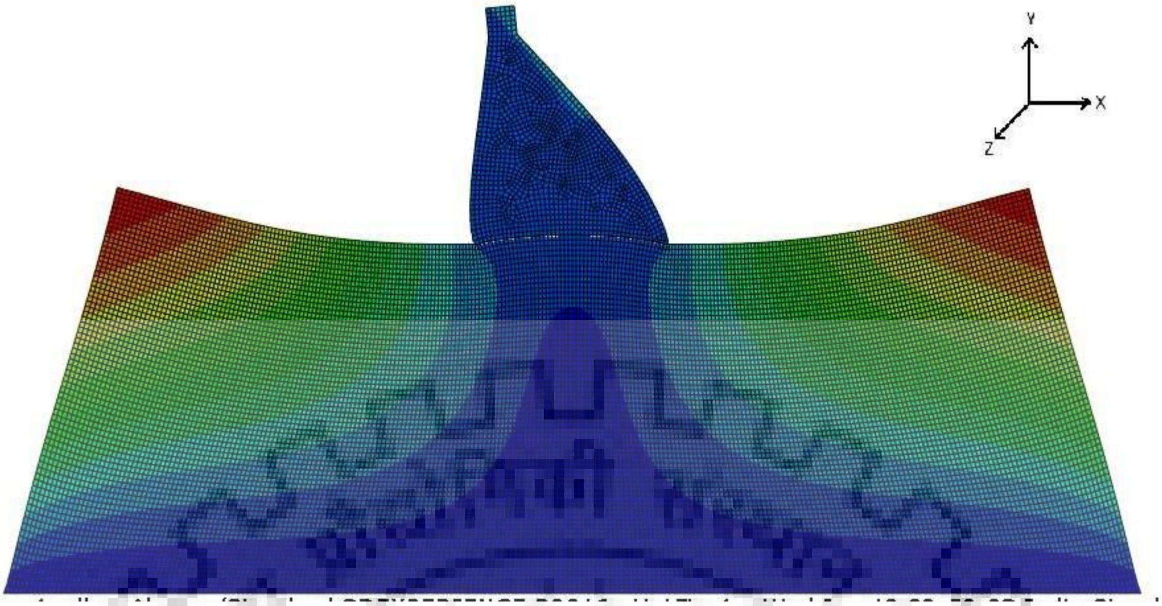

Figure 4.11: Third mode shape

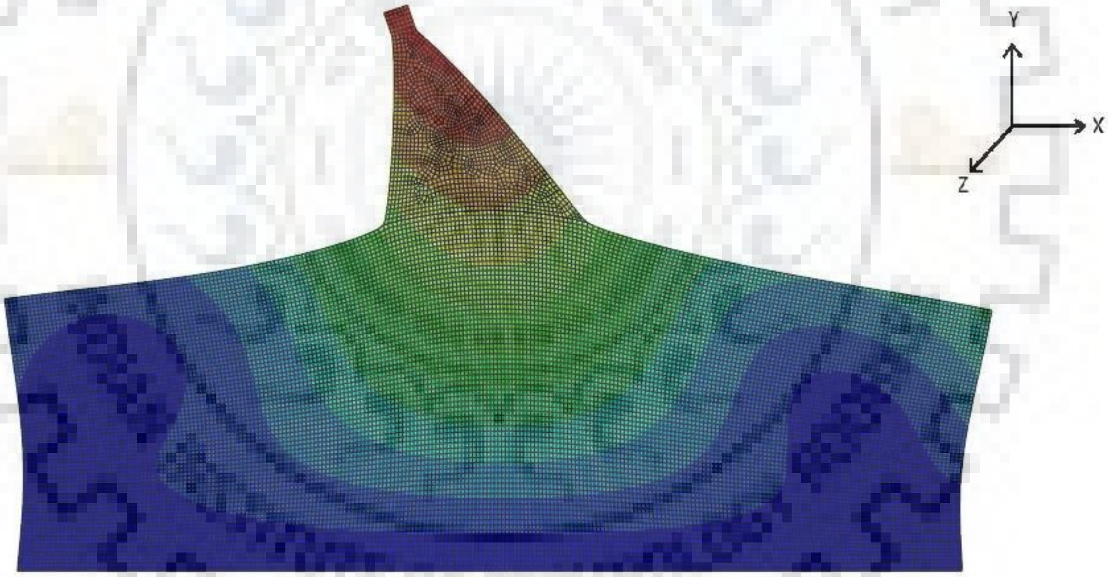

Figure 4.12: Fourth mode shape

Chapter-5

Result and Discussion

5.1 Equivalent Static Analysis

The force profile acting on the dam section are shown in figure [5.1]

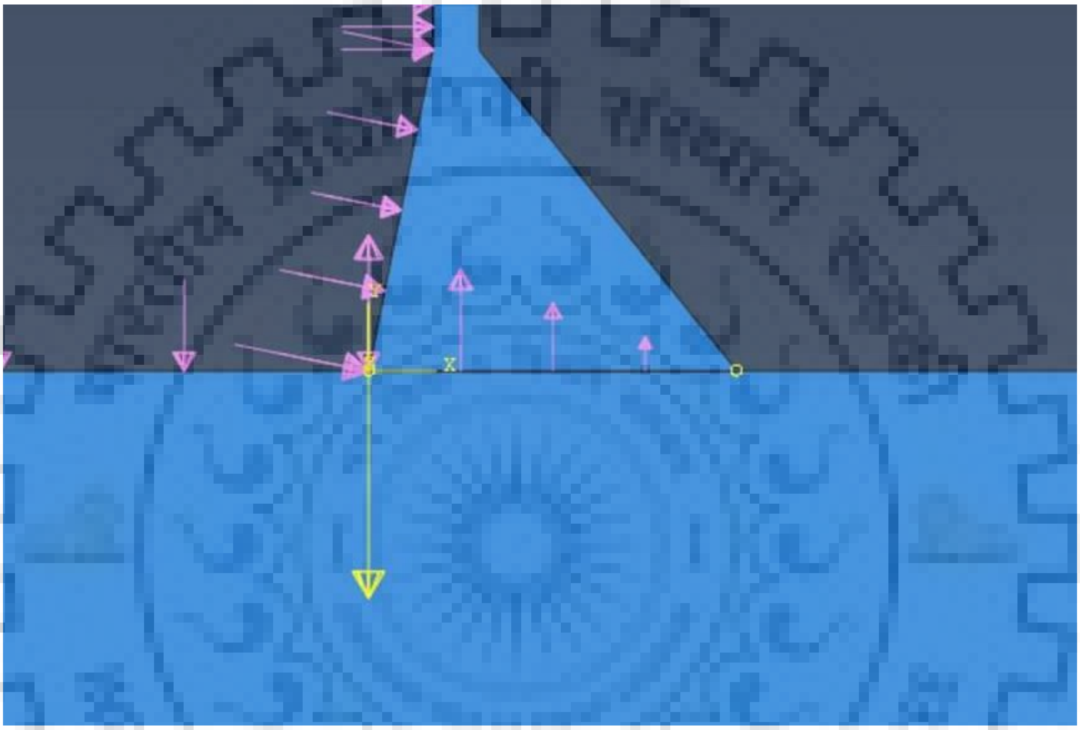

Figure 5.1: Static loads acting on my dam model

Static analysis has been carried out on the dam reservoir foundation system considering static loads gravity load, hydrostatic load, uplift press on the dam and weight of water on the foundation alone.

For static analysis, the boundary condition is taken as fixed at the foundation base and roller at the foundation sides.

The maximum principal stresses developed are in the heel and toe of the dam. The maximum principal stresses are listed down in the Table 5.1.

Table 5.1: Major Principal Stresses (MPa)

Toe	Heel
- 1.55	0.812

Here positive principle stresses are taken as tension and negative principle stresses as compression. Analysis result of it can be seen in figure 5.2.
The major principle stresses in heel and toe is at the nodes 621 and 810 respectively. The values developed are well within the limiting static tensile strength of 3.0 MPa calculated as per Raphael (Raphael, 1984).

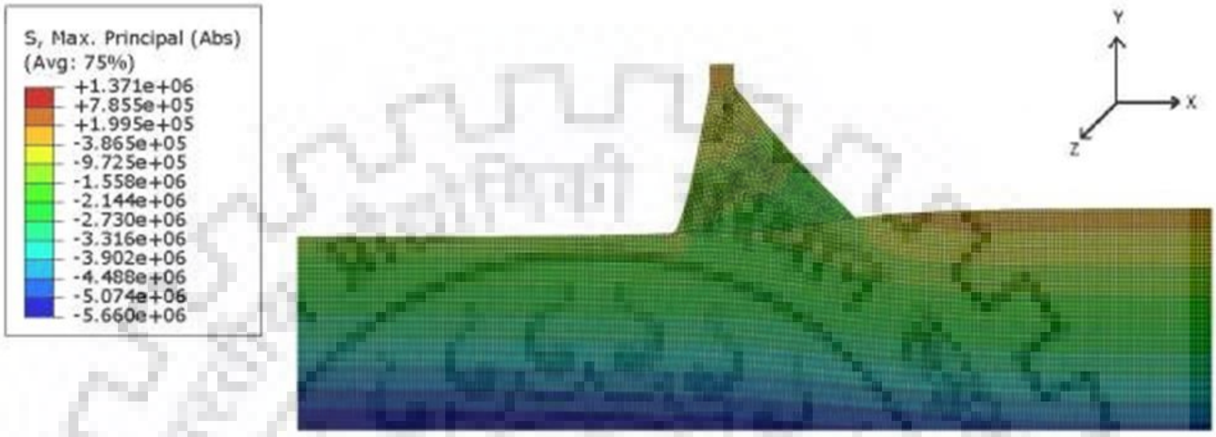

Figure 5.2: Principal stresses after initial static analysis

5.2 Time History analysis

5.2.1 Input ground motion

The time history earthquake data helps us compute deformation, stresses etc. more accurately by considering time dependent nature of dynamic response to earthquake ground motion. The near field and far field earthquake data has been obtained from PEER database(https://ngawest2.berkeley.edu/). Total of 16 earthquakes have been selected 8 of them being near field and other 8 being far field.
The near field data being selected is having an apparent velocity pulse with pulse duration being more than 1 sec. Also in addition to that ratio of PGV/PGA was checked to be more than 0.1 second. The far field data in was selected from same site condition and same earthquake but with epicenter being at larger distance.
The data selected had the magnitude of $M_w > 6.5$, shear wave velocity around 800 m/s and fault distance was considered to be within 20 km for near fault and more than 20 km for far fault earthquakes with consideration of the PGV/PGA ration greater than and less than 0.1 respectively. Scaling of the ground motions were carried out using the provisions of ASCE.

The ground motions were selected after comparing their corresponding acceleration spectra with the target spectrum (IS Code Spectrum).

The ground motion data has been divided into two parts. First part shows the near fault data in table (5.3) and correspondingly second part shows the data of the far fault in table (5.2).

Table 5.2: For far fault ground motion

Record Sequence Number	Earthquake Name	Year	Station Name	Magnitude	Fault Distance (km)	Vs30 (m/sec)	PGA (m/sec2)	PGV (m/sec)	PGV/PGA (sec)
774	Loma Prieta	1989	Hayward City Hall - North	6.93	55.11	735.44	2.21518	0.16007	0.07226
1109	Kobe	1995	MZH	6.9	70.26	609	2.2876	0.15152	0.066235
1256	Chi-Chi	1999	HWA002	7.62	56.93	789.18	1.9015	0.165517	0.087045
1613	Duzce	1999	Lamont 1060	7.14	25.88	782	2.2557	0.17624	0.078131
1767	Hector Mine	1999	Banning - Twin Pines Road	7.13	83.43	667.42	2.9009	0.1914	0.06598
3955	Tottori	2000	SMNH11	6.61	40.08	670.73	2.13799	0.1372	0.064172
4858	Chuetsu-oki	2007	Tokamachi Chitosecho	6.8	30.65	640.14	2.2646	0.1675	0.073964
5791	Iwate	2008	Maekawa Miyagi Kawasaki City	6.9	74.82	640.14	2.7443	0.12955	0.047207

Table 5.3: For near fault ground motion

Record Sequence Number	Earthquake Name	Year	Station Name	Magnitude	Fault Distance (km)	Vs30 (m/sec)	PGA (m/sec2)	PGV (m/sec)	PGV/PGA (sec)
763	Loma Prieta	1989	Gilroy - Gavilan Coll	6.93	9.96	729.65	2.4198	0.25139	0.103888751
1111	Kobe	1995	Nishi-Akashi	6.9	7.08	609	2.0843	0.2858	0.137120376
1507	Chi-Chi	1999	TCU071	7.62	5.8	624.85	1.5981	0.2611	0.163381516
1618	Duzce	1999	Lamont 531	7.14	8.03	638.39	1.7847	0.3274	0.183448199
1787	Hector Mine	1999	Hector	7.13	11.66	726	1.5258	0.2626	0.172106436
3943	Tottori	2000	SMN015	6.61	9.12	616.55	2.2373	0.256	0.114423636
4876	Chuetsu-oki	2007	Kashiwazaki Nishiyamacho Ikeura	6.8	12.63	655.45	2.1893	0.2824	0.128991002
5618	Iwate	2008	IWT010	6.9	16.27	825.83	1.9004	0.22762	0.119774784

The Amplitude scaling method according to ASCE 7-16 recommends that the average of the acceleration spectrum of all the earthquakes should be greater than 0.9 times that of the target acceleration spectrum. The current target spectrum is the IS code spectrum for MCE (Maximum Credible Earthquake) for zone IV and soil type I given by IS 1893-2016. The ground motions of both near fault and far fault were chosen carefully were chosen separately according to the provisions also that they meet this criterion in between the periods of interest of the Dam, as shown in figures below.

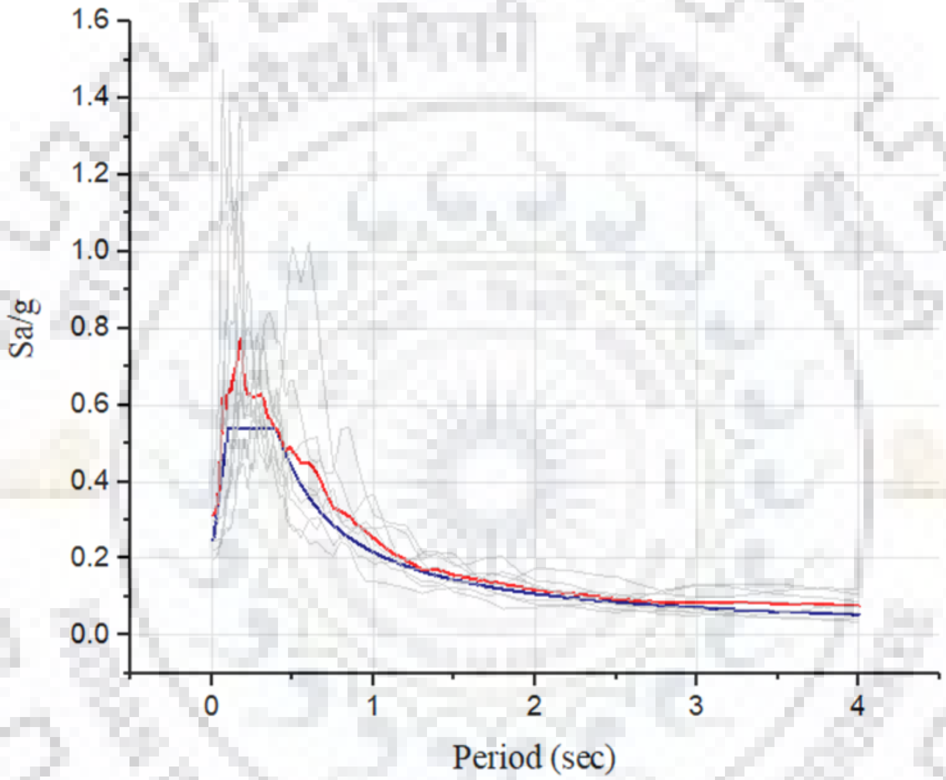

Figure 5.3: Far field average response spectrum

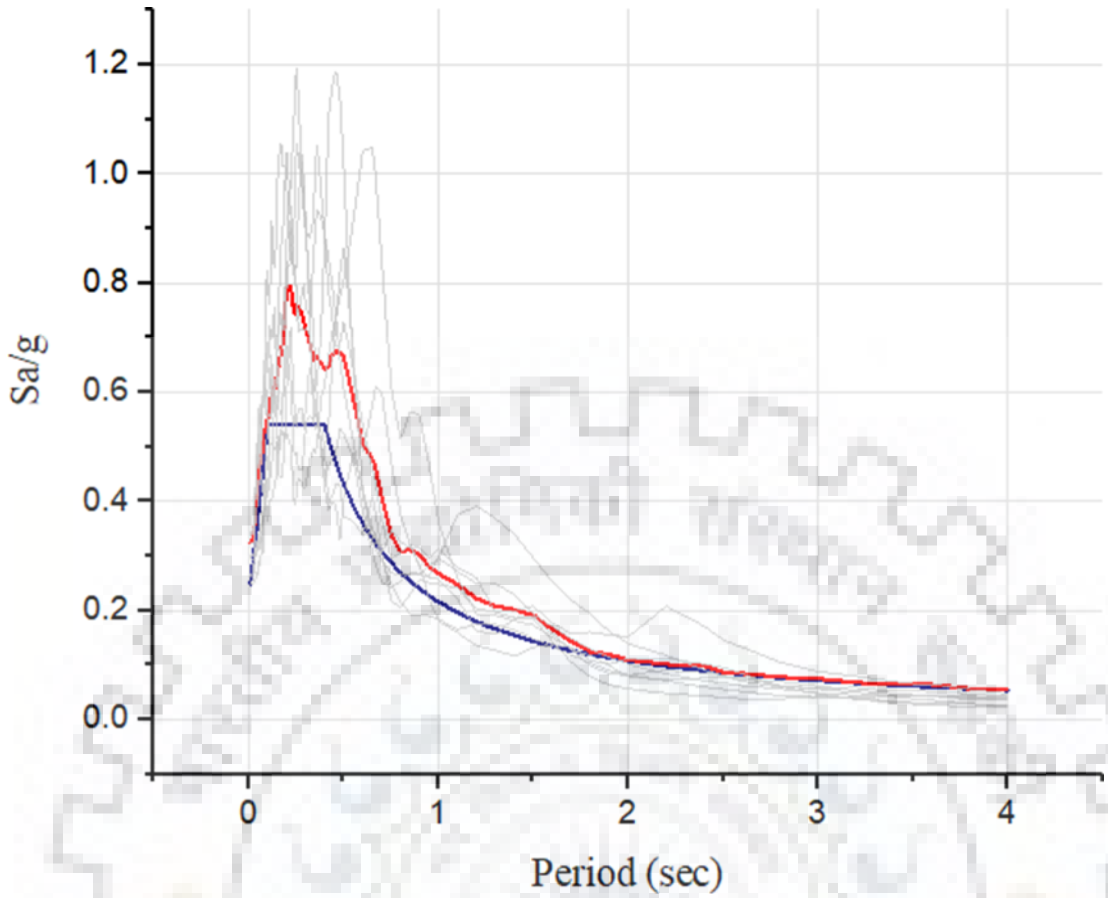

Figure 5.4*:* Near field average response spectrum

We can see from the above plots that the average of earthquakes considered are above the 90 percent of target acceleration spectrum.

5.2.2 Time History Results

The following elements were considered for performance assessment

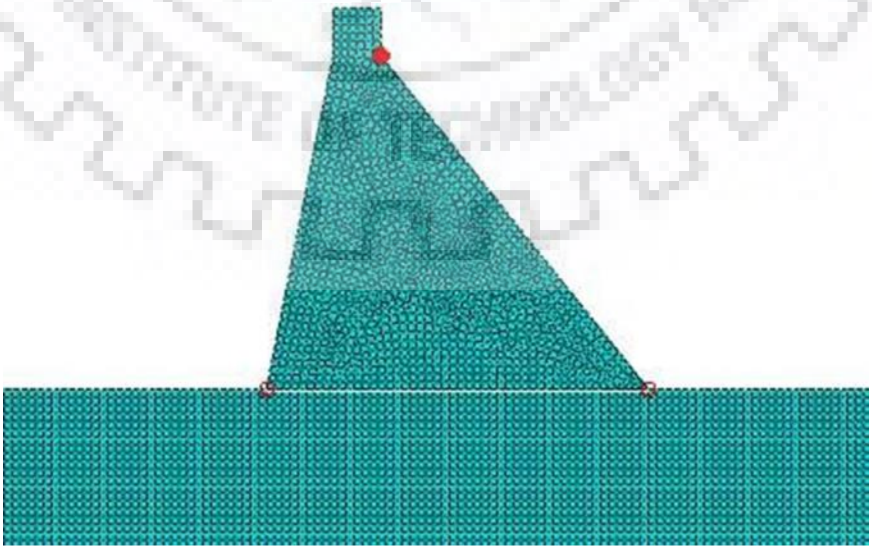

Figure 5.5: Nodes considered for performance assessment

The time history results for stresses were analyzed for 3 nodes as shown above but after analyzing the maximum stresses for above the above critical nodes it was observed that cracking initiates and propagates at toe and heel of the dam so only these two nodes were considered for further comparative study. The plot of time history for stresses at their two nodes are shown for different earthquakes comparing both near field and far field earthquakes.

The results shown indicate that the stresses in the node 621(heel) and 810(toe) exceed the DCR = 1 and 2 values. DCR 1 corresponds to the tensile strength of 3 MPa and DCR = 2 corresponds to the apparent dynamic tensile strength of 6 MPa in accordance to Raphael (Raphael, 1985).

The dam model was analyzed for the combined effect of static and seismic loads. The static loads include gravity, hydrostatic considering full reservoir level, uplift to zero tail water level and also hydrodynamic effect was considered using lumped mass approach. Uplift pressure is assumed to be not changing during the application of seismic loading.

Analysis results consist of principle stress, time history of stresses comparing the near field and far field data in indication with Demand Capacity Ratio (DCR) limits. It is the ratio of principle stress to dynamic tensile stress of concrete.

5.2.3 Principal Stress

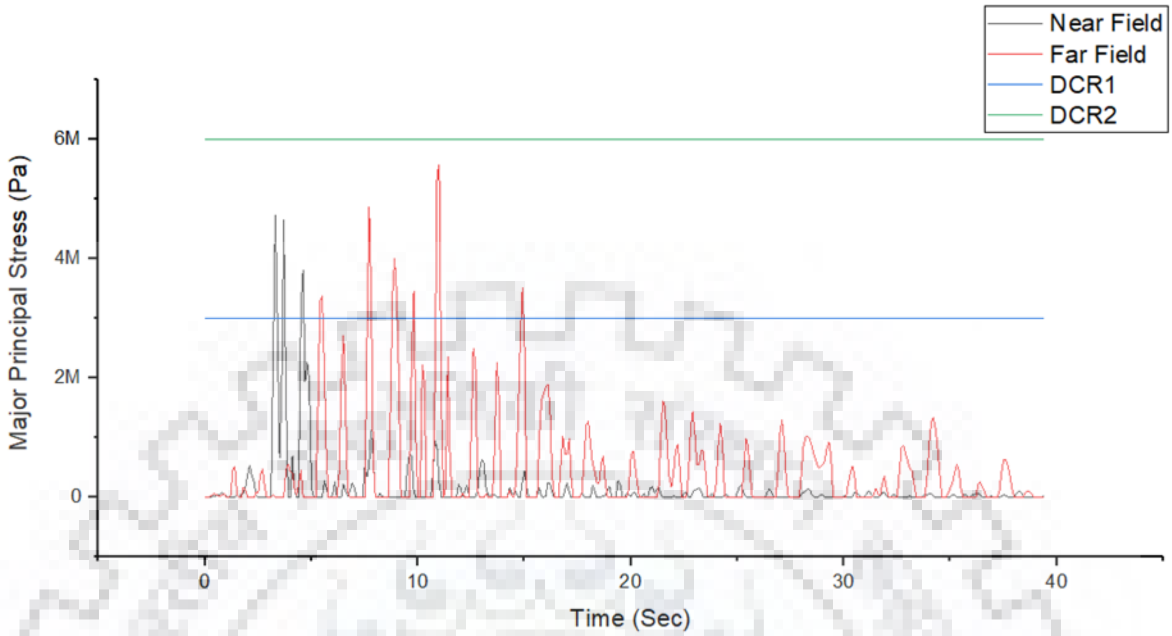

Figure 5.6: Stress time history for Loma Preita earthquake at N 621

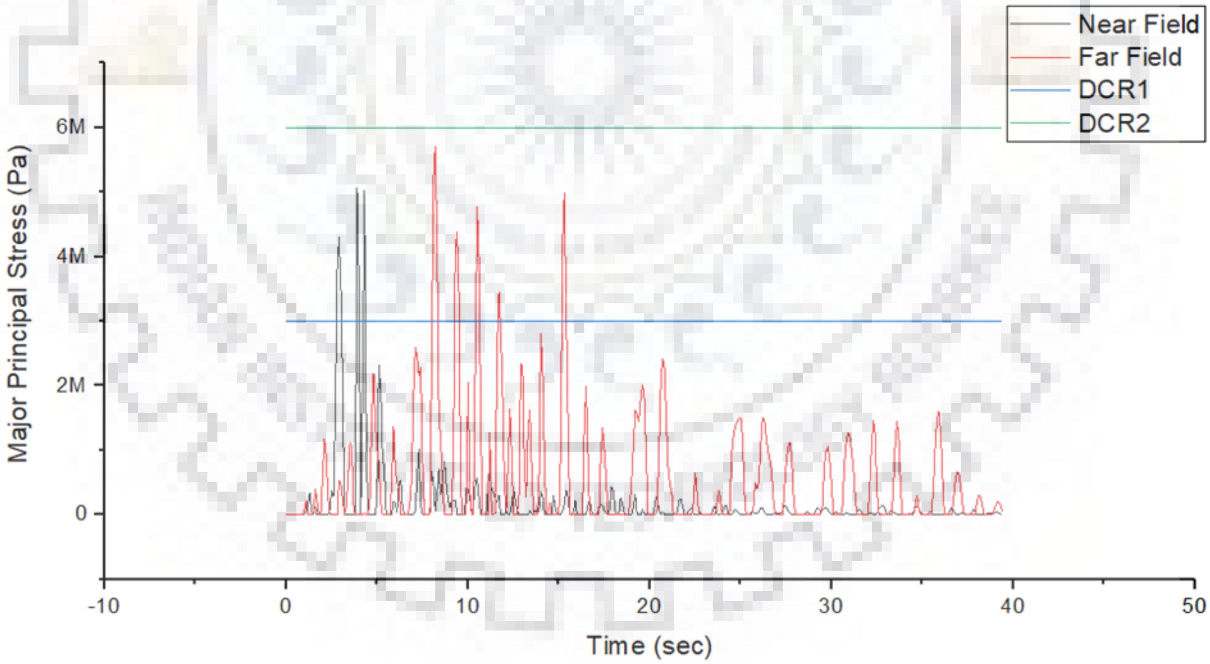

Figure 5.7: Stress time history for Loma Preita earthquake at N810

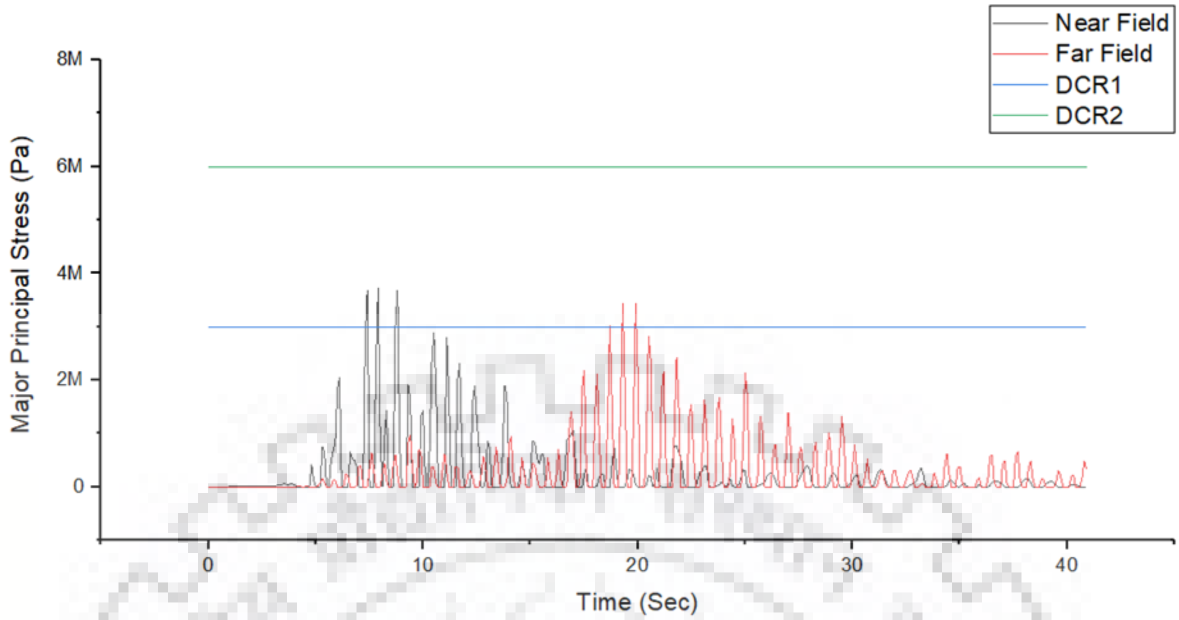

Figure 5.8: Stress time history for Kobe Japan earthquake at N 621

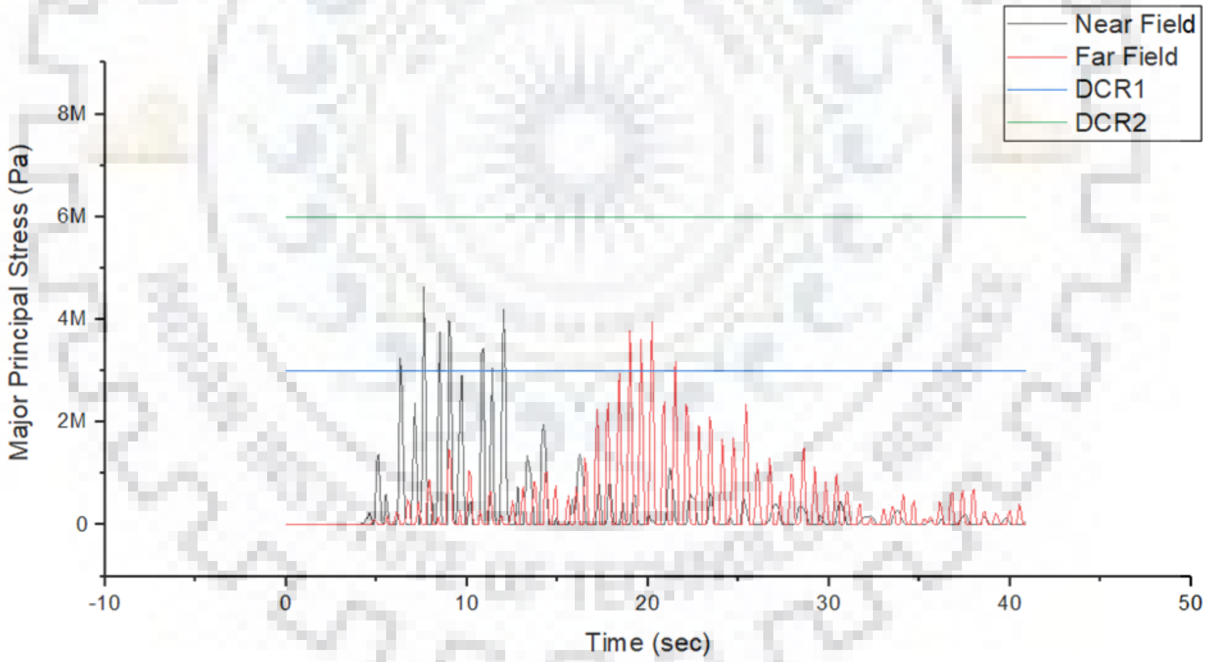

Figure 5.9: Stress time history for Kobe Japan earthquake at N 810

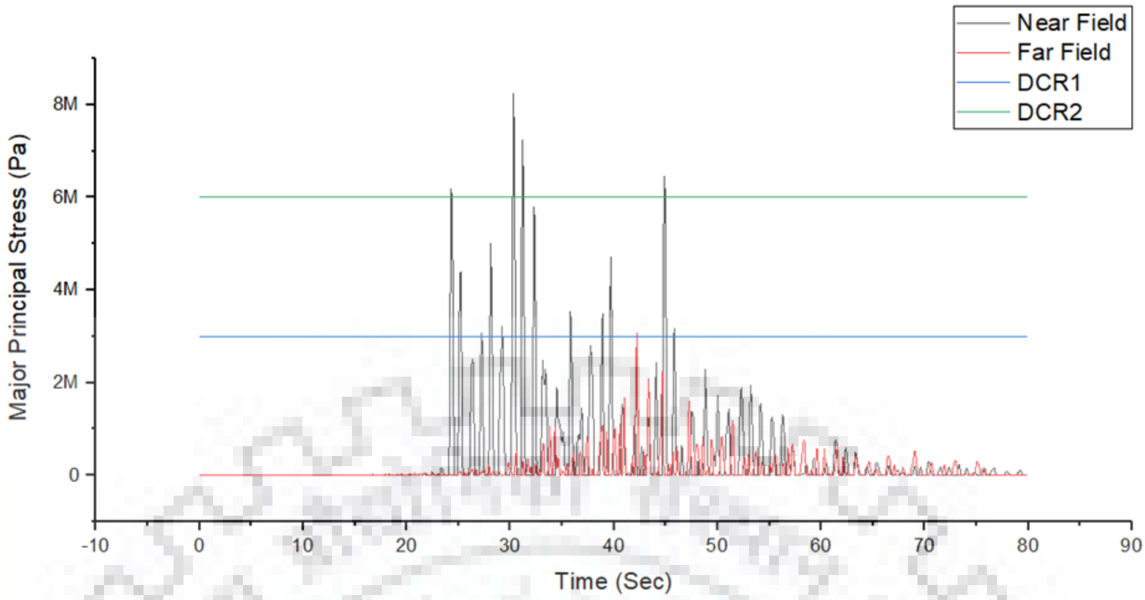

Figure 5.10: Stress time history for Chi Chi Taiwan earthquake at N 621

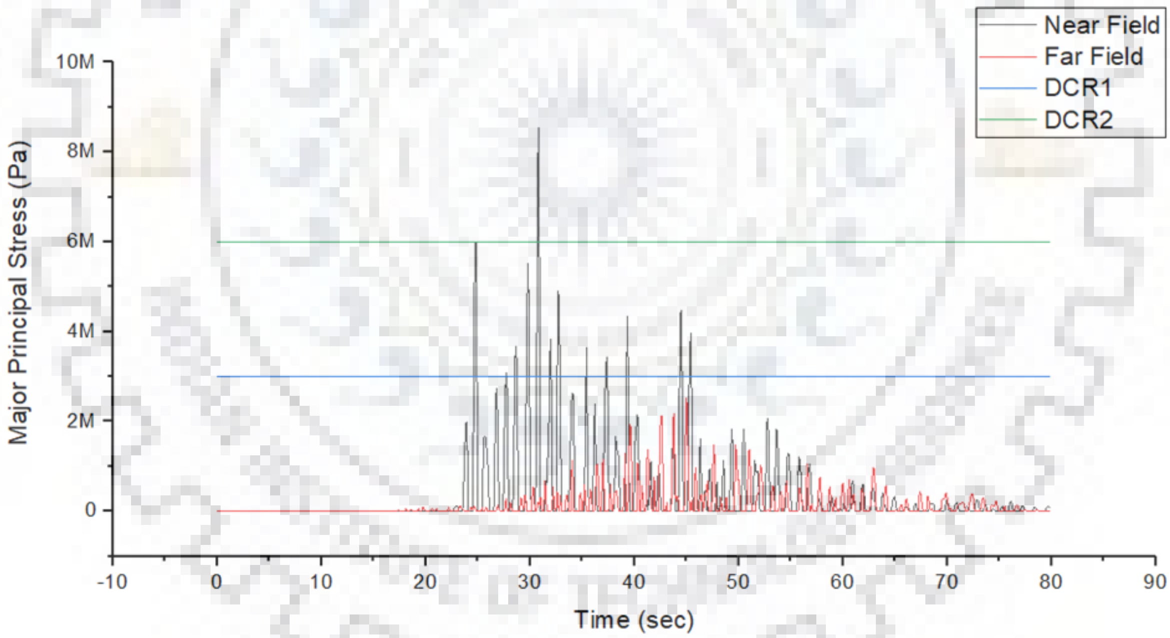

Figure 5.11: Stress time history for Chi Chi Taiwan earthquake at N 810

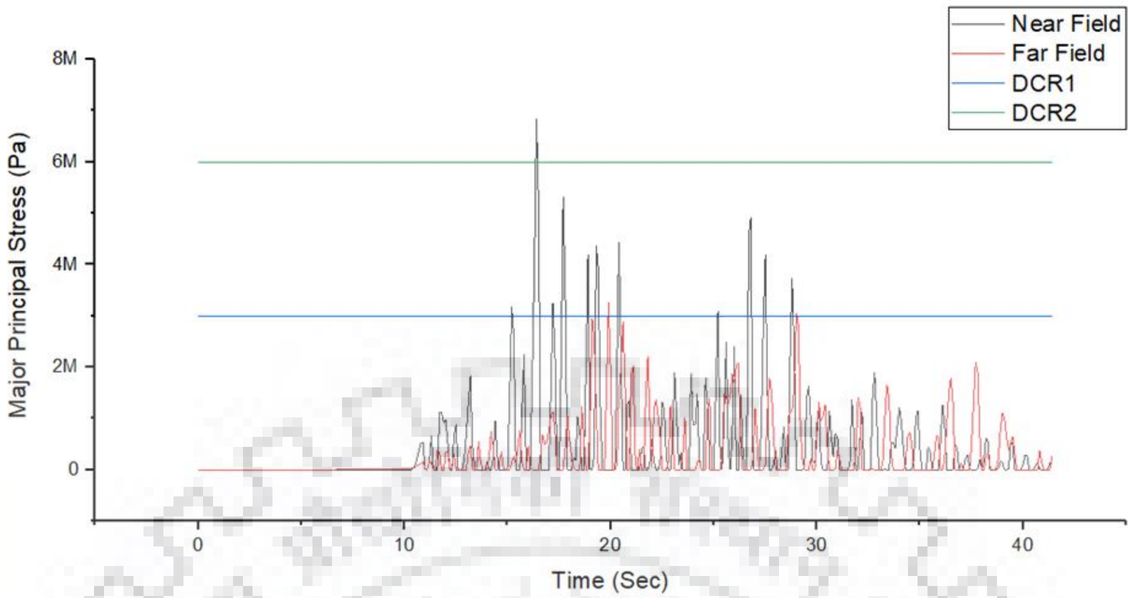

Figure 5.12: Stress time history for Duzce Turkey earthquake at N 621

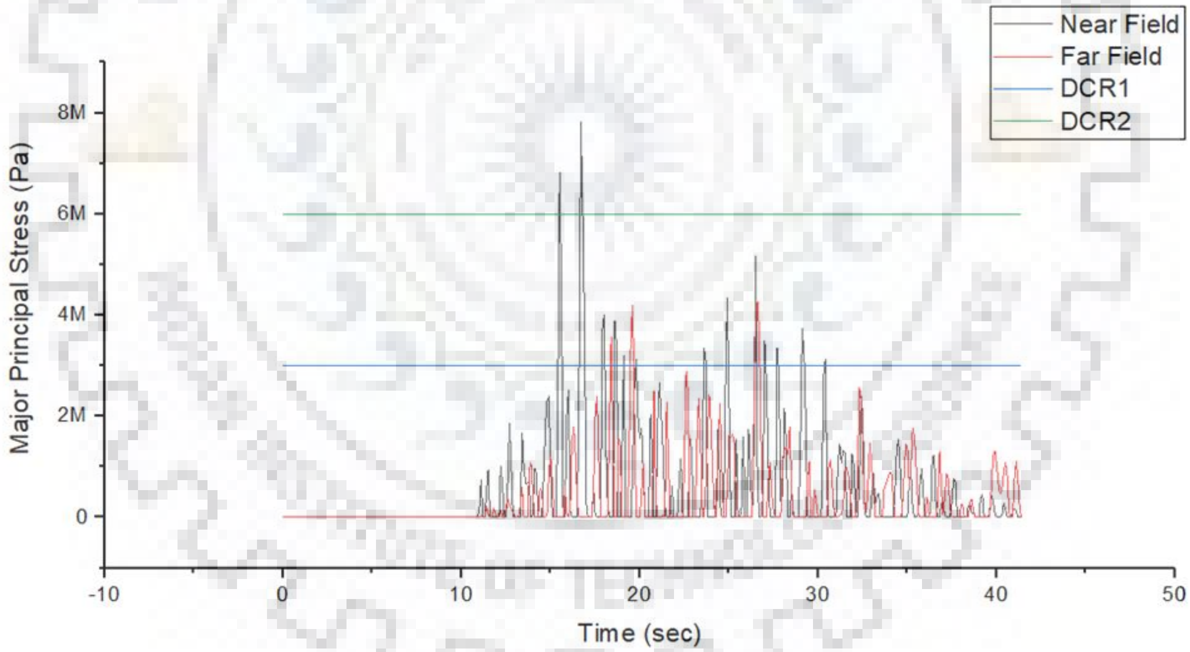

Figure 5.13: Stress time history for Duzce Turkey earthquake at N 810

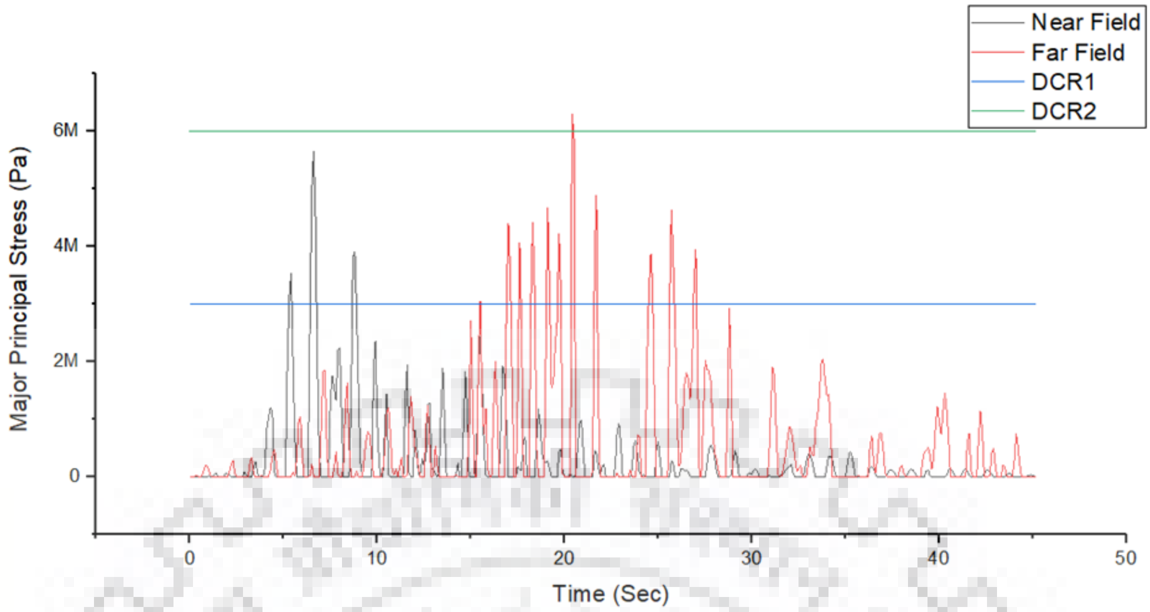

Figure 5.14: Stress time history for Hector Mine earthquake at N 621

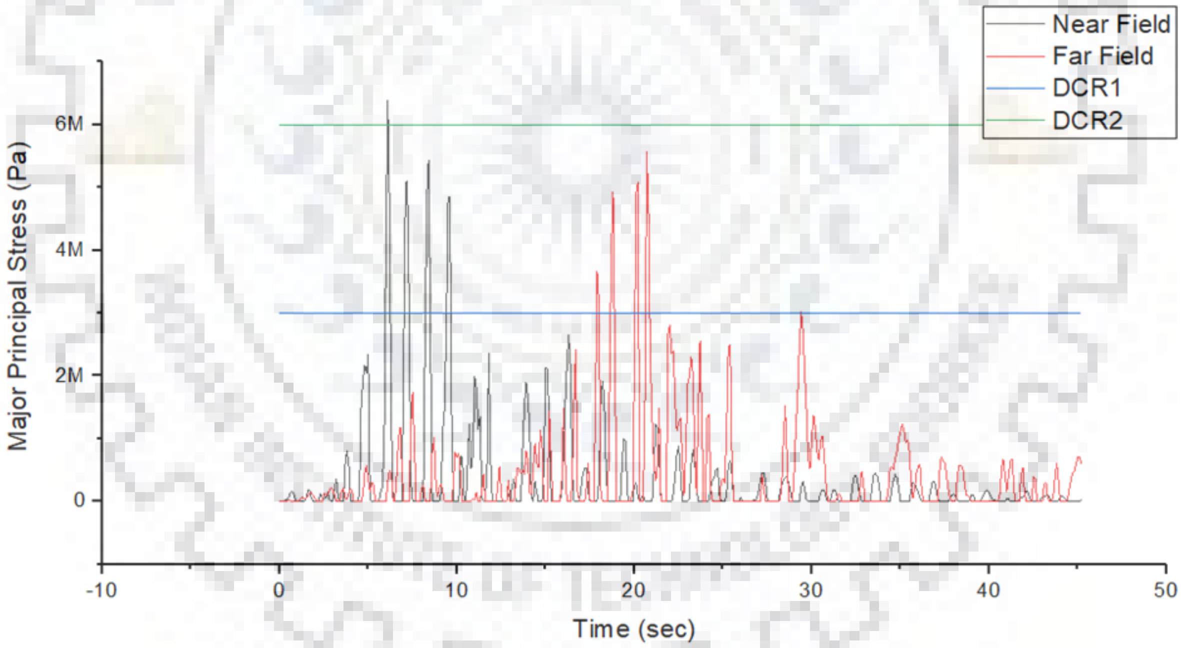

Figure 5.15: Stress time history for Hector Mine earthquake at N 810

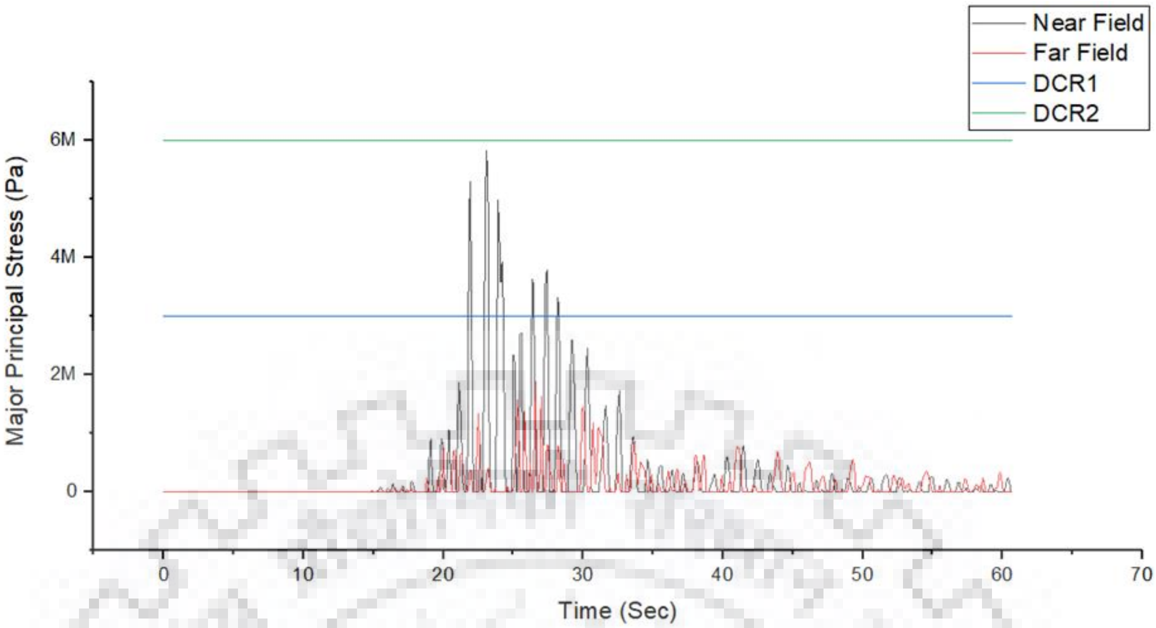

Figure 5.16*:* Stress time history for Tottori Japan earthquake at N 621

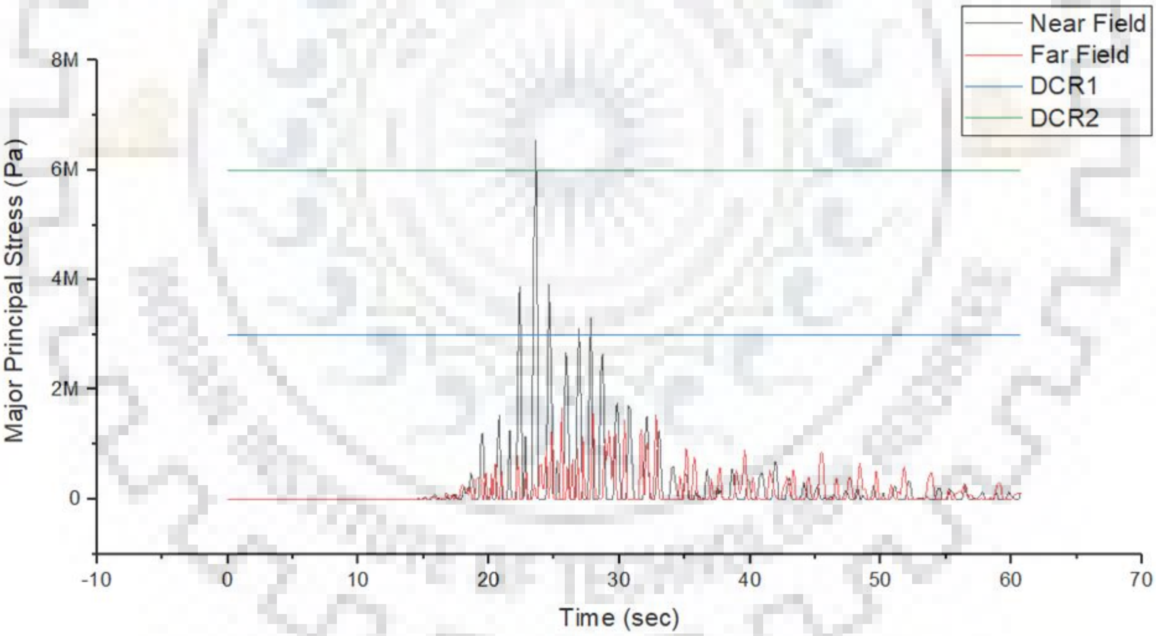

Figure 5.17*:* Stress time history for Tottori Japan earthquake at N 810

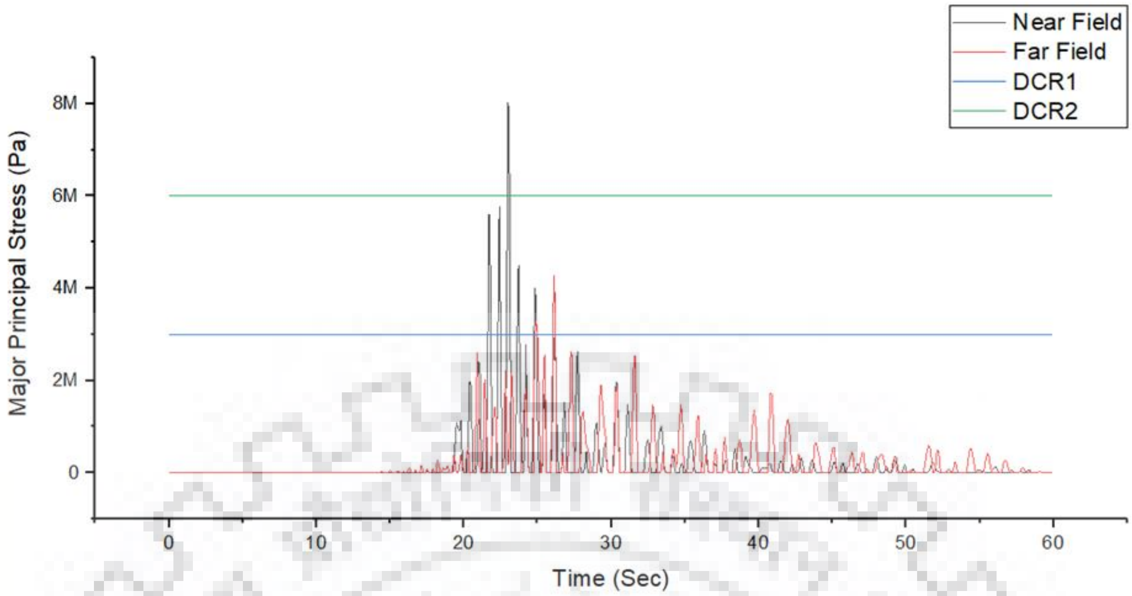

Figure 5.18: Stress time history for Chuetsu-oki Japan earthquake at N 621

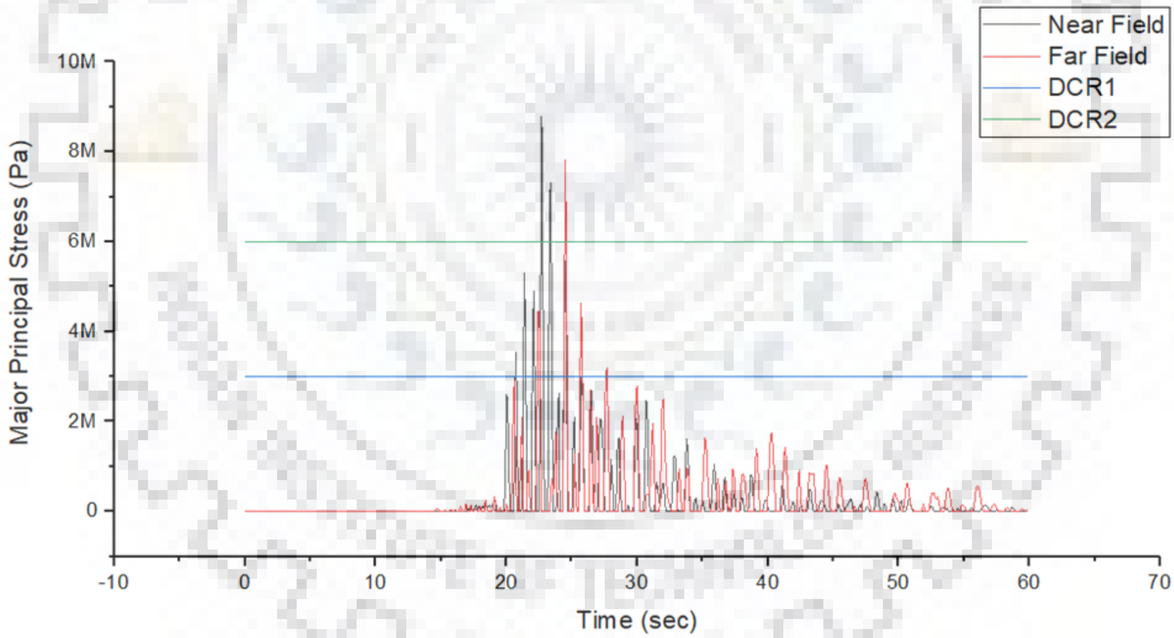

Figure 5.19: Stress time history for Chuetsu-oki Japan earthquake at N 810

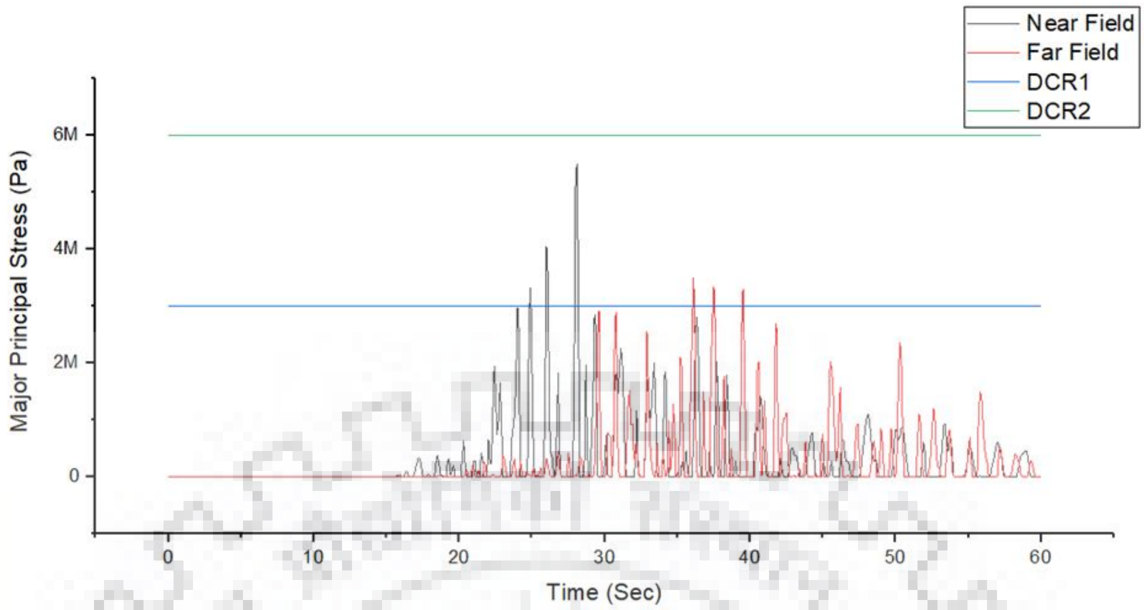

Figure 5.20: Stress time history for Iwate Japan earthquake at N 621

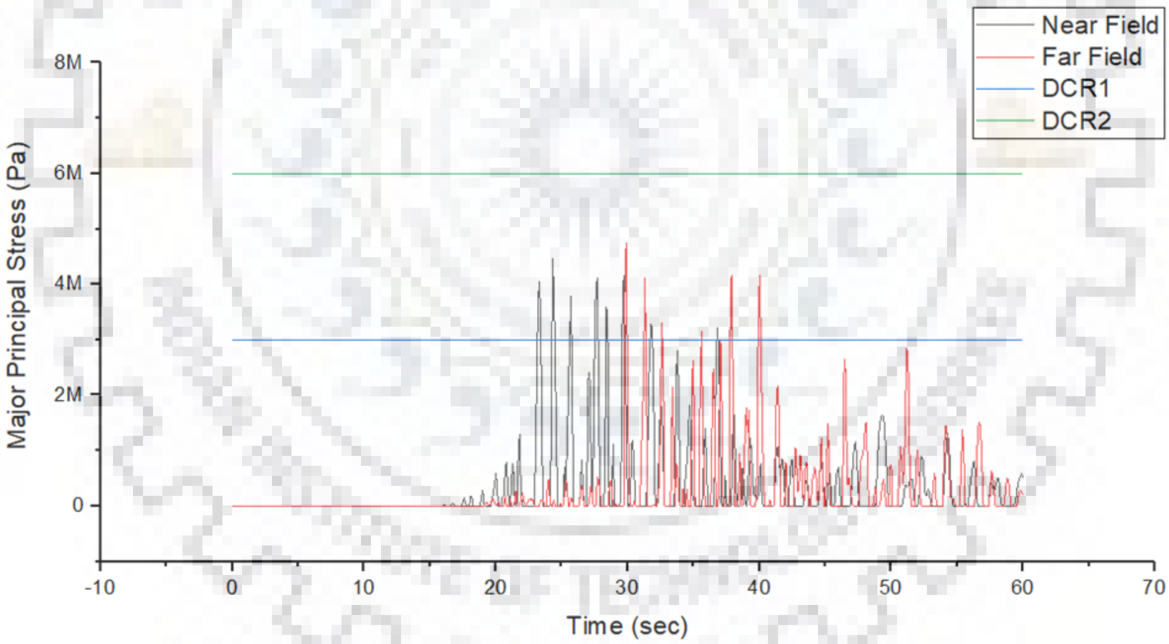

Figure 5.21: Stress time history for Iwate Japan earthquake at N 810

It can be observed from the above plots that exceedance of stress cycle above DCR = 1 and DCR = 2 is more in near fault earthquake than far fault earthquake. Also, stress obtained from Near fault earthquake is more as compared to far fault earthquake for both toe and heel and occurs at the beginning of motion in accordance with the fact the near fault motions have pulse like effect at the starting of the motion.

5.2.4 Cumulative Inelastic Duration Curve

Further performance assessment has been analyzed using DCR vs cumulative inelastic duration curve. Cumulative inelastic duration is the total time of stress exceedance in the time history of stresses above the particular DCR level.

The comparison of cumulative inelastic duration of nodes for different earthquakes is shown separately.

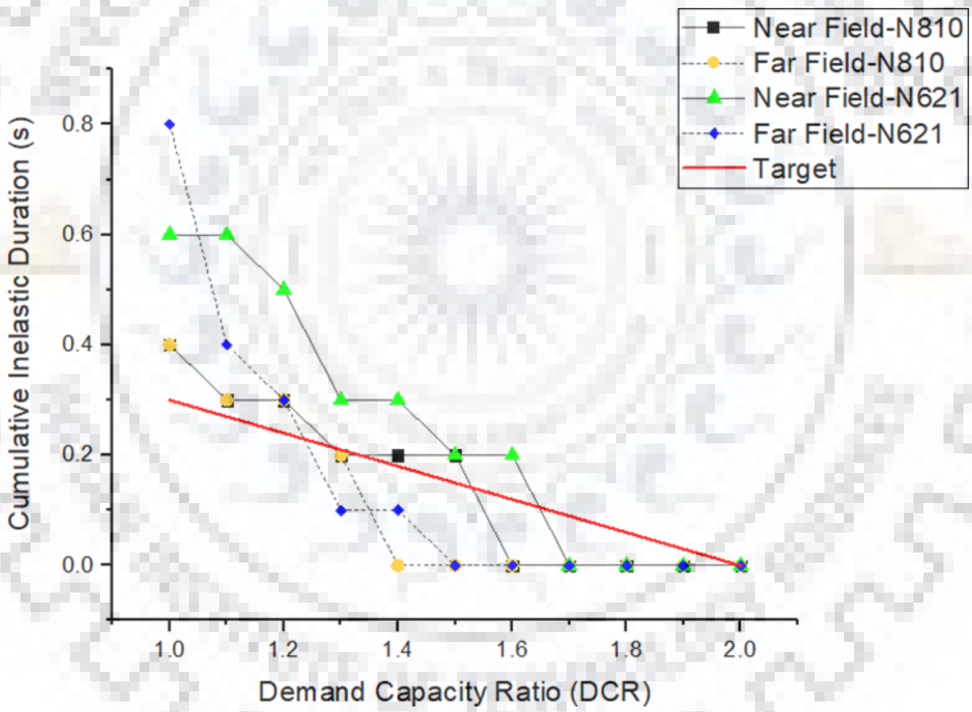

Figure 5.22*:* Cumulative Inelastic Duration curve for Loma Preita earthquake

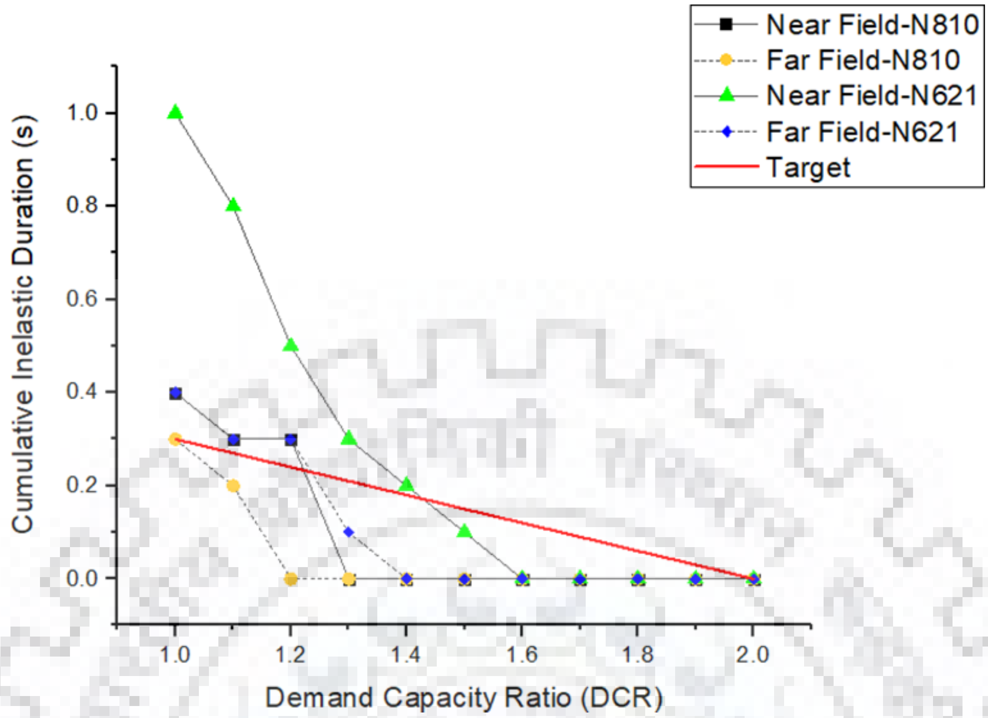

Figure 5.23: Cumulative Inelastic Duration curve for Kobe Japan earthquake

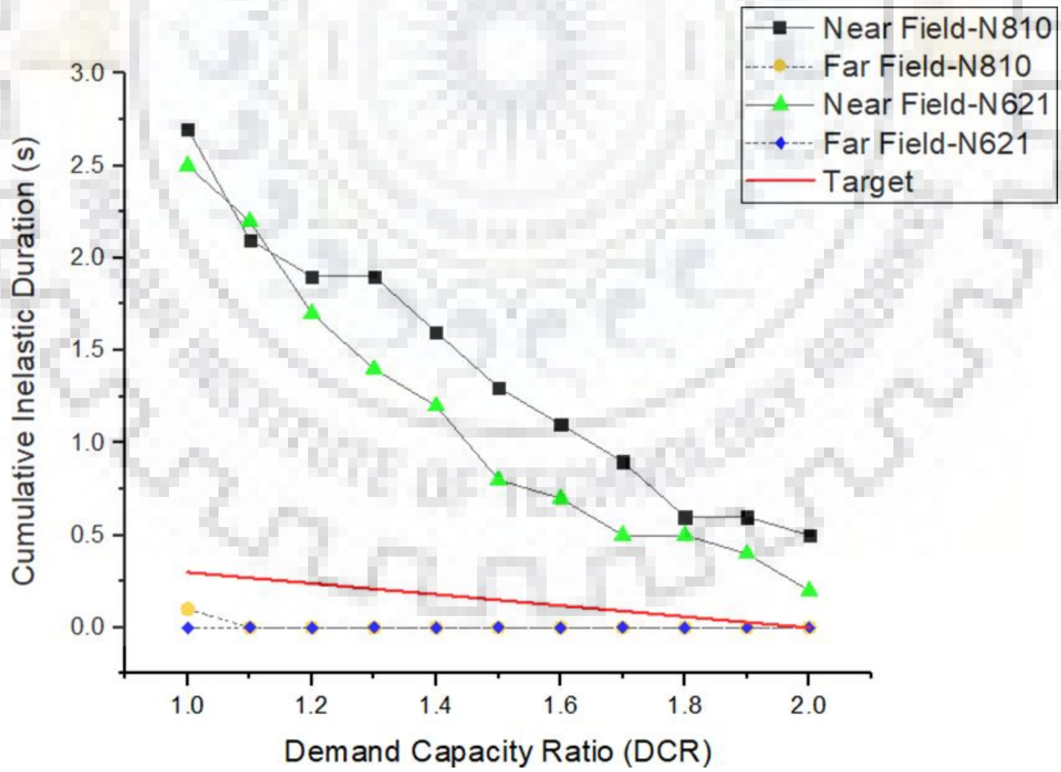

Figure 5.24: Cumulative Inelastic Duration curve for Chi Chi Taiwan earthquake

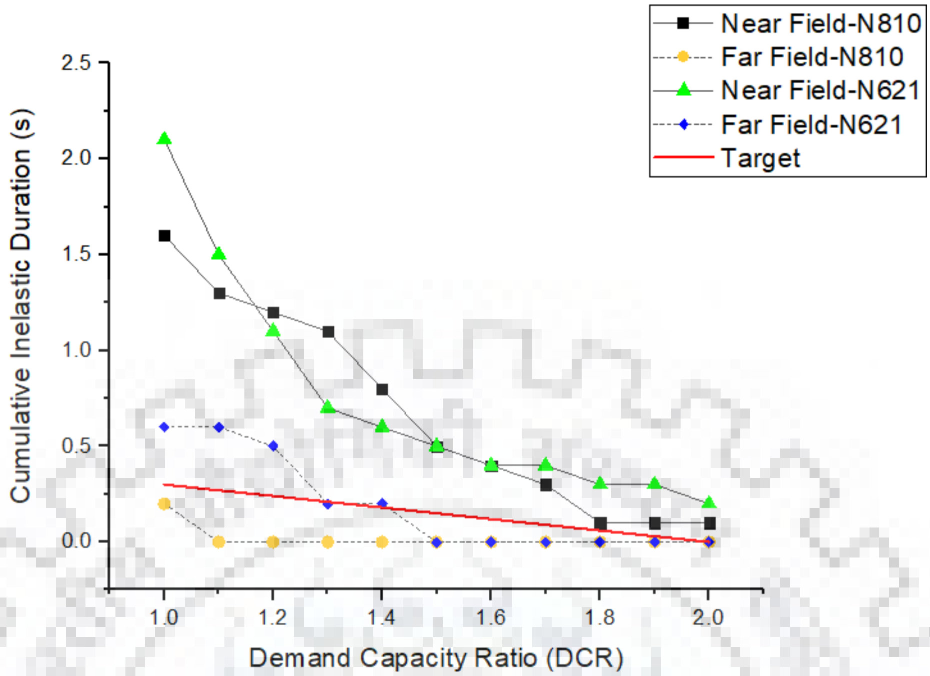

Figure 5.25*:* Cumulative Inelastic Duration curve for Duzce Turkey earthquake

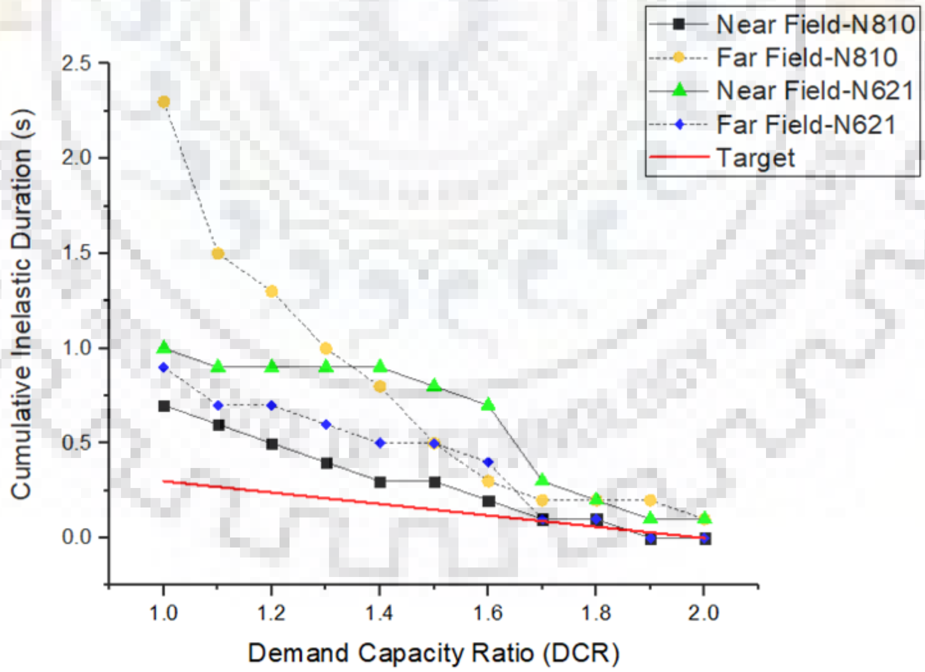

Figure 5.26*:* Cumulative Inelastic Duration curve for Hector Mine earthquake

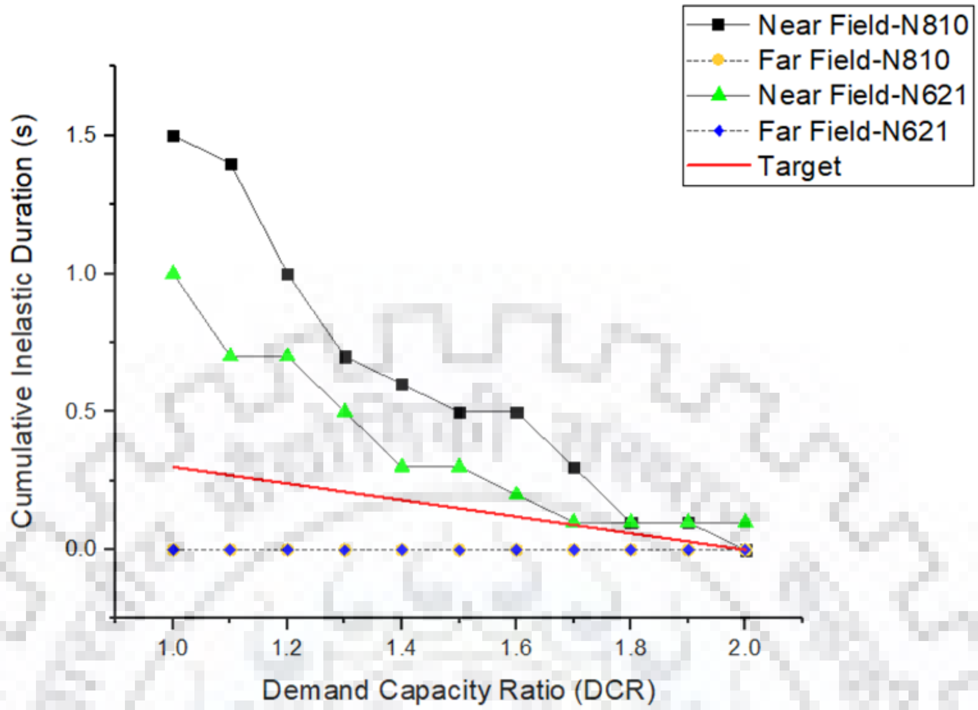

Figure 5.27: Cumulative Inelastic Duration curve for Tottori earthquake

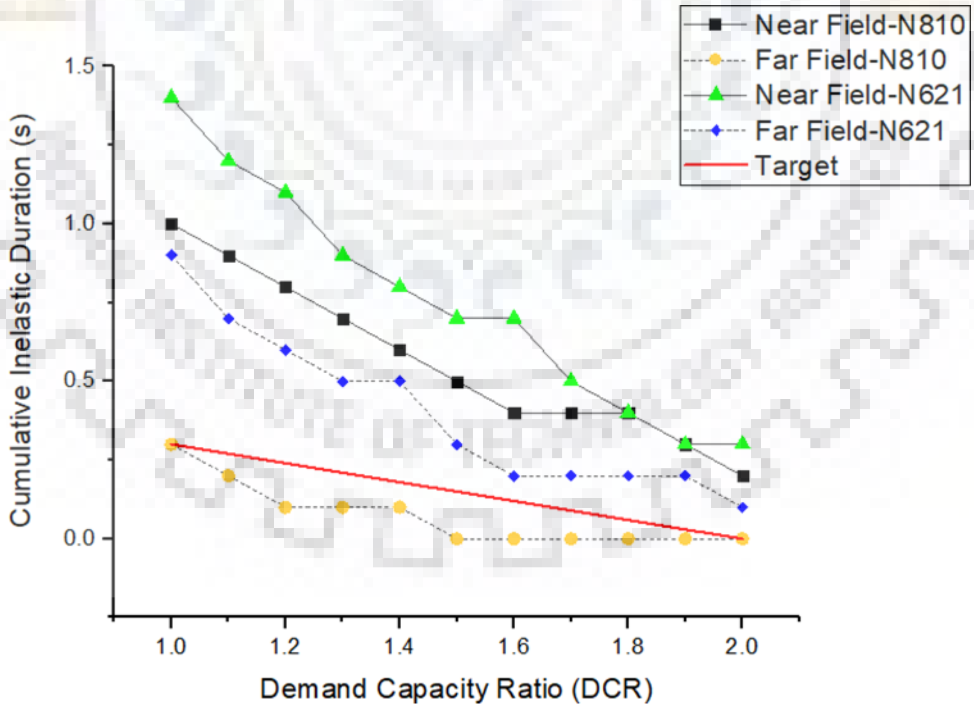

Figure 5.28: Cumulative Inelastic Duration curve for Chuetsu-oki earthquake

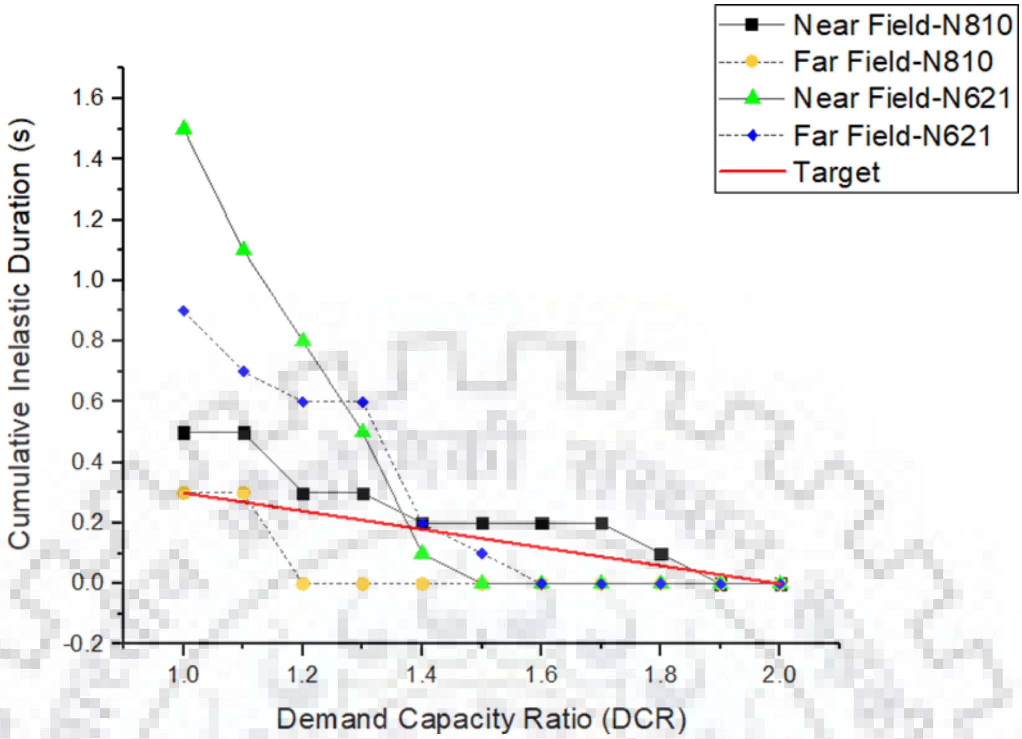

Figure 5.29: Cumulative Inelastic Duration curve for Iwate earthquake

The above plots show that the stresses at nodes 621 and 810 at heel and toe respectively are acceding the acceptable limits of cumulative inelastic duration. This suggest that cracking may initiate at heel and toe of element and propagate through body of the dam.

The DCR values are going well beyond the limit for near fault earthquakes for both toe and heel, also it can be seen that far fault results are within limit in some cases and might not cause failure to any node of our dam.

Some tensile cracking may occur but there is no possibility of failure as the number of overstress nodes are very less.

Hector Mine and Loma Preita earthquakes at heel have larger value for far fault earthquake near DCR=1, but after analyzing their principle stress results in ABAQUS, the area of overstress region was more in case of near fault than far fault.

Chapter-6

Conclusion

Series of linear analysis have been performed on the dam foundation system and time history of stresses are plotted. Modelling of the dam was carried out on ABAQUS software for plain strain condition and total of 16 ground motions were applied as input excitation. DCR limits with cumulative inelastic duration for various critical elements in the dam are plotted and following are the conclusions.

1. Linear elastic time history analysis was used to observe the dynamic behavior of the dam under near fault and far fault ground motions.
2. The results of linear analysis are used to identify the potential failure modes of the dam.
3. The results obtained are compared with the damage provision according to Ghanaat (Ghanaat, 2004). The results showed that the dam would experience cracking at the toe and heel portions of the dam. After comparing the effect of near and far fault earthquake on stress and cumulative inelastic duration curves, it can be concluded that near fault is more effective and causes more damage than far fault earthquake and should be taken under consideration for more realistic result.
4. Performance assessment show that seismic response of concrete gravity dam is more under near fault earthquake because of their severe and impulsive effect on structures.
5. Since the post-earthquake stability of the dam was not carried out, a quantitative estimate of stability of the dam for post-earthquake condition is also necessary to ascertain and support the performance evaluated for the present system.
6. The results showed that the dam would experience cracking at the toe and heel portions of the dam and it required to further proceed to non-linear analysis to better understand the behavior of concrete gravity dam.

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