ANALYSIS OF HIGH EMBANKMENT DAM USING FINITE ELEMENT METHOD

A DISSERTATION

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

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in

WATER RESOURCES DEVELOPMENT

By

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CANDIDATE'S DECLARATION

I do hereby declare that the dissertation entitled "**ANALYSIS OF HIGH EMBANKMENT DAM USING FINITE ELEMENT METHOD**" is being submitted by me in partial fulfillment of requirement for the award of degree of "**Master of Technology** in **WATER RESOURCES DEVELOPMENT (CIVIL)**" and submitted in the Water Resources Development and Management Department, Indian Institute of Technology, Roorkee, is an authentic record of my own work carried out during the period from July, 2004 to June 2005 under the guidance of **Dr. B.N. Asthana**, Visiting professor, **Prof. Gopal Chauhan,** Professor, Water Resources Development and Management Department, and **Dr. Pankaj Agarwal,** Assistant Professor, Earthquake Engineering Department, Indian Institute of Technology, Roorkee.

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

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

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SYNOPSIS

In the past, embankment dams were preferred in wide valleys up to modest heights only. The advancement of soil mechanics ,evolution of earth and rockfill dams and improved design procedures have gradually lead to construction of high embankment dams in narrow gorges with steep side slopes, unfavourable foundation conditions and seismic areas. A number of dams higher than 200m have been constructed in the recent past. Detailed and comprehensive analysis of the dam duly considering all aspects into account is required for a safe structure.

Limit equilibrium approach is extensively used ignoring the deformability characteristics of the material and concentrating on the collapse mechanism. This method fails to evaluate the stress and deformation condition in the dam and could not take nonlinearity aspects of the soil into account. The Finite Element Method (FEM), at its present stage of development, is a powerful tool in the hands of dam designer. It enables the computation of stresses and strains within the dam, taking variation of properties of materials in different zones and foundations into account, as well as the effect of sequential construction. Nonlinear FEM approaches which establish the constitutive relation provide more realistic and accurate behaviour of the dam under static as well as dynamic loading conditions.

Core plays a vital role in earth and rockfill dams. Depending on the core position, they are named as central (vertical), moderately inclined and inclined cores. The stress and deformation distribution in the dam varies depending on the position and thickness of the core. A consensus exists on the relative advantages and disadvantages of different types of cores in some aspects but, the opinion differs on the other points such as susceptibility to cracking and suitability under seismic loading conditions etc.

The present work deals with the analysis of the earth and rockfill dam in general and influence of the core position on the static and dynamic response of the dam in particular. For the purpose of the present study three core positions have been considered keeping the amount of the fill in the core as same. Two dimensional plane strain finite element method has been adopted in the present study. Kulhawy_Duncan hyperbolic elastic model has been used to describe the properties of all the dam material.

The single lift and nonlinear multi lift analysis of central core dam have been carried out to study the effect of the staged construction and reservoir filling on the deformations and stress pattern in the dam. Nonlinear multi lift static analysis considering the construction schedule, reservoir filling stages, nonlinear material properties have been carried out for the three alternative sections to study the influence of the core shape. Further, free vibration analysis and linear dynamic analysis (Time History analysis) have also been performed for checking the suitability of the core geometry under the seismic loading condition.

The results of the single lift and multi lift analyses indicate that the vertical deformation patterns in the two cases are quite distinctive. However, horizontal deformation and principal stress pattern are of the same nature but the single lift analysis yielded higher stresses in comparison with the multi lift analysis.

As far as suitability of the core shape is concerned, the central core dam has advantage over the other two alternatives under static loading condition but the inclined core is found to be safer under dynamic loading conditions. The vertical core dam may be more appropriate in low seismic areas and inclined core dam may be adopted in highly seismic areas.

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Dam forms an important element of a multipurpose projects like hydroelectric, irrigation and flood control. The dam can be of one of the types i.e. embankment, masonry, concrete gravity, concrete arch dams etc. depending on the site conditions. In the past, the embankment dams were preferred in wide valleys up to modest heights only. The advancement of soil / rock mechanics ,evolution of rockfill dams with impervious core and improved design procedures have gradually lead to construction of embankment dams in gorges with steep side slopes, a remarkable example being 300 m high Nurek dam in Russia. There are at least ten embankment dams with earth cores over 200 m in height, completed, under construction or active planning in the world. Such dams are located in unfavourable foundation conditions and seismically active areas. It is, therefore, essential that dams should be designed taking all aspects into account including the anticipated earthquake forces.

In the past, limit equilibrium approach was extensively used in the analysis of the embankment dams. But, this method fails to evaluate the stress and deformation condition of the dam and could not take non linearity aspects of the material into account. The Finite Element Method (FEM) is a very versatile method as this method can be used to evaluate stress and deformation conditions of the dam under static and dynamic loading conditions also. The method has other unique capabilities as well ;these include the fact that it can be used for problems involving non-linear and non-homogeneous material, complex boundary conditions, sequential loading and so on.

An earth and rockfill dam has an impervious core sandwiched between the upstream and downstream shells. The purpose of the core is to check seepage through the body of the dam while rockfill provides free drainage and high shear strength for the dam. Filters or transition zones are provided between core and shell for releasing the hydraulic pressure in the core and to restrict the migration of fine particles of the core. The core is generally located at the centre or upstream from the central portion of the dam.

Core, depending upon the inclination, is categorised as vertical, moderately inclined and inclined cores. Both vertical and inclined cores have their own merits and demerits which are summarized as (Singh et al.,1995):

1. Vertical core provides higher pressure on the contact surface between the core and the foundation and will provide more protection against the possibility of leakage along the contact.
2. Thickness of central core is slightly greater than thickness of the slanting core for given quantity of soils.
3. Vertical cores provide better facility for grouting of foundation or contact zone or any cracks in the core if required afterwards, as this can be done through vertical rather than inclined holes
4. In vertical core dams the foundation area is independent of depth of foundation and hence can be marked and treated in advance.
5. The problem of differential settlement between the vertical core and the shell zone may result in cracking parallel to the dam axis.
6. In case of inclined core dams, downstream shell can be placed in advance and laying of filter, core and upstream zone can be taken up later. This ensures rapid progress of the work which is advantageous in areas where a short season of dry weather exists.
7. In inclined core dams foundation grouting of the core can be carried out while the downstream shell is being placed. However, this advance treatment may present problem if the depth of the excavation increases because the contact area moves upstream.
8. In inclined core dams since the flow lines are essentially vertical and equipotential lines are almost horizontal under sudden drawdown condition, the drawdown pore pressure is very much reduced. However, a larger part of the slip surface for the upstream slope

passes through the core material than would be the case with central core dams.

9. In the case of cracking of the core, the inclined core will have a large mass of stable rockfill on the downstream side to support.
10. Filter layers can be made thinner and placed more conveniently in inclined core dams.
11. Steeper down stream slopes can be adopted in the inclined core dams as very small portion of the slip surface intersects the core.

Some broad consensus exists on the relative advantages and disadvantages of different locations of cores in a general way. However, opinion differs on the influence of the core shape on some specific aspects such as susceptibility to cracking , behaviour of the dam under seismic loading etc. FEM is the best tool to analyse the zoned embankment dams under the static and dynamic loading conditions.

1.2 EARLIER STUDIES

Model experiments for Kennedy dam in Canada (Clough et al.,1958) and Ramganga dam in India (krishna et al.,1969) indicated better performance of a slanting core in seismic conditions. Thomas (1976) recommended use of central core dam in seismically active zone. Mejia et al.(1983) found that effect of canyon geometry was to stiffen the system. Patnaik (1985), Singh (1987) showed that the effect of berm is to reduce accelerations in the dam sections. Based on the simplified assumptions, the finite element analysis by Liam Finn et al.(1966) indicated a larger concentration of stress in inclined core dams.

1.3 OBJECTIVE AND SCOPE OF THE STUDY

This present study deals with the analysis of earth and roackfill dam in general and influence of the core geometry (i.e., placement of the core in the section) on the behaviour of the dam under the static as well as dynamic loading conditions in particular. While analysing the dam, the non-linear nature of the soil and sequential construction of the dam are also taken into account. For the purpose of the present study, a 150 m high rockfill dam with three placement geometries of the core has been considered. The amount of fill in the core has been kept same in all the three cases.

Plane strain 2D non-linear finite element analysis simulating the construction sequence of the dam has been adopted in the present study. To achieve the above mentioned objective, the following analyses have been carried out:

1. Linear single lift static analysis of the central core dam for self weight and water pressure corresponding to full reservoir level
2. Non linear multi lift static analysis for the gravity and hydrostatic water pressure to model the construction sequence and reservoir filling for all the three alternative sections
3. Free vibration analysis of all three sections
4. 2D linear dynamic analysis (Time History analysis) of all the three alternative sections

1.4 METHODOLOGY

Plane strain 2D non-linear finite element analysis simulating the construction and reservoir filling sequences of the dam has been used in the present study. Pentagon 2D software developed by Emerald Soft P.E., South Korea has been used in the present study.

1.5 ORGANISATION OF DISSERTATION

The study is presented in six chapters. The content of each chapter is briefly indicated below:

Chapter 1:	Introduction of the problem, scope and objectives of the study have been presented
Chapter 2:	Review of the literature has been given
Chapter 3:	Methodology and theoretical background has been discussed
Chapter 4:	Details of Mathematical model used in the study and analysis performed have been presented
Chapter 5:	Results and discussion of the study have been given
Chapter 6:	Conclusions and future scope of the study have been presented

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

In analysis of dams, two possible approaches exist to predict the failure, namely "Limit Equilibrium Concept" and "Stress Distribution Method". The earlier one is used ignoring the deformability characteristics of the material and concentrating on the collapse mechanism, thus only strength properties are considered. This method fails to provide deformation and stress condition in the dam. In case of the later, constitutive relations are established and a complete solution for the deformation is obtained under all loads, viz. static and dynamic. Non-linear approaches provide a more realistic and accurate estimate of the behaviour of the dam.

From the study of literature, it was seen that quite a few studies concerning dams using Finite Element Method (FEM) have been carried out. The dynamic and static behaviour of earth and rockfill dams with different core positions have also been taken up but to a lesser extent.

2.2 ANALYSIS OF MAJOR DAMS

2.2.1 Analysis of Oroville Dam

Nobari and Duncan (1972) computed the deformations in the Oroville Dam with moderately inclined core (Fig:2.1). The water load on the core was applied as a boundary pressure on the upstream side of the core. The buoyant uplift forces in the upstream shell were represented by a nodal point loads applied to the submerged elements. In general, the measured values and the computed values for the deformations were in agreement. The movement of the core due to the reservoir filling sequence was simulated in three stages and downstream movements were calculated. The final calculated position of the crest was about 3.05 cm (0.1 ft) downstream from the initial position. At mid height, the movement was about 24.4 cm(0.8 ft), which matched with the observed one.

2.2.2 Analysis of Duncan dam

Eisenstein, et al. (1972) analysed the history of cracking sequence of 36 m high, upstream sloping core Duncan dam on Duncan River in British Columbia, Canada using FEM taking into account the incremental loading, non-linear stress-strain relationship and three dimensionality. The designers of the dam anticipated large amount of settlement and designed the dam to accommodate these. While the predicted and observed settlements agreed in their magnitudes but, the observed locations differed from the predicted ones. The maximum settlement, which was expected beneath the middle of the dam finally appeared to be close to the left abutment. The results of the area of the tension zone computed agreed well with the position of the cracks observed in the dam. The results from analysis were consistent both with location of the cracks and their propagation during construction.

2.2.3 Analysis of Chicoasen Dam

Marsal et al. (1979) reported the change in design of Chicoasen Dam, Mexico (Fig:2.2) based on FEM analysis. The dam is an earth and rockfill dam, 210m high over river bed with maximum height over the bed rock of 264 m.

The first stress-deformation studies were made for the original design of the dam, which had central impervious core. The FEM was used and non-linear elastic analysis was confined to the case of plane strain. Results disclosed a strong arching induced by the walls of the canyon and displayed significant interaction between the compacted zones of the shells and the core. The consequence of both effects was a substantial reduction in the vertical stresses.

Further, a 3-D finite element analysis was undertaken assuming linear elastic behaviour of the embankment materials. The analysis showed that the vertical stresses within the core were greater than the adjacent pervious zones, particularly in the lower third of the embankment. These stresses substantially decreased towards the abutments.

On the basis of the above studies, two strips of 10 m wide, uniform rockfill with nominal grain sizes comprised between 15 and 25 cm were positioned adjacent to the transition (Fig:2.3).

2.2.4 Analysis of Dartmouth Dam

Adikari, et al. (1982) studied the Dartmouth dam, a 180 m high rockfill with a central clay core well instrumented, located in North-Eastern Victoria, Australia. A non-linear finite element analysis simulating its construction behaviour was carried out, taking into account the water load prevailing at the end of construction.

Satisfactory results were obtained from the comparison of predicted and measured values in respect of displacements. The highest stress occurred in the filter zone. The maximum stress in the core was about 50% that of the filter zone. The stiffer filter zones have resulted in significant interaction effects and a reduction in vertical stresses in the core at all elevations.

2.2.5 Analysis of Tehri Dam

Paul (2002) studied about the seismic safety of the Tehri dam. The dam is 260.5 m high earth and rockfill dam with moderately inclined core with upstream and downstream transition zones.The dam was later tested for various levels of shaking as per different earthquake strong motions that were evolved by experts progressively in due consideration to all possible worst case scenario in Himalayan environment. The dam has been checked for a Maximum Credible Earthquake (MCE) producing Peak Ground Acceleration (PGA) of 0.5*g* at the dam site, which included a sequential non-linear static analysis using the Mohr-Coulomb material model for the rockfill and the clay core materials.

Two sections, corresponding to (i) Section-1, which has the maximum core height and (ii) Section-2 having maximum downstream slope, but short upstream slope were taken for analysis .Two dimensional finite element plane strain idealisation mapped with eight nodded isoparametric elements was used for the analysis. The nodes at the base of the dam were assumed to be fixed. The water pressure at full reservoir condition was taken as edge load on the upstream portion of the clay core.

The maximum horizontal/ vertical accelerations and displacements at the dam crest have been shown in Table 2.1.

Table 2.1 Maximum horizontal/ vertical accelerations and displacements at the crest

Response Parameters	Section-1	Section-2
Max. horz. Acceleration at dam crest	1.136g	0.978g
Max. vert. Acceleration at dam crest	0.703g	0.434g
Max. horz. Displ . at dam crest (cm)	40.25	36.69
Max. vert. Displ. at dam crest (cm)	25.87	14.78
Max. plastic strain	0.08	0.08

Quantitatively, the dam was found to undergo plastic deformations mostly near the upstream and downstream slopes, which were taken care of by providing a sumptuous riprap composed of blasted rock.

2.3 DYNAMIC BEHAVIOUR OF ROCKFILL DAMS

Ahmed et al. (1988) studied a number of computational models to study the effect of earthquake induced response of earth dams. These models ranged from one to three dimensional, which accounted for gravitational effects and canyon geometry. Soil behaviour is represented by an incremental plasticity constitutive relation. A multi surface kinematic plasticity model was used and the performance of the La Villita dam was analyzed for the Sept 19 ,1985 earthquake and the results were compared to the actual response of the La Villita dam, which closely matched with the observed response in all the directions.

Scott et al. (1990) discussed in detail the response of the La Villita dam during five earthquakes. These had caused significant permanent deformations. A portion of the crest moved as a sliding mass. The strong motion records showed localized deformations along with the magnitude of the associated yield acceleration. The yield accelerations imposed a barrier on the magnitude of the inertial forces, which can develop in a sliding zone.

Lavania et.al. (1990) studied the influence of soil modulus variation on the Dynamic response of a rockfill dam and concluded that:

- Natural frequencies increased with increase in shear modulus.
- The effect of increase in the value of shear modulus on the acceleration was not very significant.
- Mode shapes as well as stresses did not register any significant change, though the displacements increased.

2.4 STUDIES ON INFLUENCE OF CORE POSITION

Clough et al. (1958) performed dynamic tests on scale models (1:150) of central and inclined core dams and found that in the central core dams, vertical core breaks the continuity between upstream and downstream segments of the rockfill dams and constitutes a zone of weakness. Whereas, the entire structure of sloping core dams acts as a single unit. Also, because of its greater rigidity, sloping core dams showed less settlement that accompanied the shearing distortion.

Sherard et al.(1963) expressed that they did not have any clear evidence and opinion to indicate that the sloping cores were preferable over the vertical core owing to the less susceptibility to cracking. However, they believed that the sloping cores could be safer than vertical cores under earthquake shocks because of the larger body of stable rock exists down stream of the core.

Arya, et al.(1978) tested models of central and inclined core dam sections (scale ratio of 1:150) to compare their relative performance under reservoir full and empty conditions. Model tests demonstrated that slumping of the crest was large in case of inclined core darns, though the tendency of separation of the shell from the core was more in case of central core dams.

According to krishna (1962), a dam with vertical core vibrates transversely, there is a tendency of masses i.e., upstream shell, core and downstream shell to vibrate out of phase, because of different elastic properties. The three masses possibly tend to separate out resulting in cracks along the junction. This separation could encourage the slipping to occur on either of the two slopes. The sloping core by virtue of one mass resting on the other prevents the tendency of separation. Therefore, sloping core was suggested as the better option under the dynamic loading.

Saini et al. (1968) studied static as well as dynamic behaviour of three dams, (i) homogeneous dam, (ii) dam with central core and (iii) dam with sloping core, with the help of FEM approach considering the linear elastic properties of the soil and concluded that:

a) The effect of core was to decrease the natural frequency of vibration as compared to homogeneous dams. The effect of the inclination of the core over the central core was to increase the stiffness in certain modes of vibration, but to decrease in certain other modes. Correspondingly, the frequencies for sloping core were higher in certain modes and smaller in others.

b) The effect of core was to increase the static stresses .The sloping core gave rise to an increase in stresses over central core and tensile stresses were developed in a small region near the top.

c) The dynamic stresses for a dam with impervious core were generally lower than those for homogeneous dams with increased stresses near the top portion. For a sloping core dynamic stresses were generally higher than those for a central core. Thus, for sloping core dams the total stress distribution was less favourable giving rise to high tensile stresses near the top region of the dam.

d) Due to the presence of the core, horizontal displacements were decreased, and vertical displacements were increased. The sloping core resulted in increase of the vertical displacements than those for central core dam, though the effect on the horizontal displacement was small.

Okomoto et al. (1974) opined that high tensile stresses occurred in the high rigidity area. When the rigidity of the core was smaller than that of shell, high tensile stresses appeared along the sides of slopes of dam. In reverse situation, high tensile stresses appeared in upper part of the core producing approximately horizontal cracks in the core.

Kuberran, et al.(1978) carried out a linear elastic analysis by FEM on a typical rockfill dam with inclined core on key trench. Gravity-turn on analysis and sequential analysis were resorted to. The key findings of the investigation led to:

a) In Gravity turn-on analysis, maximum settlement occurred at the top, whereas maximum settlement occurred at mid height in case of sequential analysis.

b) Horizontal movement in the top portion of upstream shell took place in downstream direction, whereas the bottom portion moved towards upstream in case of inclined core dam.

c) Settlement in the core was more prominent as compared to shell at the same level due to high compressibility of the core.

Nayak, et al. (1978) analysed a 260 m. high rockfill dam with vertical and inclined cores under two dimensional plane strain condition with linear, mixed graded linear and parabolic isoparametric elements. They incorporated sequential construction and non-linear material properties. From the study, it evolved that:

a) Linear elements would be used for rockfill portion and parabolic elements for the core and transitions, where higher accuracy was required due to stress transfer.

b) In case of inclined core, vertical stress concentration in downstream transitions was greater than that of vertical core. Also, higher vertical stresses were observed in downstream shell in case of inclined core.

2.5 CONCLUDING REMARKS

From the above literature review the following inferences can be drawn:

1. The FEM can be effectively used in the analysis of earth and rockfill dams under static and dynamic loading conditions duly taking sequential construction, non-homogeneous and non-linear properties of dam material, complex boundary conditions in to account.

2. Few studies were undertaken to study the influence of the core geometry on the dam behaviour under static and dynamic loading conditions using model studies as well as FEM. In this aspect the opinion differed from one author to the other. Following points are made out from these studies:

   - Static Analysis

     - The settlement distribution pattern is quite distinctive in the case of gravity turn-on analysis from that of the sequential analysis. The maximum settlement occurs at the top in the first analysis while it occurs at mid height in case of later.

     - Settlement in the core remains more as compared to shell at the same level due to high compressibility of the core.

     - The sloping core results in lowering of the vertical displacements than those for central core dam, though the effect on the horizontal displacement is small.

     - Higher vertical stresses are observed in downstream shell in case of inclined core dams.

     - When the rigidity of the core is smaller than that of the shell, high tensile stresses appear along the side slopes of dam. In the reverse case the tensile stresses are observed at the top of the core.

     - The tendency of separation of the shell from the core is more predominant in case of central core dams.

- Dynamic Analysis
    - In case of central core dams, three masses (viz. upstream shell, core and downstream shell) may tend to separate resulting in cracks along the junction. The sloping core by virtue of one mass resting on the other prevents the tendency of the separation.
    - The sloping core results in lowering of the vertical displacements than those for central core dam, though the effect on the horizontal displacement is small.
    - For a sloping core, dynamic stresses are generally higher than those for a central core. Thus for sloping core dams the total stress distribution is less favourable.

The review indicates that a conclusive finding does not exist on the behaviour of the earth and roackfill dams under static as well as dynamic loading conditions. The 300 m high Nurek dam (Fig:2.4) of Russia is a classical example of dam constructed in seismic zone with central core where as 235 m high Oroville dam(Fig:2.1) (USA) and 261 m Tehri Dam (Fig:2.5) (India) in highly seismic zone were provided with moderately inclined cores. An inclined core was adopted at 130m Miboro Dam (Fig:2.6) in Japan.

It reveals uncertainty over the proper understanding and analysis of the dam behaviour with different core positions and soil properties.

The lack of adequate studies on dams with different core positions and non-linear soil flux has impeded the analysis and understanding of intricate behaviour of dams. With a view to addressing this problem, the analysis of an earth and rockfill dam with three different core positions has been attempted considering non-linear properties of the dam material. Moreover, the established analytical approach of FEM has been adopted in this study and the results are described in the following chapters.

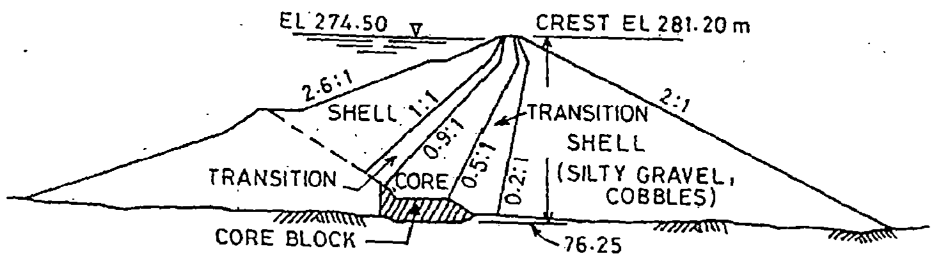


Fig:2.1 Oroville Dam (USA)

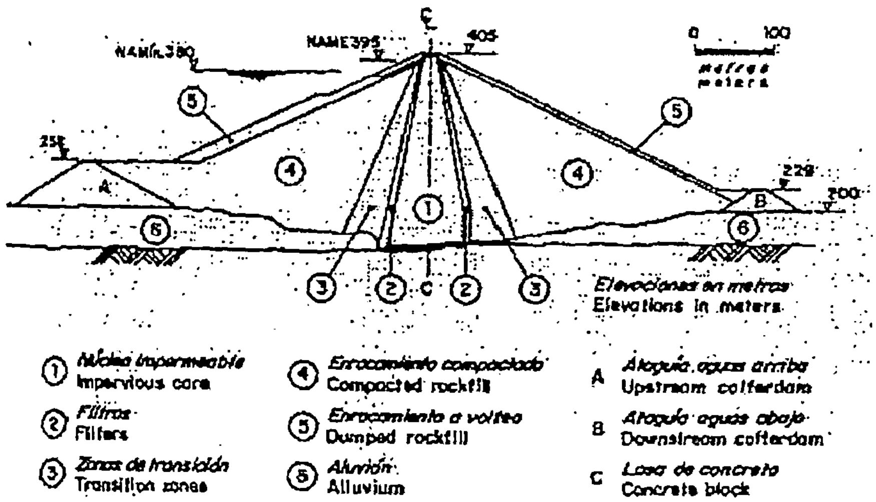


Fig :2.2 Proposed cross section of Chicoasen dam

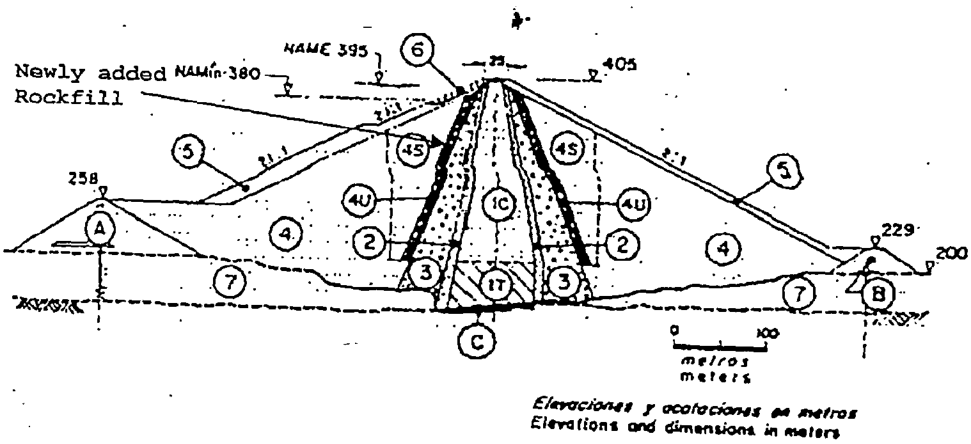


Fig :2.3 Modified cross section of Chicoasen dam

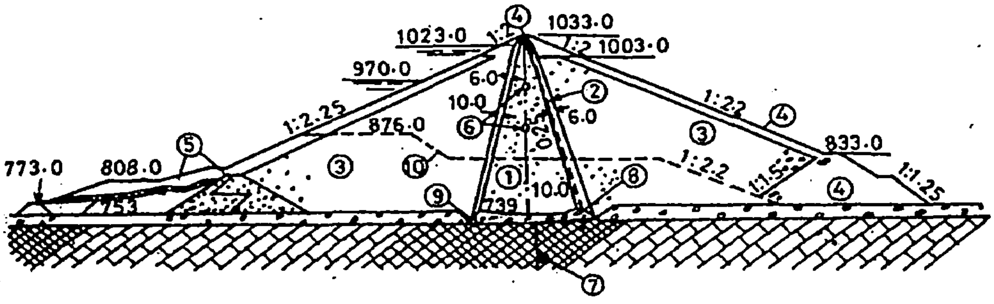


Fig: 2.4 Nurek Dam (Russia)

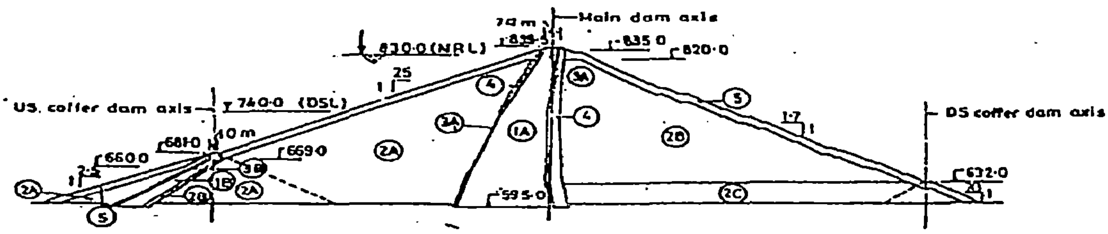


Fig:2.5 Tehri Dam (India)

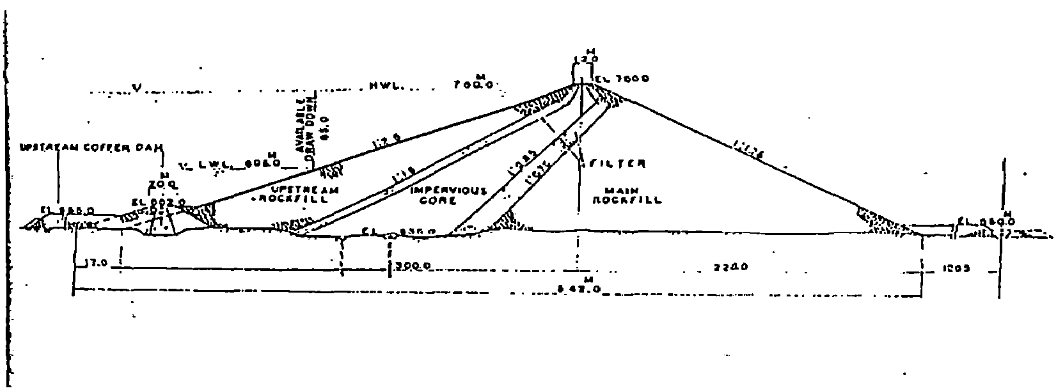


Fig:2.6 Miboro Dam (Japan)

CHAPTER 3

METHOD OF ANALYSIS

3.1 GENERAL

The finite element method was introduced to the geotechnical engineering when Clough and Woodward (1967) demonstrated its usefulness for analysis of stresses and movements in earthen dams. Geotechnical engineers had long been aware of the limited usefulness of linear elastic analysis of soil and rock masses, and it was immediately apparent that the ability to analyse non-linear behaviour by using finite element method had great potential for its use in geotechnical engineering problems.

The applications of finite element method in the analysis of earth and rockfill dams are numerous. It can be applied for computing internal deformations of the core and shell, obtaining stress distribution and load transfer within the dam section, locating the zones of potential cracking resulting from tensile stresses and investigating the likelihood of hydraulic fracturing. The technique can take into account the sequential construction using non-linear stress strain relationship of the material. The results can also be used to choose suitable location of the core of the dam in such a way so as to reduce the likelihood of existence and the magnitude of the zones of the tensile stresses.

3.2 ANALYTICAL MODALITIES

The first step in the finite element analysis of a dam is discretization of the dam section. Horizontal slices known as lifts are done first and then the slices are further divided into elements. The analysis is then carried out for the various construction and reservoir filling stages of the dam. In the construction stage the gravity load is applied on the structure based on the layer thickness of the particular stage and unit weights of constituent material. In the case of hydrostatic pressure loading due to the water impounded in the reservoir, the weight of the rockfill under water is taken as submerged while the water pressure is assumed to act on the upstream sloping face of the core as external load. If the foundation is deformable the water pressure is also assumed to act on the foundation surface upstream of the core and applied at the nodes.

In the non-linear analysis by incremental procedure the load is applied in steps and in each step the values of Young's modulus (E) and the Poisson's ratio (μ) is taken constant and modified for the next step in accordance with the stress strain relationship. In the case of sequential construction, however each layer is considered separately. Non linear analysis of each layer is done by carrying out iterations within each layer so as to have close agreement with the adapted elastic constants with the modelled values. The analysis is first carried out for the first layer using assumed values of E and μ based on the stress level and stress and strains determined for different elements. With the values of stresses thus computed the fresh values of E and μ are determined for each element from the stress strain relationship, and computation for stress and strains is carried out once again.

Free vibration characteristics and earthquake response of the dam for given accelorogram is evaluated considering the material properties obtained at the end of non-linear multi lift analysis.

In the present study, the three alternative sections with different core positions are analysed on the above lines and results are compared based on the deformations, stress conditions in the sections. In addition to this, other aspects such as load transfer between core and shell, hydraulic fracturing susceptibility and strength mobilization ratio of the core and shell are also evaluated.

Brief details of the finite element method ,constitutive law of dam material, reservoir filling effects on the dam, load transfer , hydraulic fracturing susceptibility and mobilization ratio are given in the following paragraphs.

3.3.1 Description of FEM

The analysis of the structure by the finite element method is an idealization of an actual elastic continuum as an assemblage of discrete elements interconnected at their nodal points (Singh, 1991).

The various steps involved in finite element analysis are:

i. Subdivision of the continuum into finite elements of suitable configuration
ii. Evaluation of element properties
iii. Assembly of element properties to obtain the global stiffness matrix and load vector.
iv. Solution of resulting linear simultaneous equations for the primary unknowns after introducing the boundary conditions
v. Determination of secondary unknowns such as stresses and strains.

3.3.2 Constitutive Law

It has been repeatedly shown that the most influential factor in the finite element analysis of embankment dams is the modelling of the stress-strain behaviour of the fill material by an appropriate constitutive law. However, in spite of the diversity of the stress-strain relationship being used ,reasonable agreement has been achieved when the results of finite element analysis have been compared with field observations. This is not surprising bearing in mind the fact that most of analyses were done after the field measurement had been made.

The most used function for simulation of stress-strain curves in finite element analysis was formulated by Duncan and Chang (1970) using Kondner's (1963) finding that the stress-strain curves in triaxial compression test is very nearly hyperbola. The procedure uses Mohr Coulomb failure criteria and develops relationship for the tangent modulus which may be expressed in terms of total or effective stresses as below:

$$E_t = \left[1 - \frac{R_f(1-\sin\phi)(\sigma_1 - \sigma_3)}{2(c.\cos\phi + \sigma_3 \sin\phi)}\right]^2 kP_{atm}\left(\frac{\sigma_3}{P_{atm}}\right)^n \qquad \text{............(2.1)}$$

Where

E_t tangent modulus

σ_1 major principal stress

σ_3 minor principal stress

E_i initial tangent modulus

k a dimensionless modulus number

n a dimension less modulus exponent determines the rate variation of E_i with σ_3

P_{atm} the atmospheric pressure

R_f failure ratio

C cohesion intercept

ϕ friction angle

In Duncan and Chang paper the second elastic constant, Poisson's ratio (μ), was assumed as constant. This assumption was subsequently modified by Kulhawy and Duncan. While maintaining the same expression for E_i, E_t as presented by Duncan and chang ,Kulhawy and Duncan (1972) proposed the following expression for Poisson's ratio:

$$\mu_t = \frac{G - F\log\left(\frac{\sigma_3}{P_{atm}}\right)}{(1 - d\varepsilon_a)^2} \qquad \text{.............(2.2)}$$

μ_t tangent Poisson's ratio

μ_i initial tangent Poisson's ratio

G value of μ_t at one atmosphere

F rate of change of μ_t with σ_3

d rate of change of μ_i with strain

where $$\varepsilon_a = \frac{(\sigma_1 - \sigma_3)}{kP_{atm}\left(\frac{\sigma_3}{P_{atm}}\right)^n\left[1 - \frac{(\sigma_1 - \sigma_3)}{2(c.\cos\phi + \sigma_3\sin\phi}\right]}$$

3.3.2 Deformations due to Reservoir Filling

Measurements made in many earth and rockfill dams have shown that large settlements, horizontal movements, and cracking are frequently caused by the reservoir filling. The effects of the reservoir filling are four fold:

i. The water load on the core causes downstream and downward movements.

ii. The water load on the upstream foundation causes upstream and downward movement.

iii. The buoyant uplift forces in the upstream shell cause upward movements within this zone.

iv. Deformations due to softening and weakening of the shell material on wetting.

3.3.3 Load Transfer

The possible development of cracks in the cores of zoned dams is one of the major problems confronting the dam designer. Number of field observations and studies have indicted that differential settlement leading to load transfer between the adjacent zones is the main cause of crackings. The phenomenon of load transfer (Kulhawy et al.,1976) occurs in zoned dams because of the different stiffnesses of the adjacent zones. In the construction of zoned dam with a soft core (low modulus) and a stiff shell (high modulus), the core will settle with respect to the shell and, if no separation occurs along the zone boundaries, the core will tend to "hang" on the shell. The placement of successive layers of fill accentuates this process of the core settling more than the shell and "hanging" on the shell, with the result that the stresses in the core are less than would be expected from gravity alone. For equilibrium on a horizontal plane, the reduction in core stresses must lead to the increase in the stresses of adjacent shell. Therefore, if the core is softer than the shell, load transfer occurs from core to shell leading to hydraulic fracturing or formation of cracks by high water pressure. If the core is stiffer than shell, then the load transfer may take place from shell to core which may cause local overstress in the core.

This load transfer can be evaluated by comparing the computed values of the major principal stress in the core and the core overburden pressure at any given depth below the crest. The ratio less than one indicates load transfer from core to the shell , while the ratio greater than 1.0 indicate load transfer from the shell on to the core.

3.3.4 Mobilization Ratio

The potential for local over stress leading to either plastic yield or brittle cracking can be assessed from the computed principal stress in the dam (Kulhawy et al.,1976). The ratio of mobilized deviator stress $(\sigma_1 - \sigma_3)$ to the failure strength $(\sigma_1 - \sigma_3)_f$ termed as mobilization ratio can be used to assess the failure condition in the dam. This ratio also indicates the ratio of maximum shear stress existing at any point of the dam to shear strength of the soil at that point.

The failure deviator strength can be evaluated from Mohr-Coulomb failure theory which is being adopted in the present analysis as:

$$(\sigma_1 - \sigma_3)_f = \frac{(2c\cos\phi + 2\sigma_3 \sin\phi)}{(1-\sin\phi)}$$

and mobilization ratio can be computed with the following formula:

$$\text{Mobilization ratio} = \frac{(\sigma_1-\sigma_3)}{(\sigma_1-\sigma_3)_f} = \frac{(\sigma_1-\sigma_3)(1-\sin\phi)}{(2c\cos\phi+2\sigma_3\sin\phi)} \quad \text{.........................(2.3)}$$

3.3.5 Hydraulic Fracturing

Hydraulic fracturing or the formation of hydraulically induced cracks in the core can occur when the water pressure at a given depth exceeds the total stress at the same depth (Kulhawy et al.,1976). If the crack is horizontal, as shown in the Fig: 3.1, the water pressure would have to exceed the vertical stress (σ_y) However, it is also possible that the crack would initiate normal to the upstream face of the core, in which case the water pressure would have to exceed σ_p ,the stress in the core which is parallel to the face of the core. Earlier studies indicate that the major principal stress in the core is parallel to the upstream face of core and therefore to σ_p. Further it is also conformed that the values of σ_1, σ_p and σ_y differ by no more than about 5%.So any of theses values can be

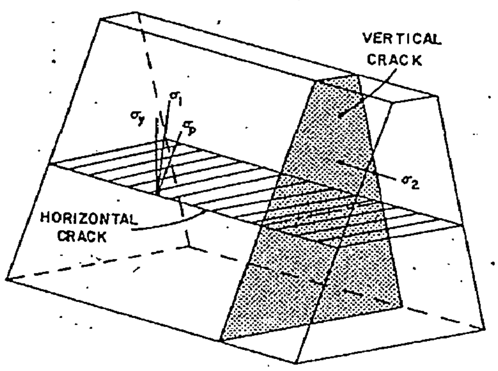


Fig:3.1 Potential Transverse Cracks and associated Stress Conditions

used regardless of the precise mode of crack initiation. The hydraulic fracturing would be most critical when the reservoir rose to maximum pool level quickly and the core did not have sufficient time to consolidate. So, in the present study the total stresses at the end of construction are compared to the hydraulic water pressure (u_h), which would occur under full reservoir condition. Based on the intermediate principal stress, σ_2, it can be studied whether vertical hydraulic fracturing is possible in the dam or not also. In this method the core is assumed to have no tensile strength, so all hydraulic fracturing potentials computed represent a lower bound.

3.4 DYNAMIC ANALYSIS

From structural standpoint, the earth and rockfill dams are continuous bodies with an infinite number of degrees of freedom, interacting with the foundation ground and the water in the reservoir. In finite element analysis of the dam response, the dynamic behaviour may be represented by a finite number of degrees of freedom associated with nodes of descritization. Generally, the interaction of the rockfill dams with foundation as well as the reservoir is neglected.

Under seismic loading represented by the acceleration $\ddot{u}$(t) of the foundation ground movement, the dam undergoes several displacements and deformations that can be mathematically described by the motion equations

$$[M]\{\ddot{\delta}\} + [C][\{\dot{\delta}\} + [K]\{\delta\} = [R] = -[M]\{r\}\ddot{u}(t) \quad \ldots\ldots\ldots\ldots\ldots\ldots(2.4)$$

where [M] Mass Matrix
[C] Damping Matrix
[K] Stiffness Matrix
$[\ddot{\delta}],[\dot{\delta}],\ [\delta]$ Acceleration ,Velocity and Displacement vectors
[R] Forcing function
$\ddot{u}$ Design earthquake accelorogram

Estimation of the dynamic response of the dam subjected to seismic excitation actually means to determine the displacements and the accelerations induced in the dam during eartquake, and then, using these values ,to calculate the dynamic stresses. In evaluation of the above quantities two integration methods are customarily used:

1. Modal analysis with seismic response spectra, which is used to determine maximum probable values of the displacements, accelerations and stresses as the function of the response spectra of the design earthquake.
2. Direct integration of the system (Step by step integration or time history analysis) which gives the time history of the displacements, accelerations and stresses induced by the design earthquake, which is described by the accelorogram.

In the present study the time history analysis has been adopted to study the stress and deformation conditions of the dam under earthquake loading. Even though the mode shapes, fundamental frequencies are not necessary in this analysis but the free vibration analysis has been carried out to check the general behaviour of the dam. The details are given here under:

3.4.1 Free Vibration Analysis

For the computation of eigenvalues, the problem is defined as:

$$[K][\Phi] = \omega^2 [M][\Phi] \qquad \text{......................(2.5)}$$

here ω natural circular frequency

Φ mode shape vector or eigen vector

Lumped mass approach has been used to formulate the mass matrix. For computation of eigenvalues and the mode shapes the inverse iteration technique is used.

3.4.2 Step by Step integration Method

In this method motion equation in its general form is numerically integrated to obtain the time history of the response to a given excitation accelogram. This method requires knowledge of variation rule of response accelerations during the time increment "h". The acceleration customarily assumed to remain constant or vary linearly during the time increment. Newmark method is considered as more general formulation of the problem as this method can be applied in other versions also. As per this method the velocities and the displacements at the end of (t+ h) of the time increment are given by:

$\{\dot{\delta}\}_{t+h} = \{\dot{\delta}\}_t + (1\text{-}A)h\{\ddot{\delta}\}_t + Ah\{\ddot{\delta}\}_{t+h}$ ....(2.6)

$\{\delta\}_{t+h} = \{\delta\}_t + h\{\dot{\delta}\}_t + (\tfrac{1}{2} - B)h^2\{\ddot{\delta}\}_t + Bh^2\{\ddot{\delta}\}_{t+h}$ ....(2.7)

A factor provides linearly varying weightage between influence of initial and final acceleration on the change of velocity

B factor provides weightage of initial and final acceleration on the change of displacement

If A= ½ and B= ¼ then the above equations result to the constant average acceleration method.

The above relations are introduced into the general dynamic system of equations eq2.4 ,and the following equation is obtained for constant acceleration condition

$$([M]+h/2[C]+h^2/4[K])\{\ddot{\delta}\}_{t+h} = [M]\{r\}\ddot{u}_{t+h} - [C](\{\dot{\delta}\}_t + h/2\{\ddot{\delta}\}_t) - [K](\{\delta\}_t + h\{\dot{\delta}\}_t + h^2/4\{\ddot{\delta}\}_t) \quad ....(2.8)$$

Using the above equation and following initial conditions $\{\ddot{\delta}\}_{t+h}$ is evaluated at each time step.

At t= 0: $\{\delta\}_0 = \{\dot{\delta}\}_0 = \{\ddot{\delta}\}_0 = \{0\}$, $\ddot{u}_0 = 0$, $\ddot{u}_{t+h} \neq 0$.

Basically, the analysis procedure consists of the following steps:

Evaluation of $\{\ddot{\delta}\}_{t+h}$ from eq 2.8 and the values are introduced in the eq 2.6 & 2.7 to obtain $\{\dot{\delta}\}_{t+h}$ and $\{\delta\}_{t+h}$. The values at the end of preceding increment become those at the beginning of the next increment. Same procedure is followed to evaluate the response of the system for a given excitation.

CHAPTER 4

MATHEMATICAL MODEL

4.1 DAM SECTION

A 150 m high earth and rockfill dam having upstream slope as 2.5H:1V and downstream slope as 2H:1V with three core geometries was analysed. The section of the dam is shown at Fig. 4.1.The width of the core at the top of the dam section and free board were kept as 5 m. The thicknesses of upstream and downstream filters were 4 m and 10m respectively. The details of placement of the cores are given in the Table: 4.1

Table:4.1 Placement of core positions

Type of the core	U/S slope of the core	D/S slope of the Core	Remarks
Central	0.2H:1V	0.2H:1V	-
Moderately inclined	0.5H:1V	0.1H:1V	Both inclined towards upstream shell
Inclined	0.9H:1V	0.5H:1V	Both inclined towards upstream shell

4.2 MATHEMATICAL IDEALIZATION

In the present study it has been proposed to investigate the influence of the core position on the behaviour of the earth and rockfill dam which can be performed with the help of 2D FEM analysis as the length of the dam has been assumed to be long enough to represent it as 2D plane strain problem. Educational version of PENTAGON finite element analysis software developed by Emerald Soft P.E., South Korea has been used in the study. The restrictions on the software left a chance to use either coarser 3D mesh or finer 2D mesh in modelling the problem; in view of the problem requirement and computational restriction, the finer 2D mesh has been opted.

Accordingly, two dimensional plane strain non-linear multi lift finite element analysis was carried out to model the construction and reservoir filling sequence of the dam. Eight nodded isoparametric elements with two degrees of freedom at each node (displacements in x and y directions) were used for descretization of the model. All the three sections were discretized with 180 elements and 595 nodes. The elements of the three sections are shown in Fig 4.3, 4.4 and 4.5. The foundation of the dam was considered to be rigid and accordingly the nodes at the base were taken as fixed. To simulate the sequential construction, the dam was assumed to be raised in 5 stages with the layer thicknesses as 40 m, 40 m, 15 m, 30 m, 25 m and filled in four stages with water levels at 40 m, 95 m, 125m and 145 m. The construction and reservoir filling sequences are shown in the Fig: 4.2 (a & b).

Khulawy_Duncan hyperbolic elastic model was used to describe the properties of all the dam material.

4.3 ANALYSIS PERFORMED

The three sections were analysed under static as well as dynamic loading conditions. The details of the analyses carried out are elaborated in the following paragraphs:

4.3.1 Static Analysis:

As part of assessing the behaviour of the dam section, the following studies were undertaken:

1. Single stage construction and single stage reservoir filling analysis of vertical core section
2. Non-linear multi stage analysis (five construction stages and four reservoir filling stages) of all three alternative sections

In the single stage static analysis the dam was assumed to be constructed in single stage and considered to be filled up to full reservoir level in a single step. Accordingly, the gravity load and water loads were applied on the model. In case of impounded water in the reservoir, the unit weight of the upstream shell under the water was taken as submerged while water pressure was assumed to act as edge load on the upstream sloping face of the core. Saturated unit weight for the core and moist density for the downstream shell were considered in the analysis. This analysis for the central core dam was carried out to find out the deformation and stress distribution in the dam section and to study the variation of the distribution pattern between the multi stage construction and gravity-turn analysis. For multi lift non-linear analysis, the construction schedule and the sequence of reservoir filling was taken into account by appropriately altering the mesh and material properties of the dam sections.

4.3.2 Dynamic Analysis:

Following two analyses were undertaken to study the displacement and stress distribution in the dam under seismic loading:

1. Free vibration analysis of all three alternative sections
2. Linear dynamic analysis of the dam (Time history analysis) of all the three alternative sections

The material properties corresponding to the final stress state obtained from non-linear multi lift analysis have been used for evaluation of free vibration characteristics and earthquake response of the dam. The frequency of vibration and mode shapes are worked out.

In the 2D linear dynamic analysis, the response spectra compatible earthquake accelerogram generated for Tehri dam (Fig:4.6) with Peak Ground Acceleration pegged at 0.5*g* was used as the input motion at the base of the dam. The vertical ground motion was taken as 2/3 the strength and in phase with the motion in horizontal direction. The damping in the first two modes of vibration was taken as 10% in the dynamic analysis.

While analysing the influence of the core on the performance of the dam in different loading conditions , the following factors were studied:

- Vertical and horizontal deformations
- Major and minor Principal Stresses
- Shear stress
- Mobilisation Ratio
- Load Transfer between core and shell
- Hydraulic fracturing susceptibility
- Displacement and Acceleration during seismic loading
- Combined stresses (static + dynamic)

4.4 MATERIAL PROPRTIES

Material properties used in the present study are given in Table:4.2

Table:4.2 Material properties used in the study

No.	Parameters	Shell	Filter	Core
1	Cohesion (t/m^2)	0.5	0	5.0
2	Angle of internal friction (degree)	42	42	28
3	Moist Density (t/m^3)	2.45	2.00	2.00
4	Saturated Density (t/m^3)	2.49	2.16	2.15
5	Modulus Number (k)	2500	3000	500
6	Modulus Exponent (n)	0.25	0.30	0.60
7	G value	0.43	0.43	0.31
8	F value	0.19	0.19	0
9	D value	14.80	14.80	0
10	Young's Modulus (t/m^2)	16200	19000	5000

4.5 SIGN CONVENTION

The following sign convention was followed in the study:

The positive x-direction is from upstream to downstream, positive y-direction is from down words to up words along the height of the dam.

The displacements u and v are positive in the positive x and y direction respectively.

The major and minor principal stresses are positive if tensile and negative if compressive.

4.6 SOFTWARE USED

Pentagon 2D finite element analysis software developed by Emerald Soft P.E., South Korea was used. This is specifically developed for geotechnical applications. This software enables the user to perform the following tasks:

- Built computer model for geotechnical structures
- Apply operating loads or other design performance conditions

- Study the physical response, such as deformation level and stress distribution etc.

Linear and non-linear two dimensional finite element analysis of any geotechnical structure (dams, tunnels etc) can be carried out with the help of inbuilt material property models of the software. The sequential construction and reservoir filling of the dams can be simulated through this software.

The software has a comprehensive Graphical User Interface (GUI) that gives the users easy, interactive access to programme functions, commands and manuals. The programme is organised into three basic levels:

1. AUTOMESH
2. PENPRE
3. PENPOST

The *AUTOMESH* module acts as a gateway to the programme and this is used for certain global programme controls such as to build the model, applying the loads and to solve FEM problem with the main processor *Pen2_dbx*. The data and model can be checked with the *PENPRE* module. The remaining *PENPOST* module is used for viewing the different aspects of the results.

The programme, which is permitted to be used for limited version only by the developer, is feasible to cope up with 250 elements of 8-noded isoparametric quadrilaterals only. The programme was executed on PC PENTIUM IV- 266 MHZ.

4.6.1 Validation of the Software

The accuracy of the programme was checked by analysing the Tehri Dam (B-15 Section) for which the finite element analysis results were already available in the literature (Paul,2002).The evaluated results were found to be within the permissible limits (i.e.5 to 10%) of the available results.

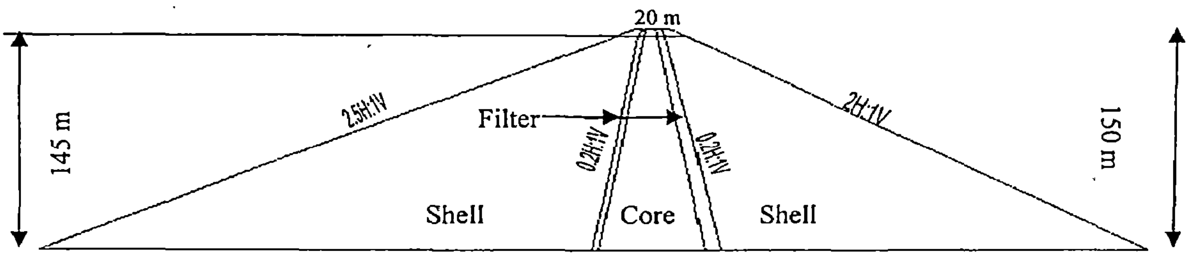


Fig:4.1(a) Central core dam section

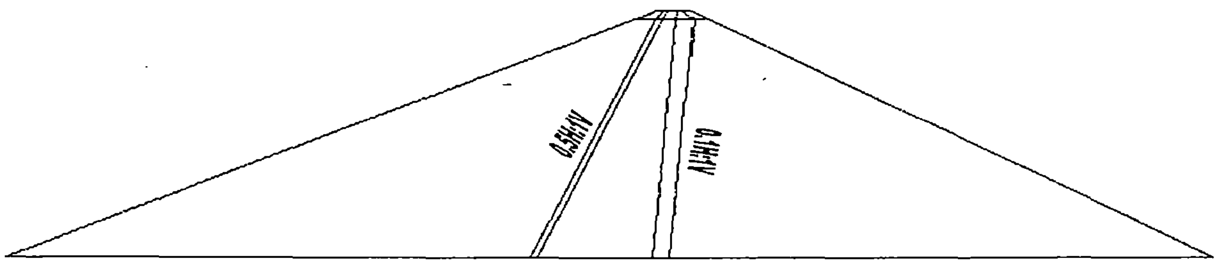


Fig:4.1(b) Moderately inclined core dam section

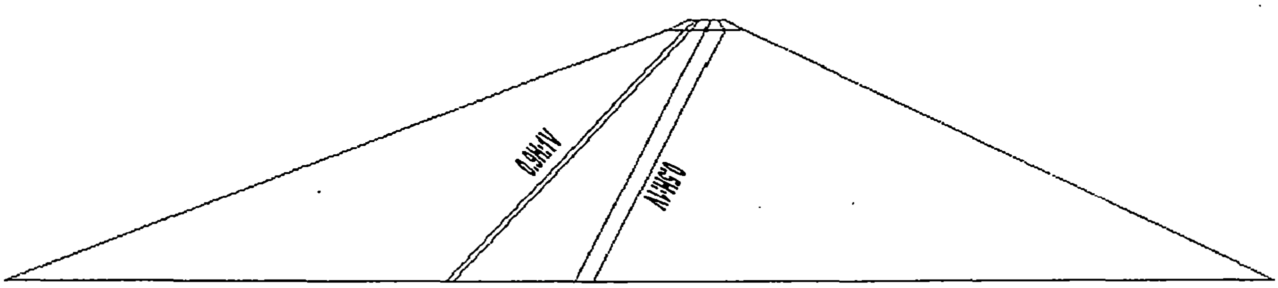


Fig:4.1(c) Inclined core dam section

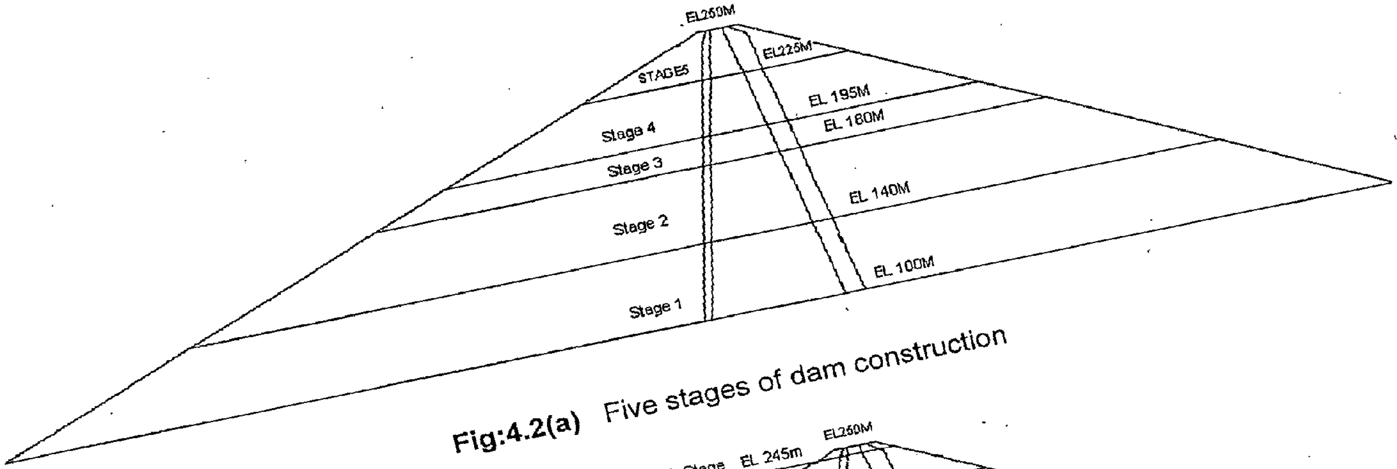


Fig:4.2(a) Five stages of dam construction

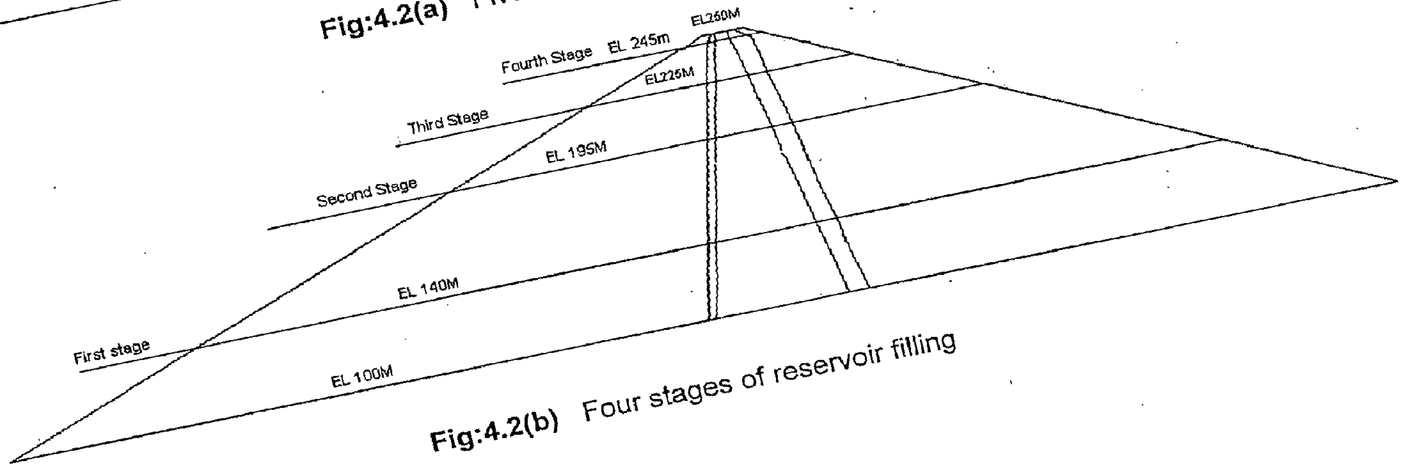


Fig:4.2(b) Four stages of reservoir filling

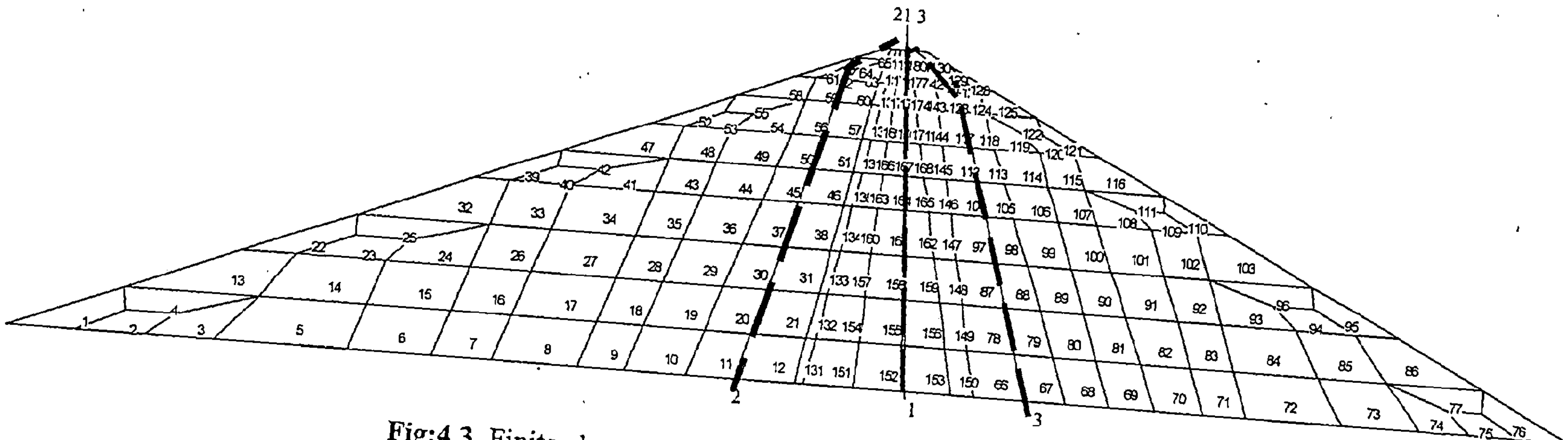


Fig:4.3 Finite element idealisation of Central core dam section

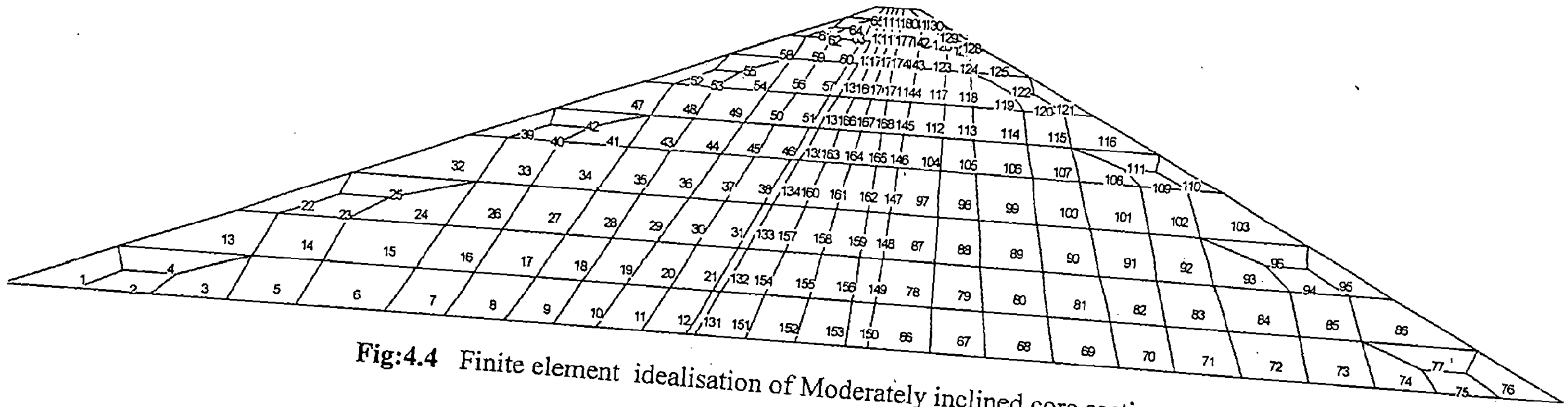


Fig:4.4 Finite element idealisation of Moderately inclined core section.

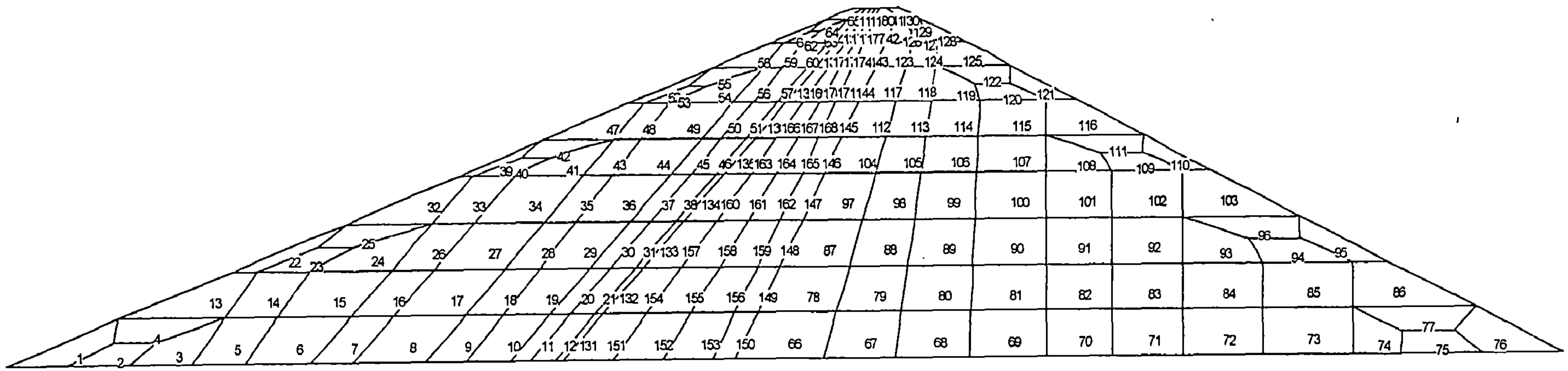


Fig:4.5 Finite element idealisation of Inclined core section

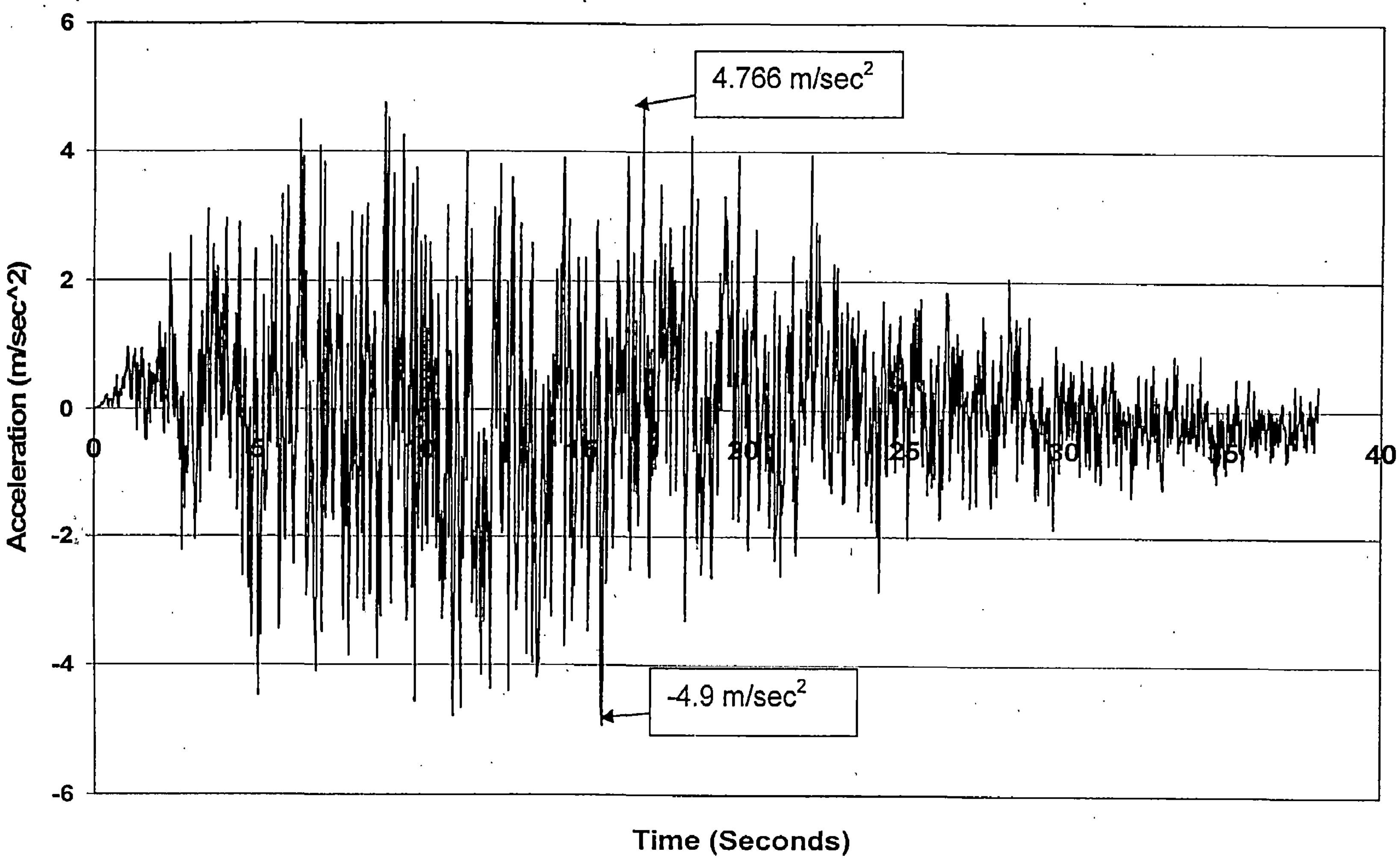


Fig: 4.6 Input Accelorogram

CHAPTER 5

RESULTS AND DISCUSSION

As stated earlier, the static and dynamic analyses were performed on all the three dam sections, viz. with central core, moderately inclined and inclined core. The study is accordingly split into Static and Dynamic analysis.

5.1 STATIC ANALYSIS:

The following static analyses were performed:

A. Single stage construction and single stage reservoir filling for central core dam

B. Non linear five stage construction and four stage reservoir filling for all the three alternative sections

The deformations and stresses are studied at three interfaces henceforth called verticals; one each in upstream and downstream shell one in the core. Figure 4.3 elaborates the interfaces chosen for the study.

Under case A, the results obtained from single stage analysis are presented. The results of the three alternative sections obtained from the multi lift analysis are discussed under the Case B.

5.1.1 Single Stage Analysis (Case A)

Deformation and stress contours of the single lift analysis at the end of construction and reservoir full stages are given at Fig.5.1 (a & b) respectively. From the contours and the analysis results, the following inferences can be drawn:

- The maximum vertical deformation at the end of construction stage occurs at the crest of the dam and the deformation is 0.94m.The deformation continuously increased from the base of the dam to the crest. The vertical deformations reduced at the full reservoir stage and the maximum deformation is 0.90m.The maximum deformation still seen at the crest of the dam though the maximum deformation contour slightly moved upstream side of the core.

- At the end of the construction stage, the horizontal displacements in the upstream shell portion vary from negative to positive from upstream slope towards the core and in the reverse way in the downstream shell area. The maximum positive displacement of 0.12m is observed in the downstream shell portion and the maximum negative displacement of -0.11m occurred in the upstream shell portion. Both the maximum displacements occur at a height of 50m from the base of the dam. At the full reservoir stage, the total dam experiences downstream side movement except upstream side toe of the dam which shows slight upstream side movement of a small magnitude. The maximum displacement of 0.34m is observed at the upstream side slope at a height of 130m from the base of the dam.
- From the contours of the major principal stress at the end of construction stage, it is observed that the stresses in the core are lower as compared to the shell material at all the levels except near the crest of the dam. The maximum principal stress (compressive stress) in the upstream shell, core and downstream shell are 358 t/m^2, 154 t/m^2,334 t/m^2 respectively and observed at the base of the dam. At the reservoir full stage, the maximum principal stress (compressive stress) in the upstream shell, core and downstream shell portion(at the reservoir full stage) are 337 t/m^2, 170 t/m^2, 336 t/m^2 respectively. Thus, at reservoir full stage major principal stresses slightly reduced in the upstream shell portion due to buoyancy of upstream shell and the stresses increased in the core as well as in downstream shell portion due to water pressure on the core.
- Similar to major principal stresses, the minor principal stress showed reduction in the upstream shell, and increased in the core as well as downstream shell portion in reservoir full stage. The maximum minor principal stress (compressive stress) in the upstream, core and downstream shell portion at reservoir empty and full stages are 185 t/m^2, 87 t/m^2,

161 t/m^2 and 129 t/m^2, 90 t/m^2, 173 t/m^2 respectively. The maximum values are observed at the base of the dam. Tension zone is observed in the upstream slope at about 0.6H at the full reservoir stage and the maximum tensile stress is $32t/m^2$.

5.1.2 Multi Lift Analysis Results (Case B)

5.1.2.1 Displacements

Fig: 5.2 presents the vertical deformation contours of all the three sections at the end of construction stage. In the central, moderately inclined and inclined core dam sections, the maximum vertical displacement took place within the core at a height of 80m from the base of the dam and the deformations are 0.85 m , 0.88 m and 0.94 m respectively. Fig 5.3 shows the vertical deformation contours at the full reservoir condition and the maximum vertical deformations in the three sections are 0.75m, 0.80m and 1.08m respectively. The deformations reduced in central core and moderately inclined core section due to reservoir filling but the same is increased in the inclined core section. The reduction in the first two cases may be due to buoyancy of the upstream shell material where as in the inclined core dam, the water pressure on the core dominates the buoyancy of the upstream shell material hence the increase. Fig. 5.4,5.5&5.6 shows the vertical displacement variation with respect to the height in the core, upstream and downstream shell potions of the central, moderately inclined and inclined core dams respectively. In all the three sections, the deformations in the core are found to be higher than that of the shell portion due to high compressibility of the core material.

Fig 5.7 and 5.8 presents the contours of horizontal deformations at the end of construction and reservoir full stage for the three sections. In all the three sections, at the end of the construction stage, the horizontal displacements in the upstream shell portion vary from negative to positive from upstream slope towards the core and in the reverse way in the downstream shell portion. These displacements in the three sections remain more or less equal and the values are 0.16m, 0.12m and 0.12m in central core, moderately inclined and inclined core sections respectively. The maximum displacement appears at mid height just upstream side of the core in the central and moderately inclined core sections but within in the core in the case of inclined core section. At the full reservoir stage, the total dam experiences down stream side movement except upstream side toe of the dam in all the three cases. These maximum displacements are observed to be 0.5m, 0.42m, and 0.46m in the three sections respectively and occurred upstream side of the core almost at the mid height of the section. Fig. 5.8(a), 5.8(b) & 5.8(c) presents the horizontal displacement variation along the height of the dam in the core, upstream and downstream shell portions of the three sections. From the graphs it can be deduced that the central core continuously moves down stream side in the construction as well as in reservoir filling stages where as in remaining two cases the core moves first upstream side and then to the down stream side.

As far as static vertical displacements at the full reservoir condition are concerned the central core dam appears to be slightly better than other alternatives. Horizontal displacements in all the three cases are practically same.

5.1.2.2 Stresses at the End of Construction

Fig 5.9 shows the major principal stress contours for the three dam sections. The principal stresses in the core are lower as compared to the shell material at all the levels except near the crest of the dam. This happens due to load transfer from core to shell as the stiffness of the core is less than the shell. However, this load transfer shows an increasing trend from central core to inclined core dams.

Fig 5.10 shows variation of load transfer ratio ($\sigma_1/\gamma_t h$) with respect to height of the dam in the three cases. Excess load transfer from core to the shell may lead to hydraulic fracturing as discussed in chapter 3 .Out of the three alternatives, the load transfer ratio in case of the central core dam is comparatively close to 1 indicating less load transfer from core to shell. In this respect, central core dam has slight advantage over the other two alternatives.

The maximum principal stresses (compressive stress) in the core portion of the central, moderately and inclined cores are 180 t/m^2, 195 t/m^2,195 t/m^2 respectively and appear at the base of the core. The maximum principal stress (compressive stress) in the upstream shell show steady rise from central core to inclined core dams and the respective values are 350 t/m^2, 370 t/m^2 and 400 t/m^2. However, small zones of higher stresses are observed in all the alternative sections.

The Fig 5.11 shows contours of minor principal stresses in the three sections. Small tension zone (tensile stress of the order of 1t/m^2) is observed in the moderately inclined and inclined core dams at upstream slope portion. The maximum stress (compressive stress) values in the central, moderately and inclined cores are 80 t/m^2, 88 t/m^2, 86 t/m^2 respectively. The principal stress in shell portion also shows steady increase from central core to inclined core sections.

Fig 5.12 presents shear stresses contours in the three alternative dams. Out of the three, the central core dam experiences the least shear stresses in the core as well as in upstream shell portion. However, higher stresses are observed in the downstream shell of the central core dam. Fig 5.13 shows the variation of shear stresses in core, upstream and downstream shell with respect to height in the three alternative sections. The maximum negative shear stresses in the upstream shell and positive shear stress in the downstream for the central core, moderately inclined and inclined sections are -46 t/m^2 ,-60 $t/m^2$and -80 t/m^2 and 49 t/m^2,42 t/m^2 and 54 t/m^2 respectively.

To judge the safety of the dam, the mobilisation ratio can be treated as an indicator which gives ratio of mobilised shear strength to shear strength at failure as explained in the Chapter 3 and for a safe design mobilisation ratio should be around 0.5 to ensure a factor of safety of 2.0. Mobilisation ratios in core ,upstream shell and downstream shell in all the three alternatives are shown in Fig:5.14(a to e).In the core, the mobilisation ratio remains low in case of central core dam as compared with other two alternatives. The Fig 5.14(a) clearly shows the advantage of the central core dam in this respect over the other two alternatives. The mobilisation ratio in the upstream shell portion is observed to be on the lower side in central and moderately inclined core sections .The ratio is found to be high in downstream shell portion in central and moderately inclined core sections.

From the above discussion for end of construction stage it can be seen that the principal stresses, shear stresses are comparatively less in the central core dam and the mobilisation ratio in the core is also relatively less. From the safety consideration of core, central core position appears safe but in this case the higher shear strength of downstream shell material will have to be ensured.

5.1.2.3 Stresses at the Reservoir Full Condition

Fig 5.15 and 5.17 show the contours of the major and minor principal stress for full reservoir condition.

In all the alternatives, the major principal stress reduced in the upstream shell portion due to buoyancy of the upstream shell and water pressure on the impervious core. The stresses remained practically unchanged in the core and downstream shell in all the alternatives. Fig 5.16(a,b&c) shows the major principal stress variation from the end of construction stage to full reservoir stage in the core, upstream and downstream shell portions of the three alternative sections.

In all the three sections, the minor principal stress reduced in the upstream shell and increased in the downstream shell portion as well as core of the dam due to buoyancy of the upstream shell and water pressure on the impervious core. Fig 5.18 (a,b&c) shows the minor principal stress variation from the end of construction stage to full reservoir stage in the core, upstream and downstream shell portions of the three alternative sections. From the minor principal stress contours it is observed that the tension zone appears in all the three sections but the tensile stress value increased from central core to inclined core section (3 t/m^2 to $7t/m^2$).

Fig 5.19(a) shows the shear stress contours at full reservoir condition in three alternatives. The maximum shear stresses in upstream and downstream shell have shown some increase in all the three sections. Fig 5.19(b) shows the variation of shear stresses in core, upstream and downstream shell with respect to height in the three alternative sections. The figures clearly show the core and upstream shell portion of the central core dam experiences less stresses in comparison with other two alternatives.

The mobilisation ratio in the core portion is less in the case of central core dam even at the full reservoir condition. Fig 5.14(b) shows the comparison of the mobilisation ratio variation in the cores of three sections. The Fig:5.14 (d&f) shows the variation of the above ratio in the upstream and downstream shell portions. The mobilisation ratio in the downstream shell portion is also observed to be on the lower side in case of central core and moderately inclined core sections than the inclined core section although the ratio is higher in case of upstream shell portion. From the consideration of mobilisation ratio for the core the central core section may be preferred but higher shear strength of upstream shell at certain locations will have to be ensured.

Fig 5.20 (a & b) shows the variation of the minimum hydraulic fracture potential ratio (σ_1/u_h , σ_2/u_h) along the height of the dam. In the Fig 5.20 (a), the hydraulic fracture potential ratio is higher than 1 at all the heights which indicate that all the alternatives are safe against horizontal cracking. The ratio in Fig:5.20 (b) is less than 1 up to a height about 100 m. It shows the susceptibility of the cracking along the vertical plane. Since the computations do not take into account the tensile strength of core material, it is expected that vertical cracking will not take place. If it happens the upstream filter will take care of these cracks.

In the full reservoir condition , which is more critical than the reservoir empty condition, the principal stresses and shear stresses in the dam have shown steady increase from central core dam to inclined core dams. The mobilisation ratio in the core as well as downstream shell also found to be less in the central core dam. From this, it can be concluded that the static stress distribution favours central core section over the other two alternatives.

5.2 DYNAMIC ANALYSIS

In the dynamic analysis, following studies have been carried out to compare the performance of the three alternative sections:

1. Free vibration characteristics of the sections.
2. 2D linear dynamic analysis of the sections

The results of the analysis are discussed here under:

5.2.1 Mode Shapes

The Fig 5.21 shows the mode shapes of the three dam sections in the first four modes.

5.2.2 Frequency

Natural frequencies of the three dam sections for the first eight modes are shown in the table 5.1. Out of the three sections, the central core section has lower frequencies except in the second and eighth mode in which inclined core section has slightly lower frequency.

Table:5.1 Natural frequencies of the three sections

Mode No.	Natural Frequency (radians / sec)		
	Central Core	**Mode. Inclined**	**Inclined**
1	3.7	4.10	4.11
2	4.45	4.26	4.25
3	5.42	6.28	6.28
4	5.56	6.83	6.61
5	6.76	8.16	7.84
6	7.31	8.38	8.27
7	7.85	8.73	8.38
8	8.48	8.98	8.43

5.2.3 Displacements and Acceleration

In the dam, the maximum displacements and acceleration due to earthquake occur at the crest of the dam. The time history of displacement, accelerations (in horizontal and vertical directions) at the crest portions of all the three alternative sections are presented in Fig: 5.22 (a to f). The table:5.2 presents the maximum values of different parameters of the three sections.

Table:5.2 Maximum displacements and accelerations in three alternative sections

Parameter	Central Core	Mod Inclined	Inclined
Hor. Displacement (m)	0.27 to -0.28	0.24 to -0.21	0.20 to -0.21
Hor. Acceleration(m/sec^2)	8.77 to -8.19	9.17 to -9.46	8 to -9.14
Ver. Displacement (m)	0.15 to-0.17	0.13 to -0.11	0.08 to -0.09
Ver. Acceleration(m/sec^2)	5.6 to -4.85	6.3 to -5.76	5.5 to -5.27

From the above table it is clear that the horizontal and vertical displacements due to dynamic load show decreasing trend from central core to inclined core sections. The central core section has minimum acceleration in horizontal direction and inclined core experiences minimum acceleration in vertical direction, though the variation of the acceleration amongst the three sections is very marginal.

5.2.4 Stresses

For studying the combined stress (Static + Dynamic stresses) variation in the dam section, 3 elements one each in core, upstream shell and downstream shell at the base of the dam are considered (element nos.152, 11 and 67 respectively). The time-histories of the combined principal stresses in respect of the above elements pertaining to the three sections are given at Fig 5.23 to 5.25. The maximum values of the combined principal stresses from the above figures are tabulated in the table 5.3.

Table:5.3 Maximum combined principal stresses in the three alternative sections

Stress in	Major principal stress (t/m^2)			Minor principal stress (t/m^2)		
	C	MI	I	C	MI	I
Core	207	236	244	101	110	113
U/S Shell	296	286	340	131	120	107
D/S Shell	335	350	354	149	174	176

All stresses are compressive only

C - Central core section

MI- Moderately Inclined core section

I - Inclined Core Section

General trend clearly indicates that the stresses increased from central core ıed core section except in the case of minor principal stress in the upstream 'urther variation of the mobilisation ratio in all the three elements is plotted э graphs are presented at Fig.5.26. The central core section has shown эge over other two sections as far as core and upstream shell are ned, however, the values are slightly higher in down stream shell portion.

Tension zones are formed at upstream slope of all the three sections during rth quake and the tensile stress values increased from central core to 1 core sections. Stress distribution contours in the three sections at the time ‹imum tensile stresses are given at Fig: 5.27.The tension zones in the ım slope may create local failures and some permanent settlements during thquakes and may not lead to total dam failure. From the above figures and ıe time history data, it is observed that in case of central core dam the um tensile stresses appeared at the junction of the core and shell portion could cause cracks along these junctions. In the earthen and rockfill dams 'ety of the core is very vital and any damage to this portion may lead to the am failure. In this respect, the central core section seems to be more ›tible to failure in seismic loading conditions and inclined core section could эtter option.

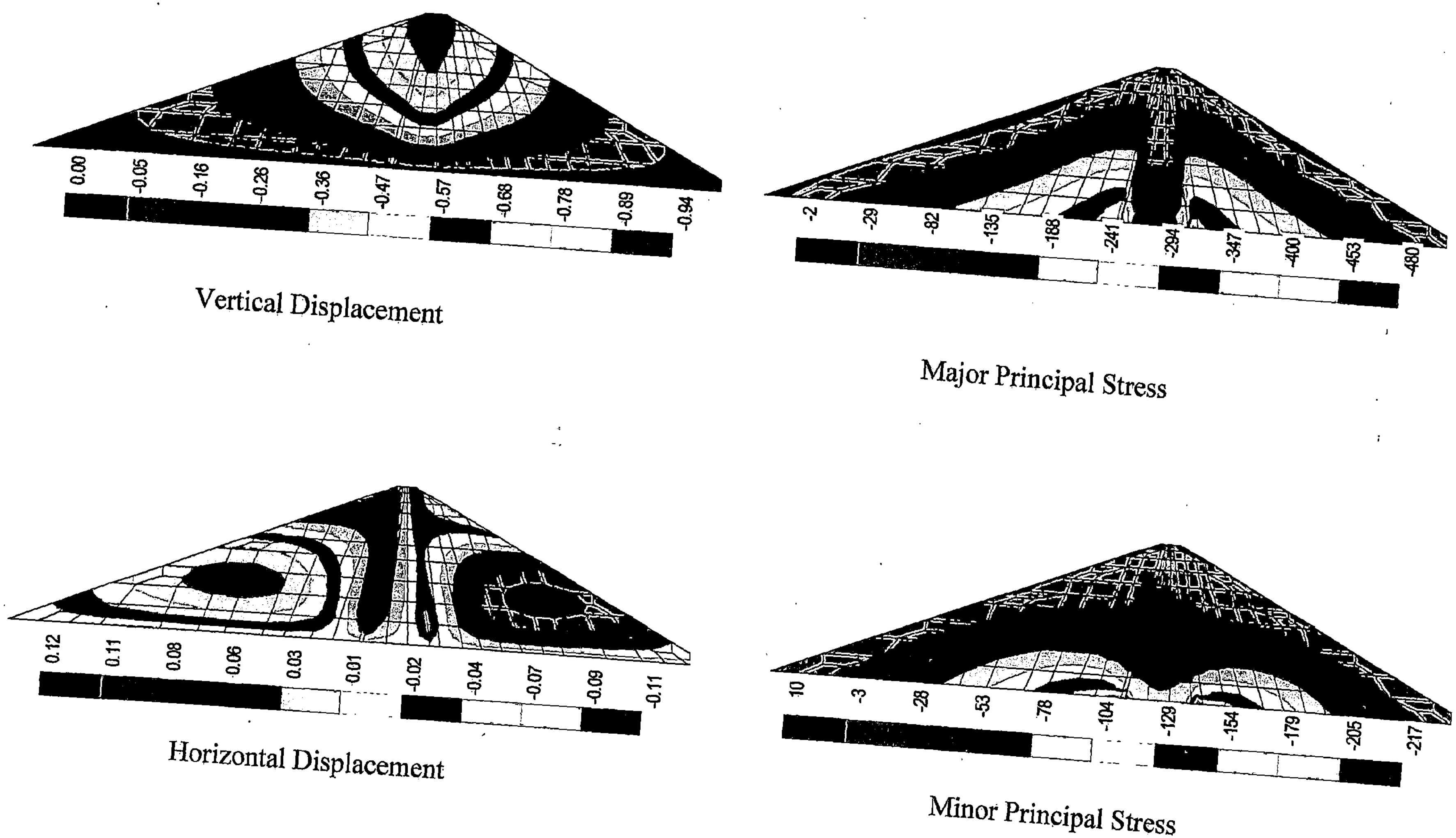
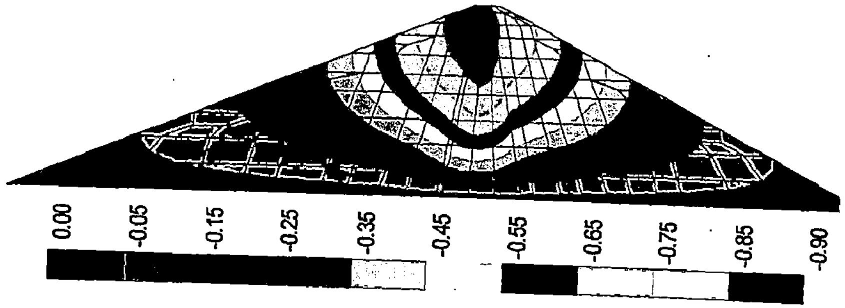
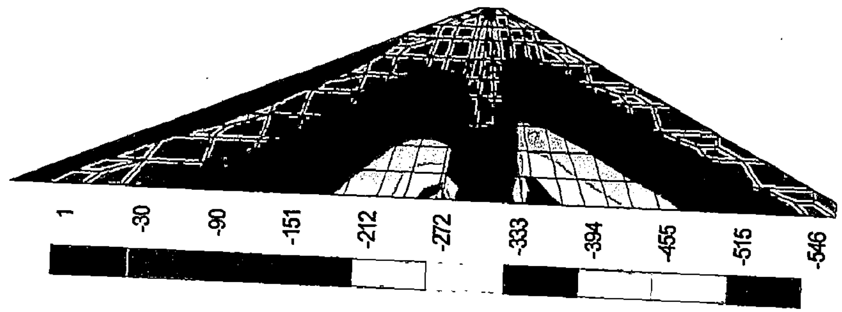


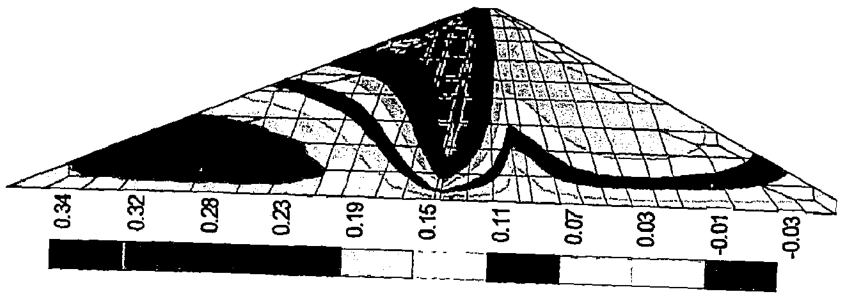
Fig 5.1(a) Contours of single lift analysis at the End of Construction



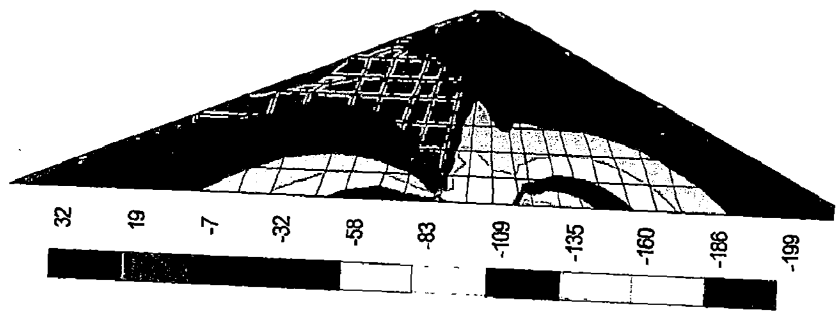
Vertical Displacement



Major Principal Stress



Horizontal Displacement



Minor Principal Stress

Fig 5.1(b) Contours of single lift analysis at Full Reservoir Stage

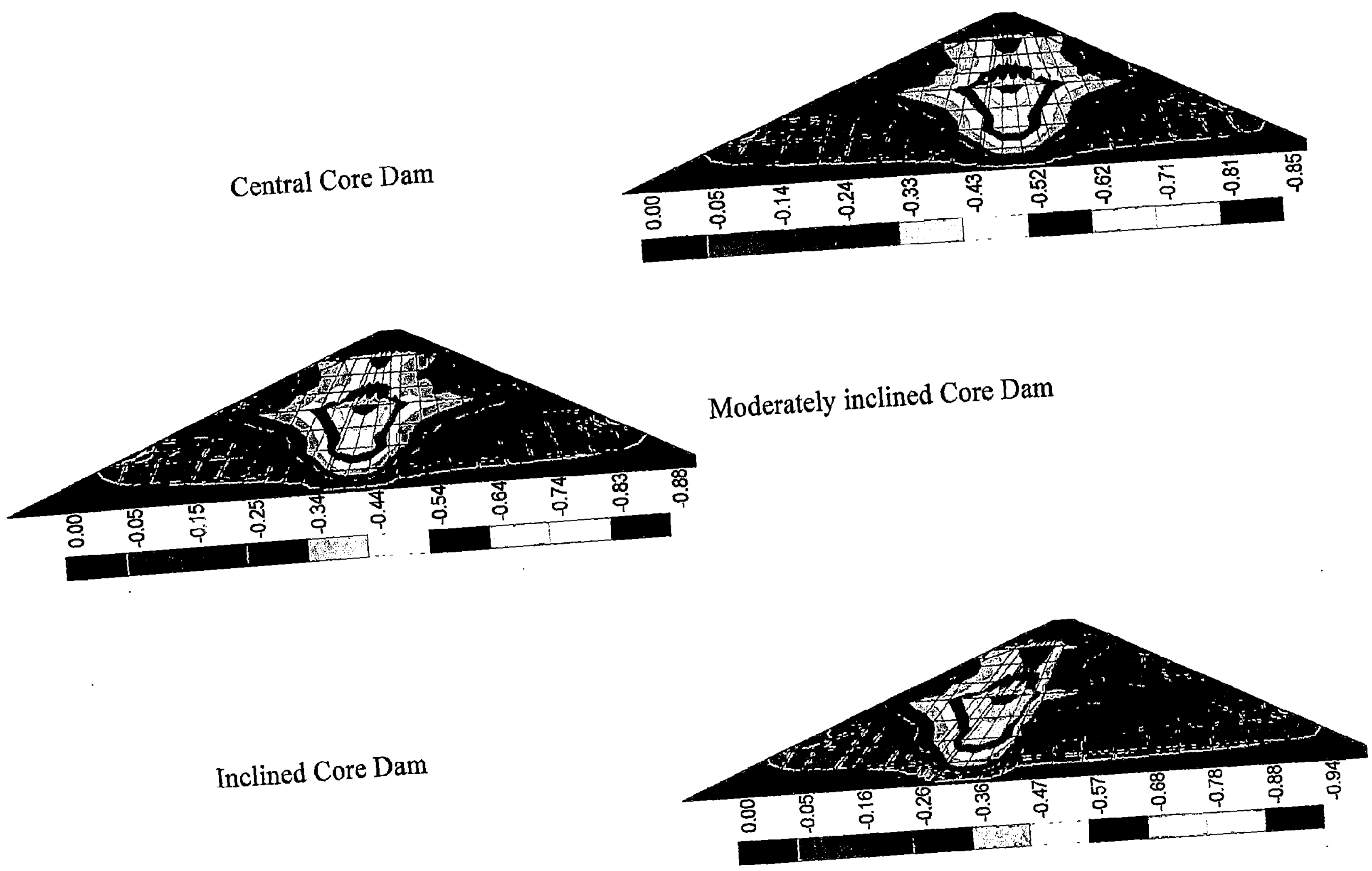
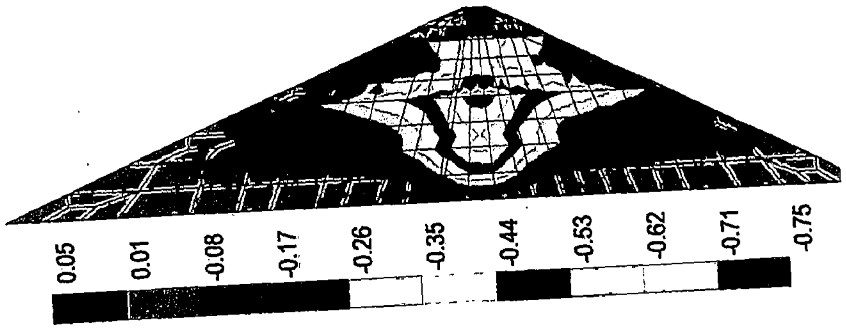
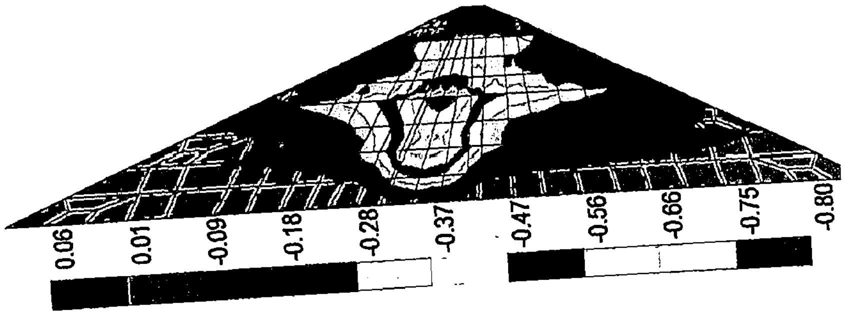


Fig 5.2 Vertical Displacement contours at the End of Construction

Central Core Dam





Moderately inclined Core Dam

Inclined Core Dam

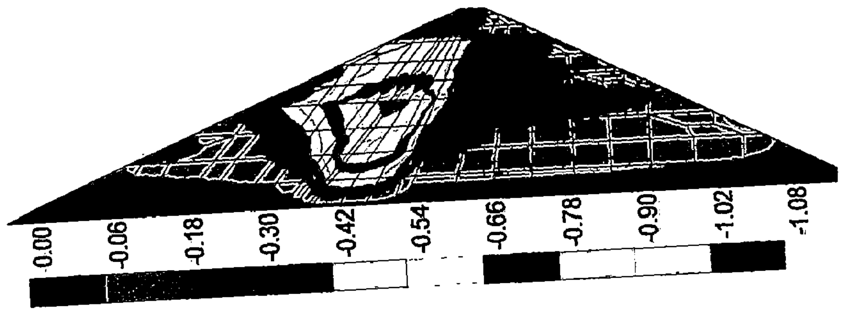
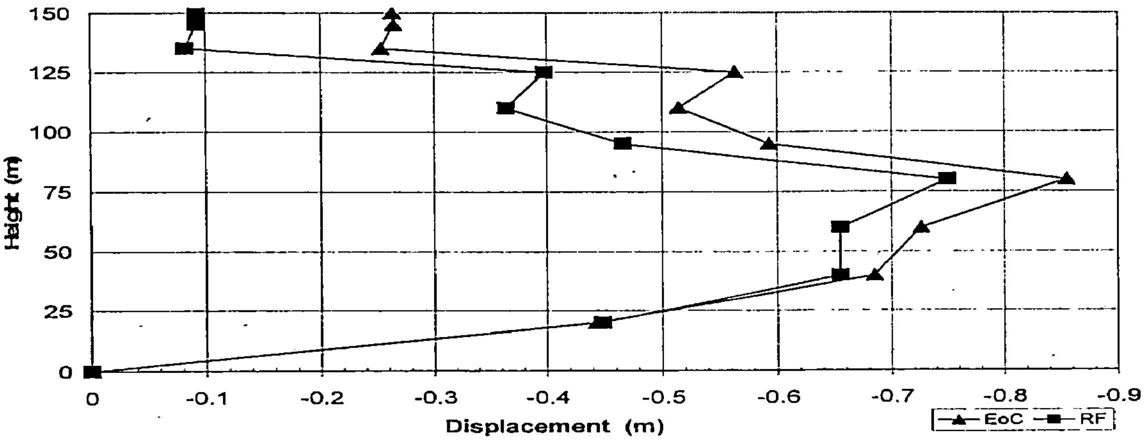
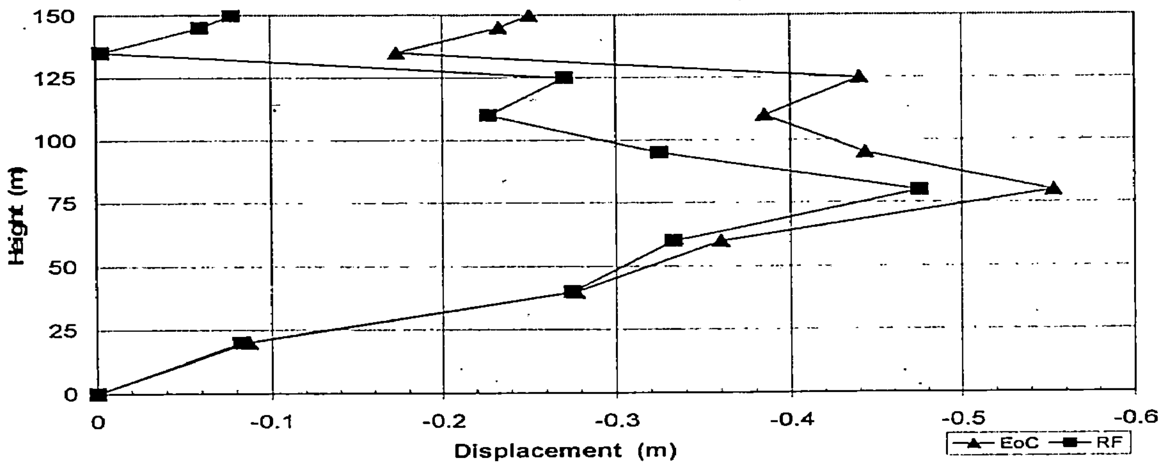


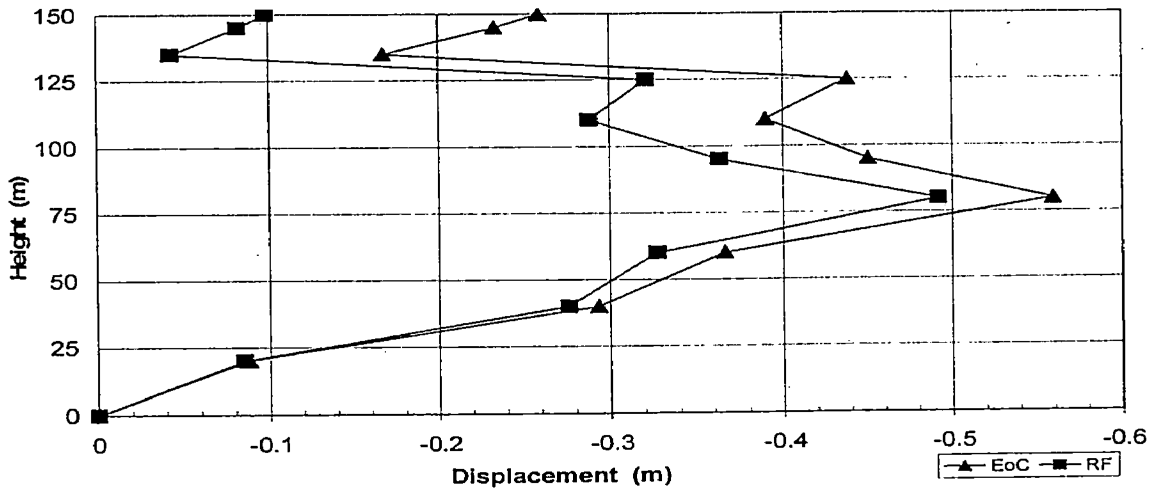
Fig 5.3 Vertical Displacement contours at Full Reservoir Stage



Core

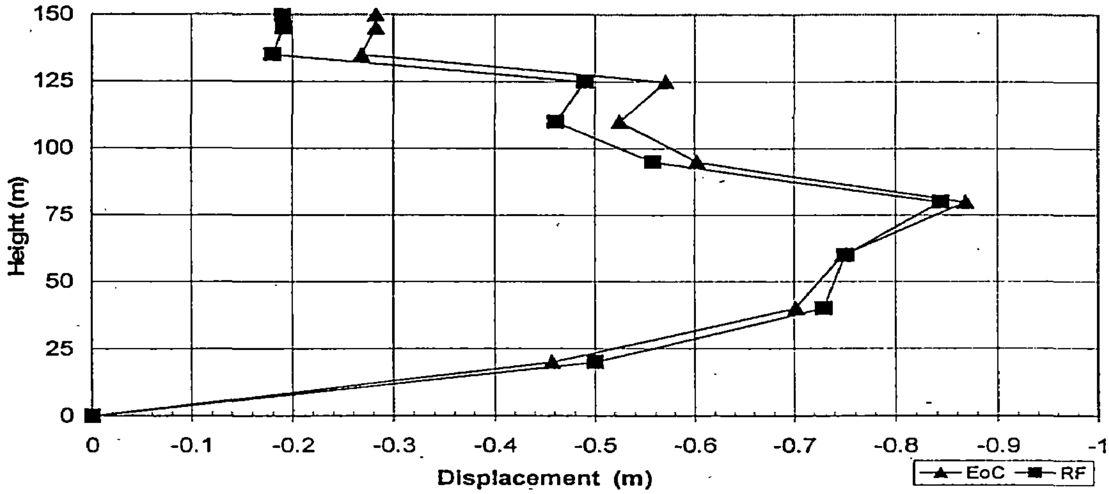


Upstream shell

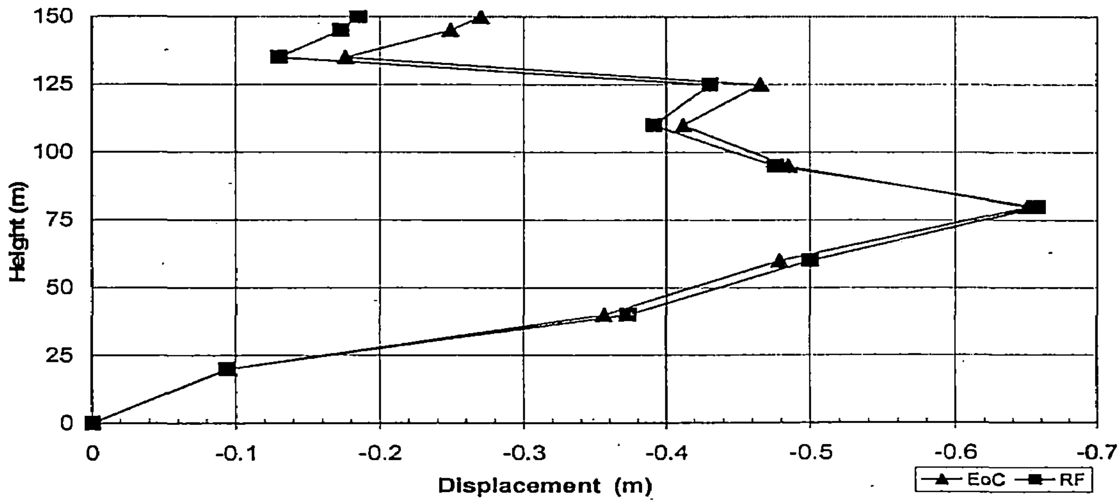


Downstream Shell

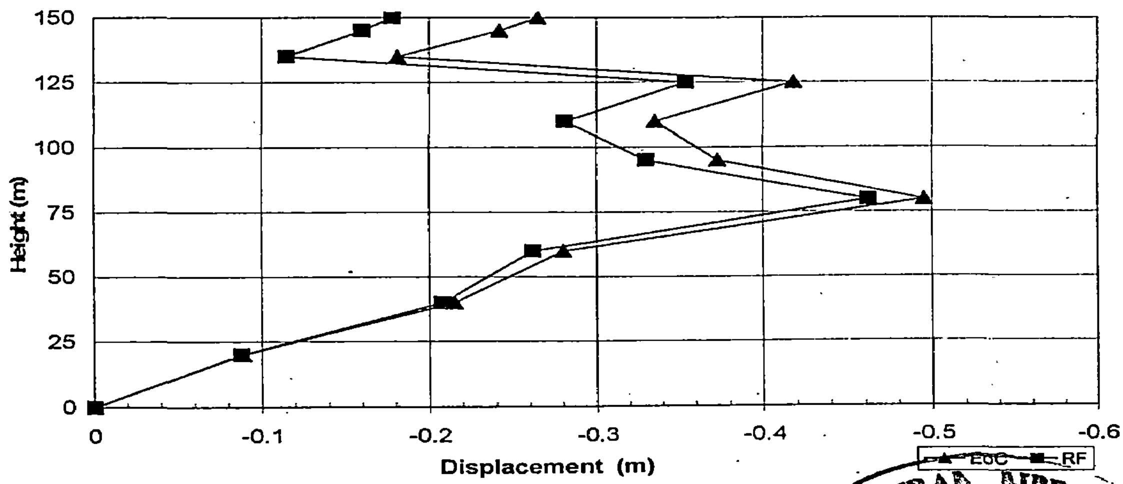
Fig: 5.4 Vertical displacements variation in the central core dam



Core

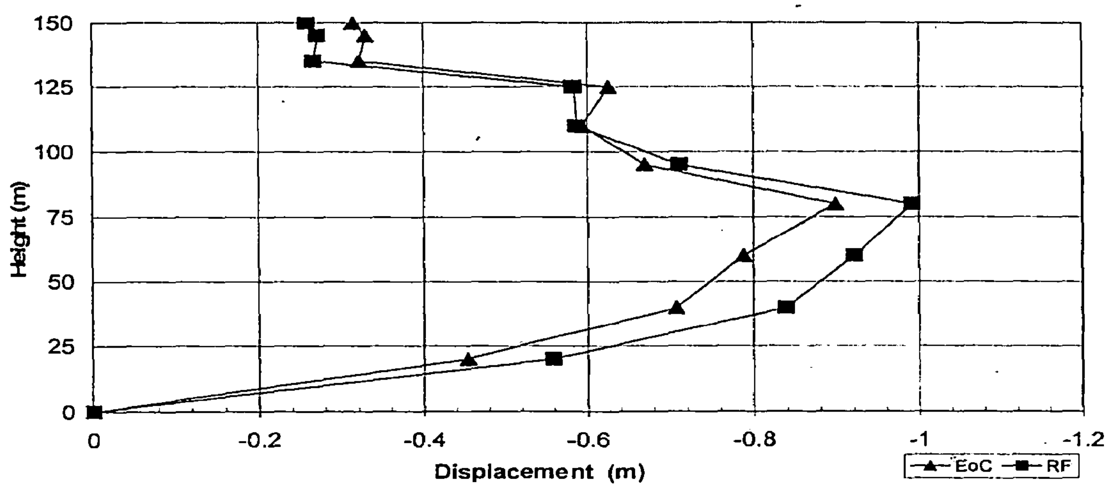


Upstream Shell

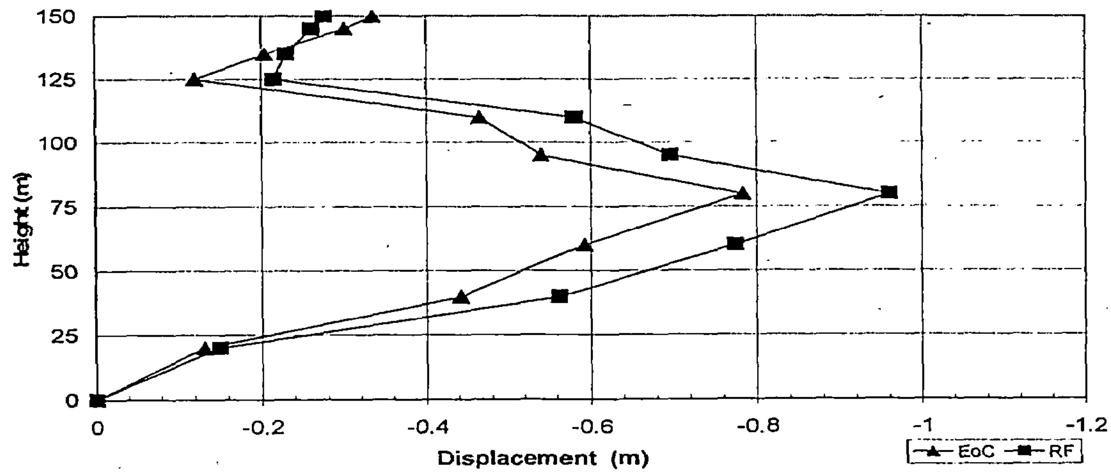


Downstream Shell

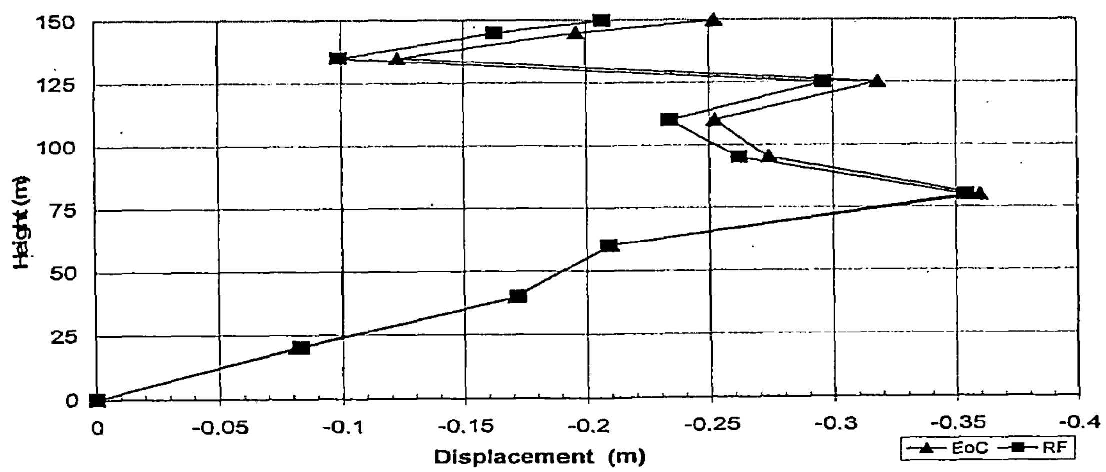
Fig: 5.5 Vertical displacements in the moderately inclined core dam



Core



Upstream Shell



Downstream Shell

Fig: 5.6 Vertical displacement variation in the inclined core dam

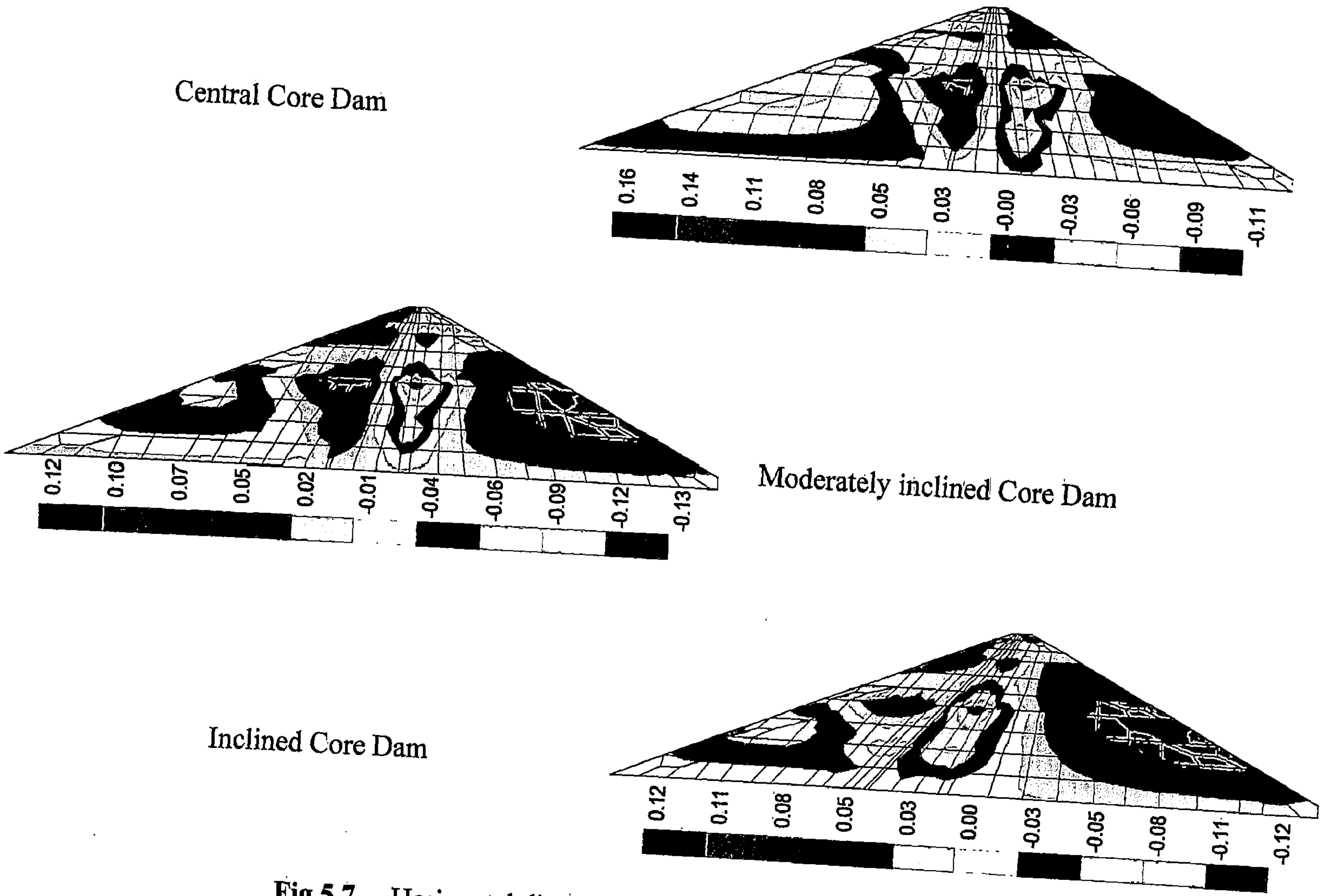
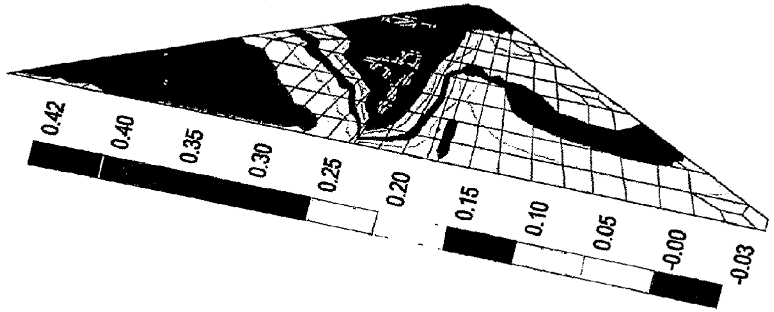
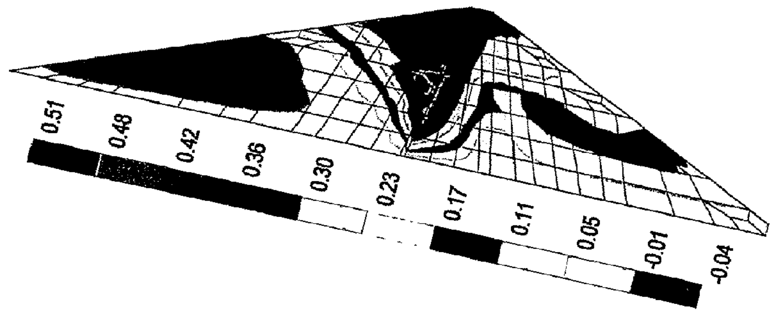


Fig 5.7 Horizontal displacement contours at the End of Construction

Central Core Dam





Moderately inclined Core Dam

Inclined Core Dam

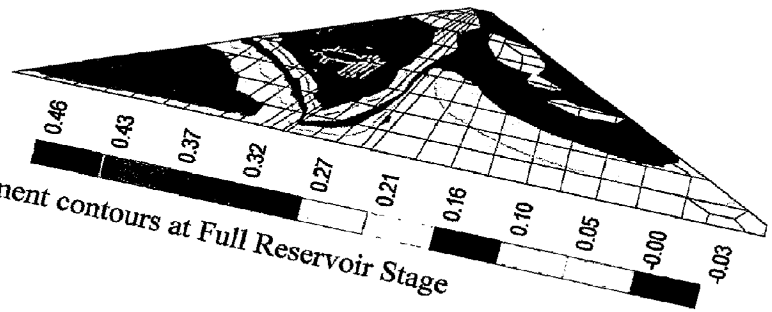
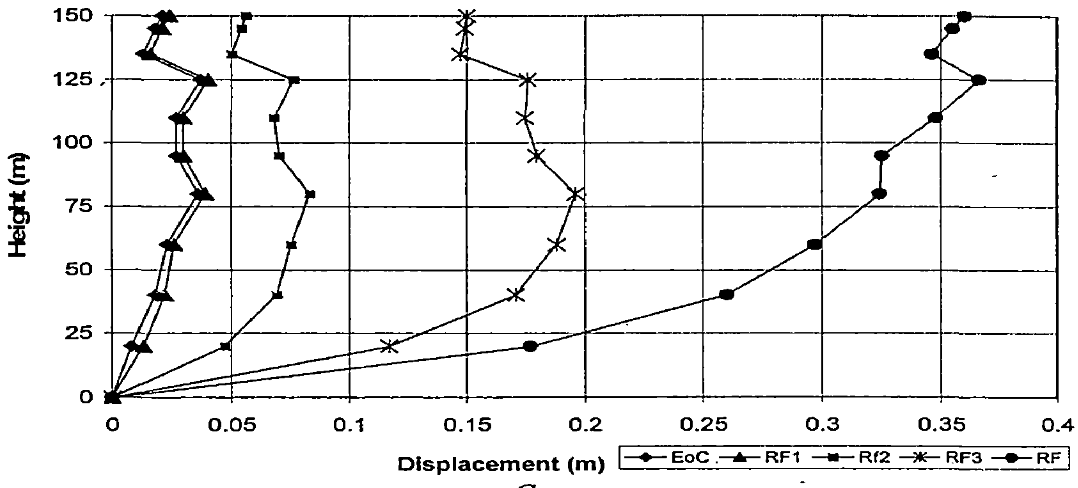
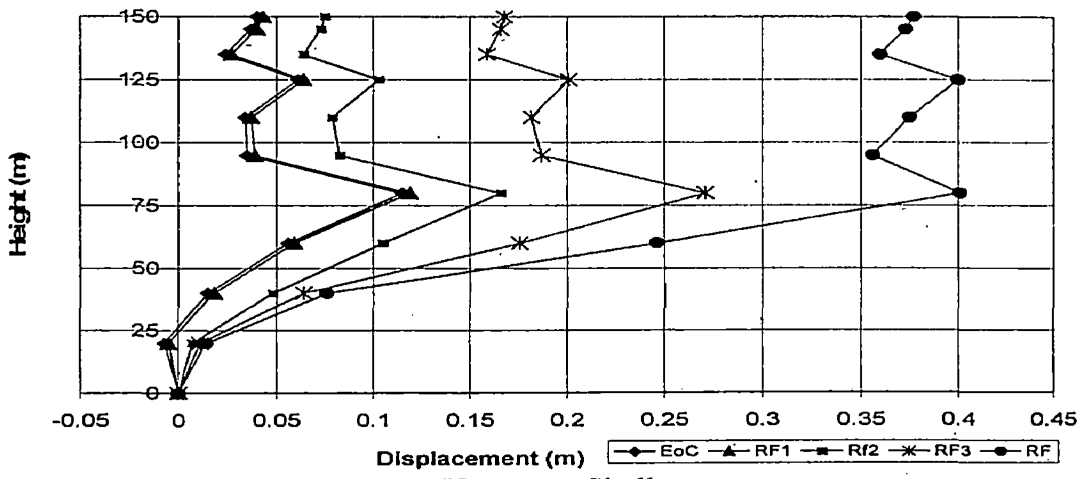


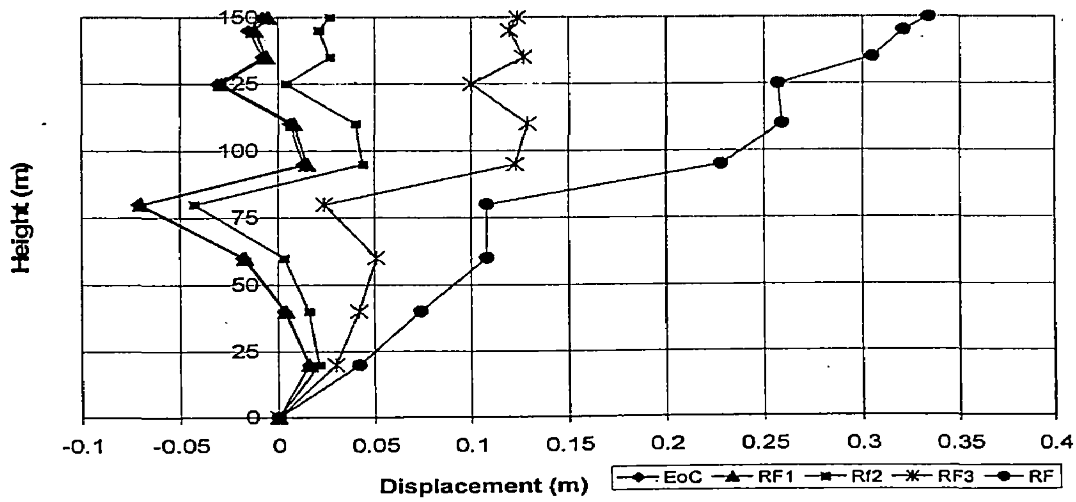
Fig 5.8 Horizontal displacement contours at Full Reservoir Stage



Core

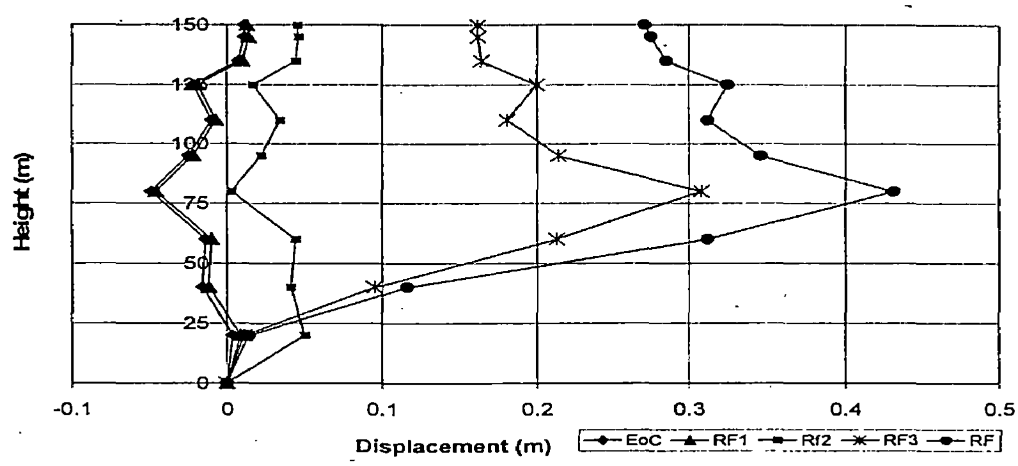


Upstream Shell



Downstream Shell

Fig: 5.8 (a) Horizontal displacement variation in the central core dam



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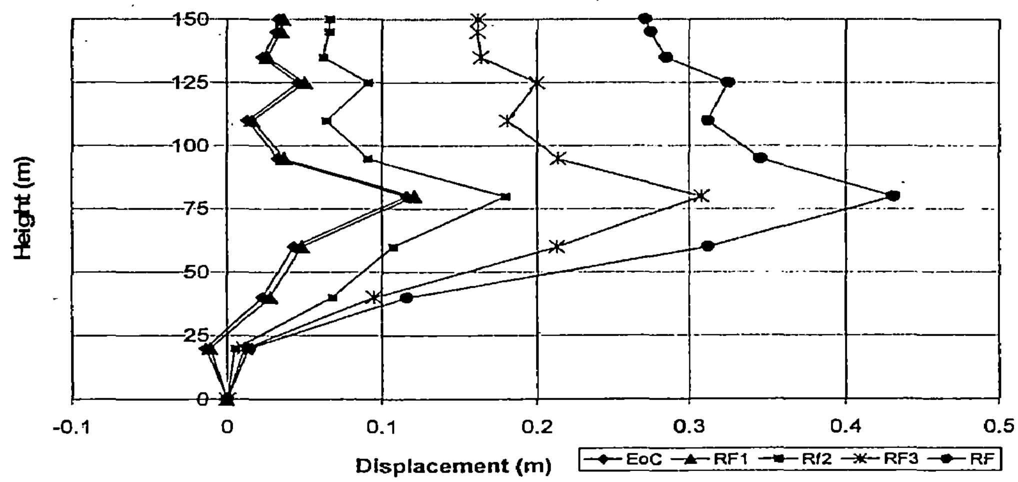
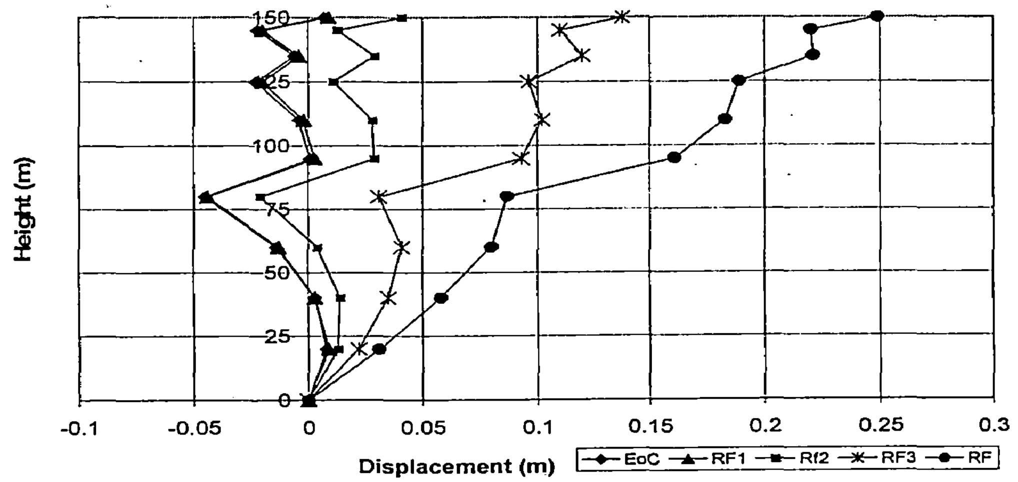
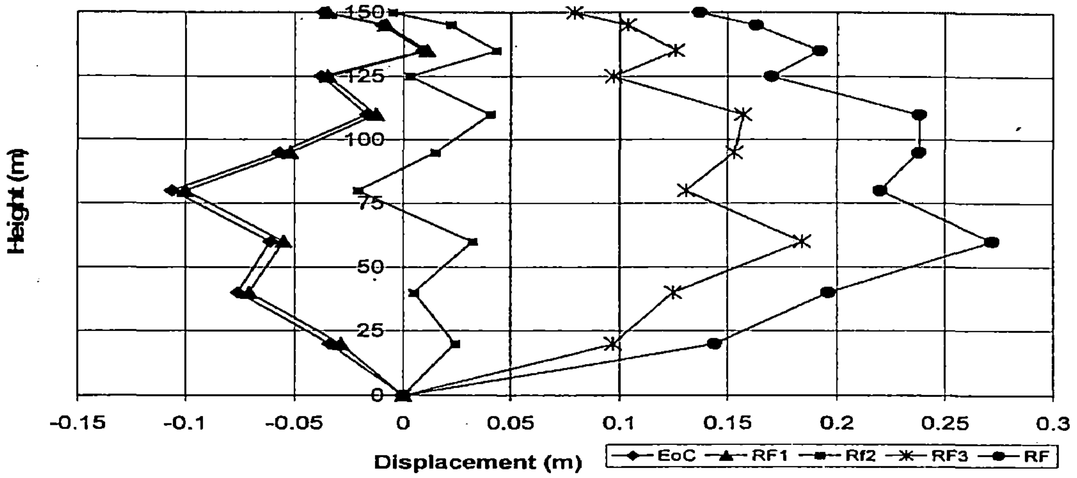


Upstream Shell

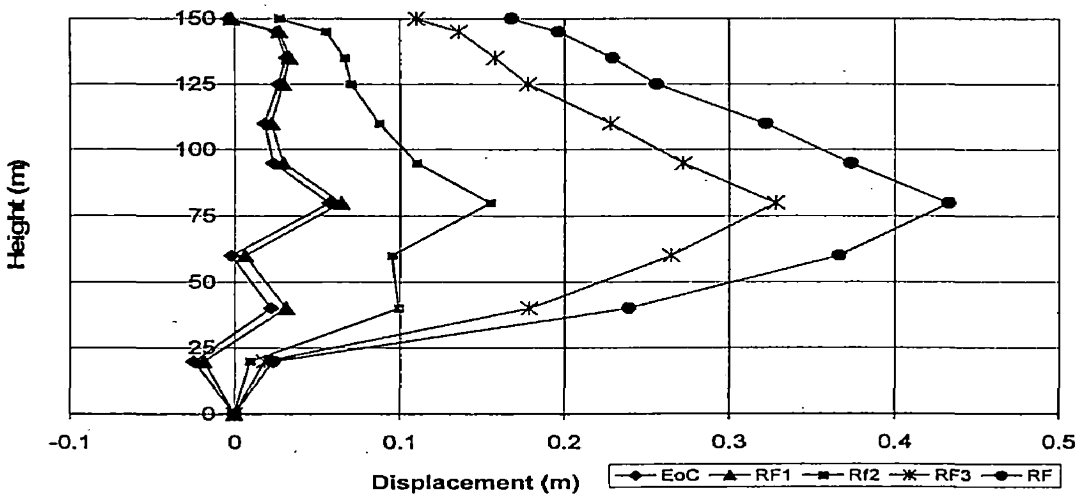


Downstream Shell

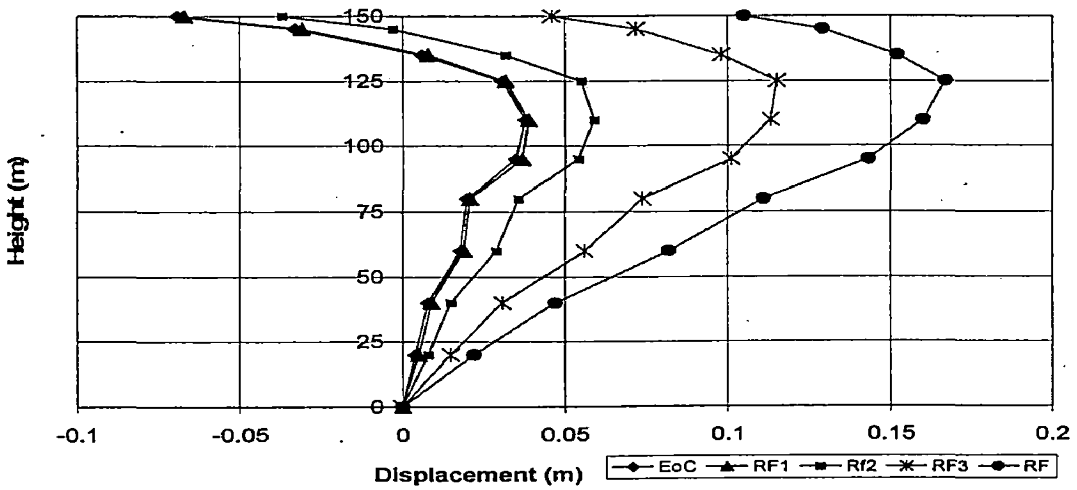
Fig: 5.8 (b) Horizontal displacement variation in the moderately inclined core dam



Core



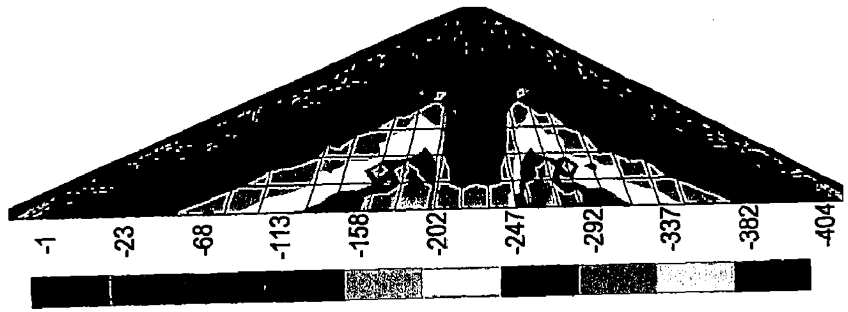
Upstream Shell



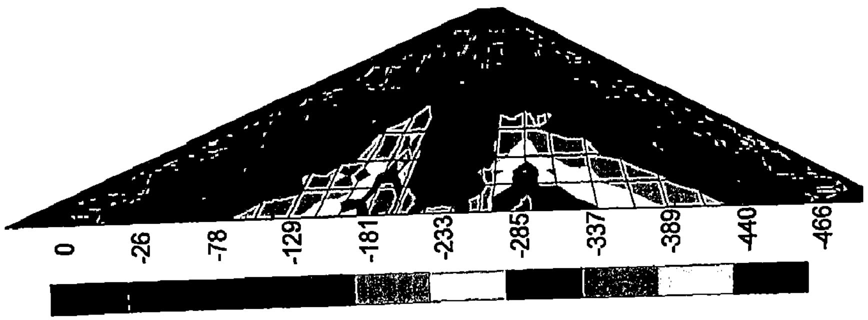
Downstream Shell

Fig: 5.8 (c) Horizontal displacement variation in the inclined core dam

Central Core Dam



Moderately inclined Core Dam



Inclined Core Dam

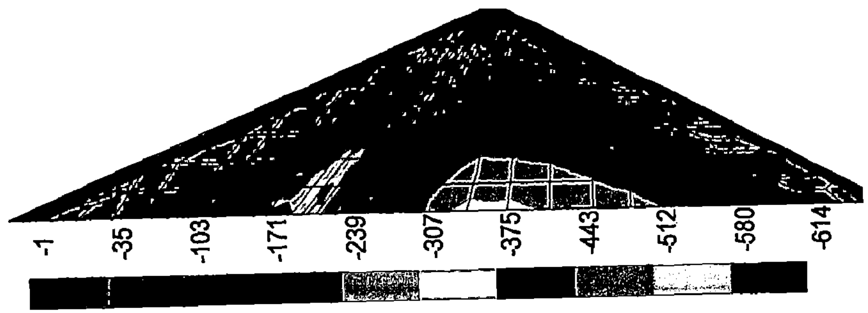


Fig 5.9 Major principal stress contours at the End of Construction

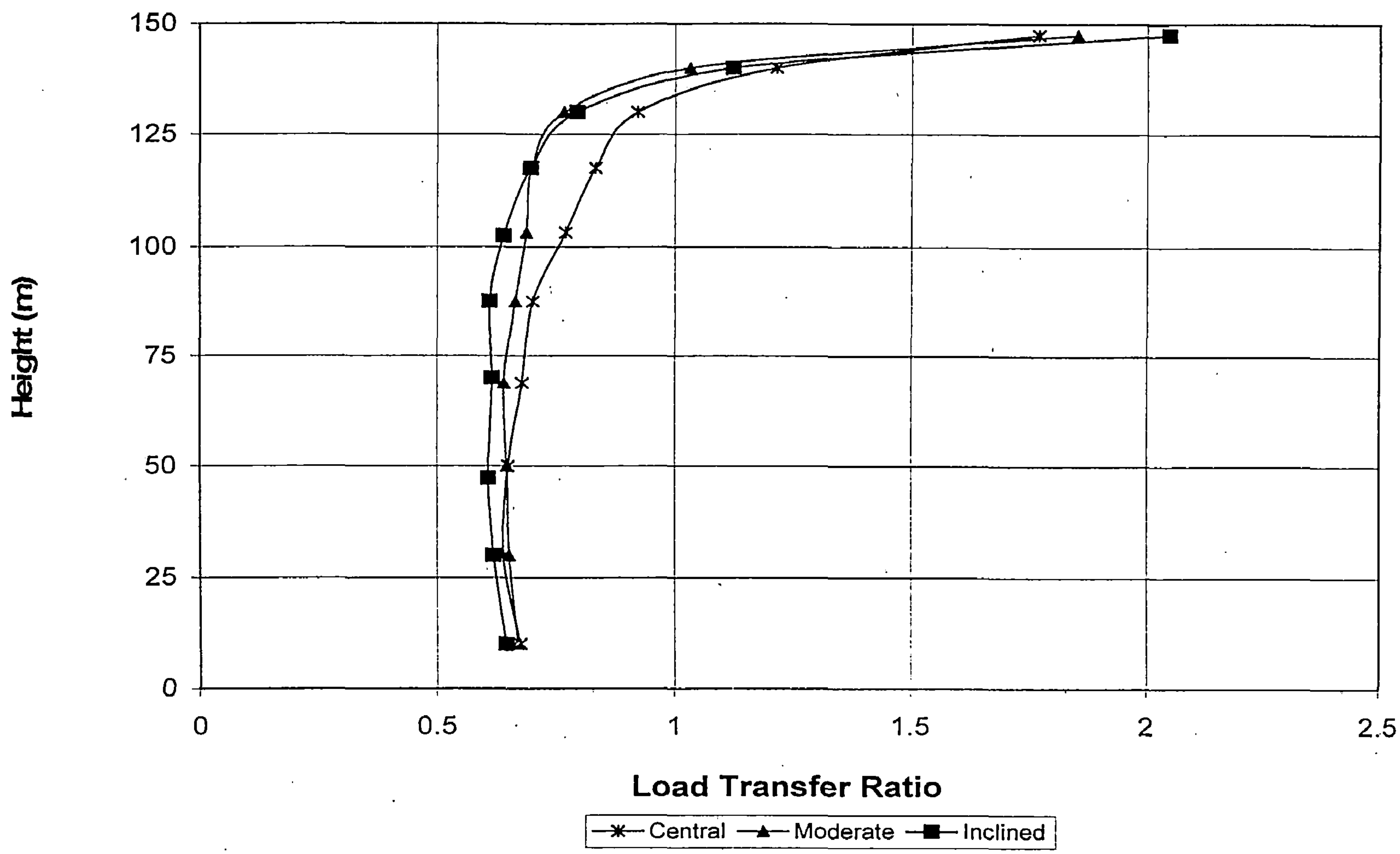
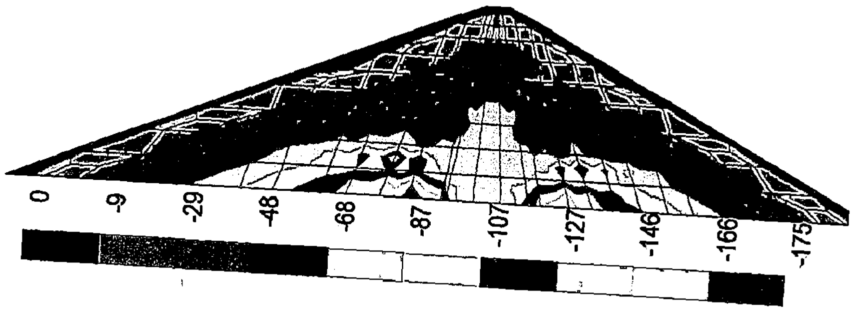
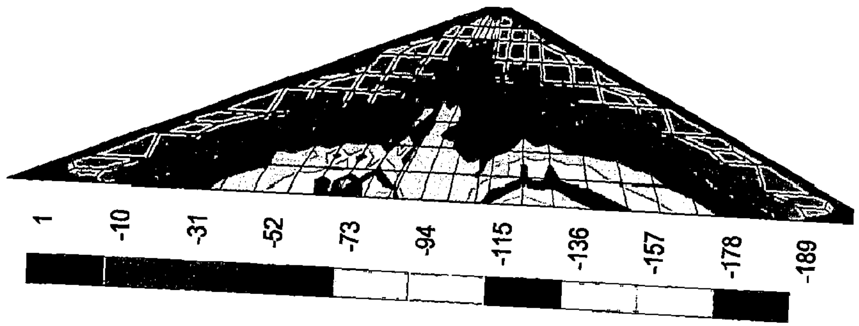


Fig:5.10 Variation of load transfer ratio for the three sections

Central Core Dam





Moderately inclined Core Dam

Inclined Core Dam

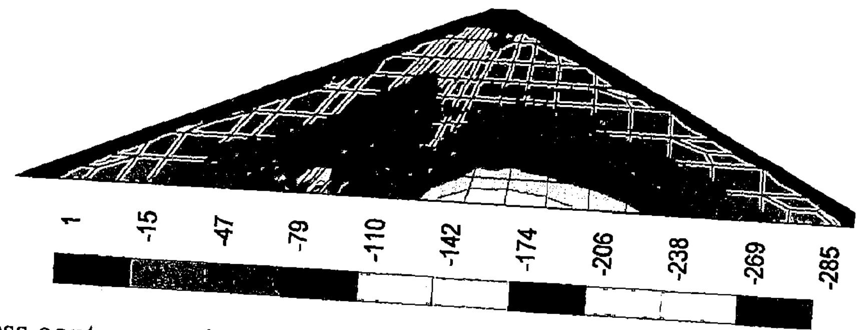
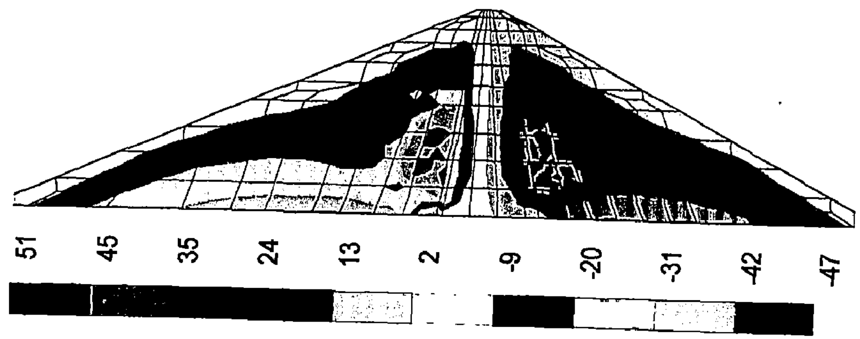
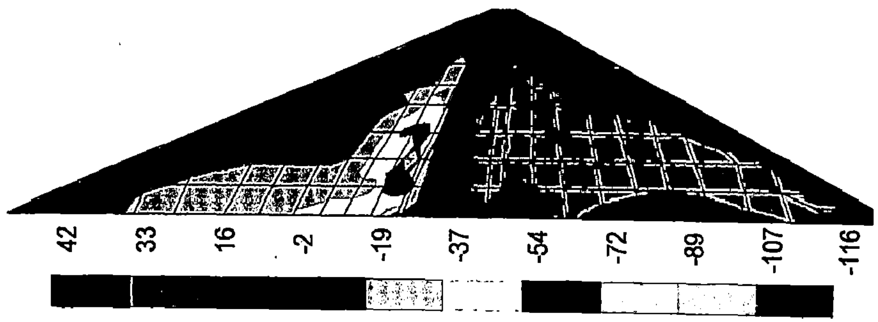


Fig 5.11 Minor principal stress contours at the End of Construction

Central Core Dam





Moderately inclined Core Dam

Inclined Core Dam

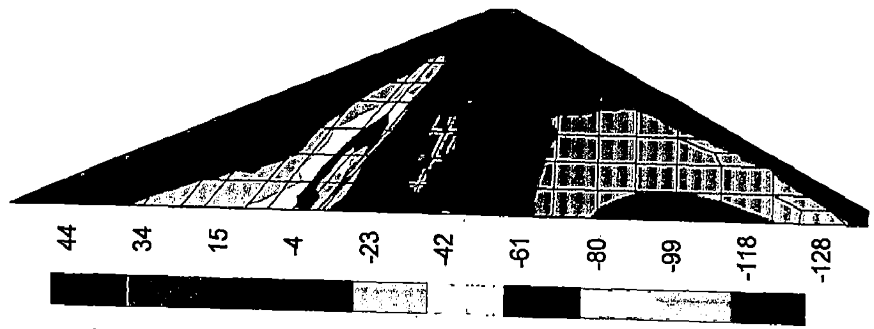
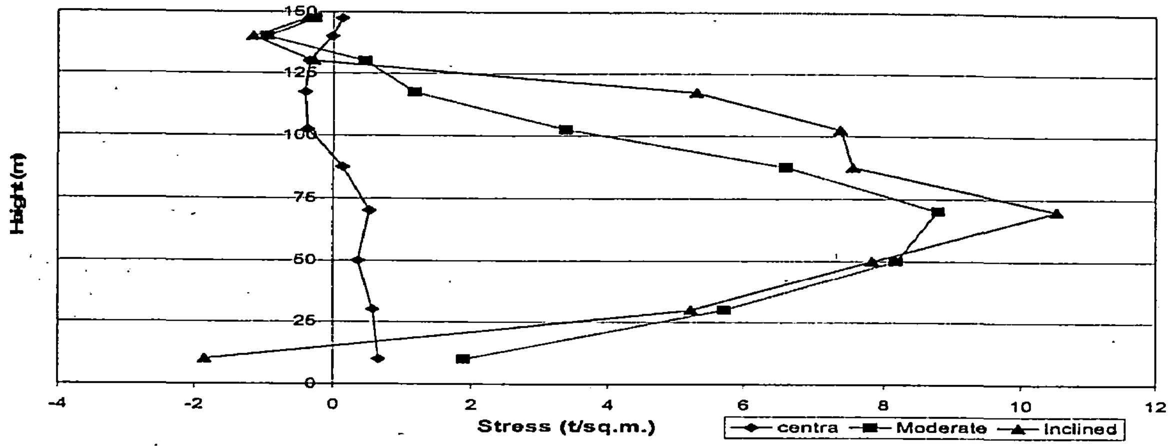
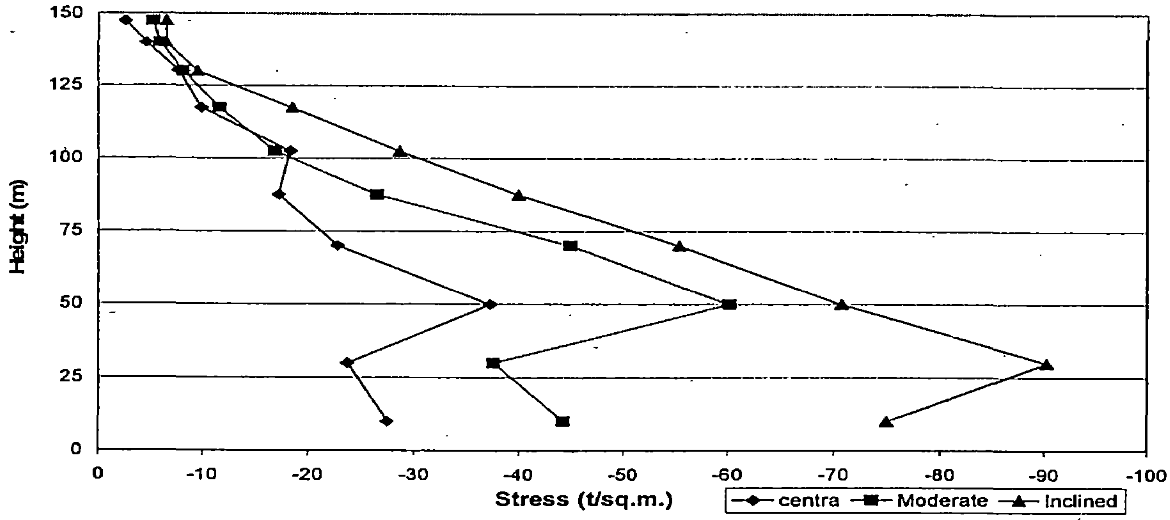


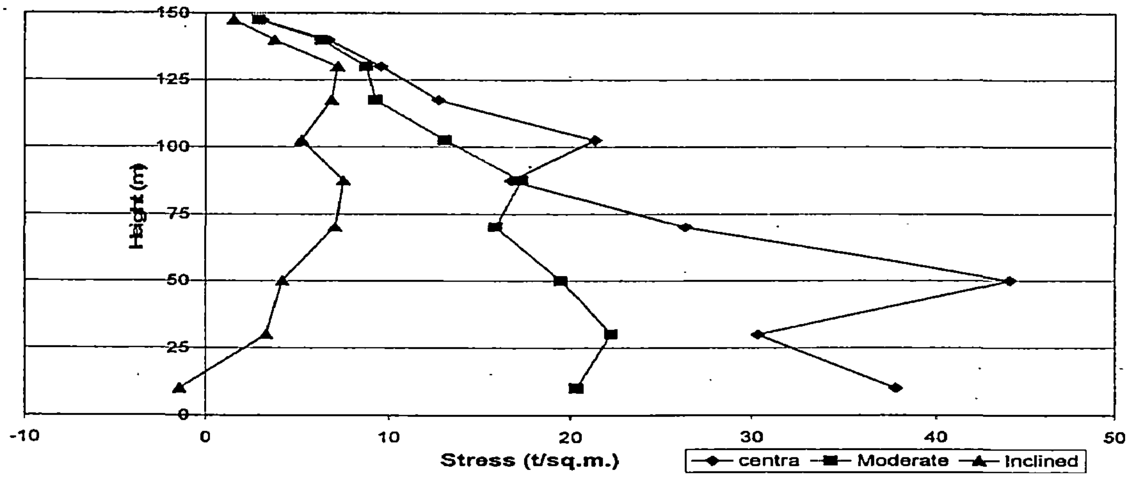
Fig 5.12 Shear stress contours at the End of Construction



Core



Upstream shell



Downstream shell

Fig:5.13 Shear stress variation in three dams at the end of construction

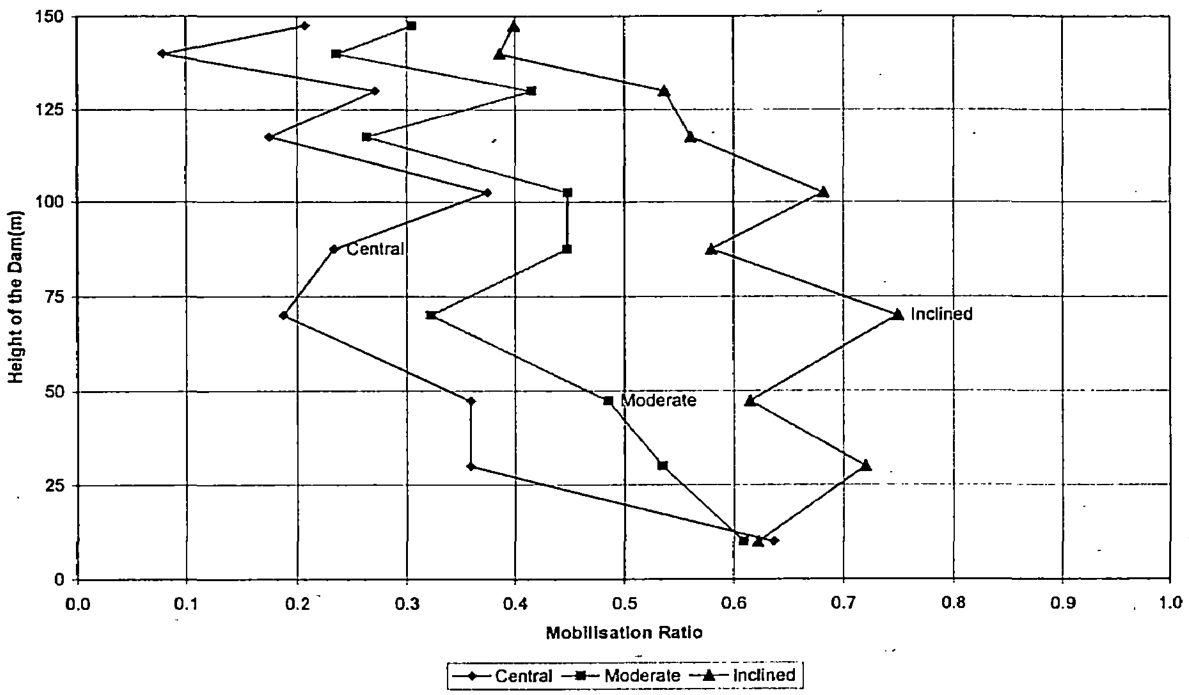


Fig:5.14(a) Mobilisation ratio in core at the end of construction

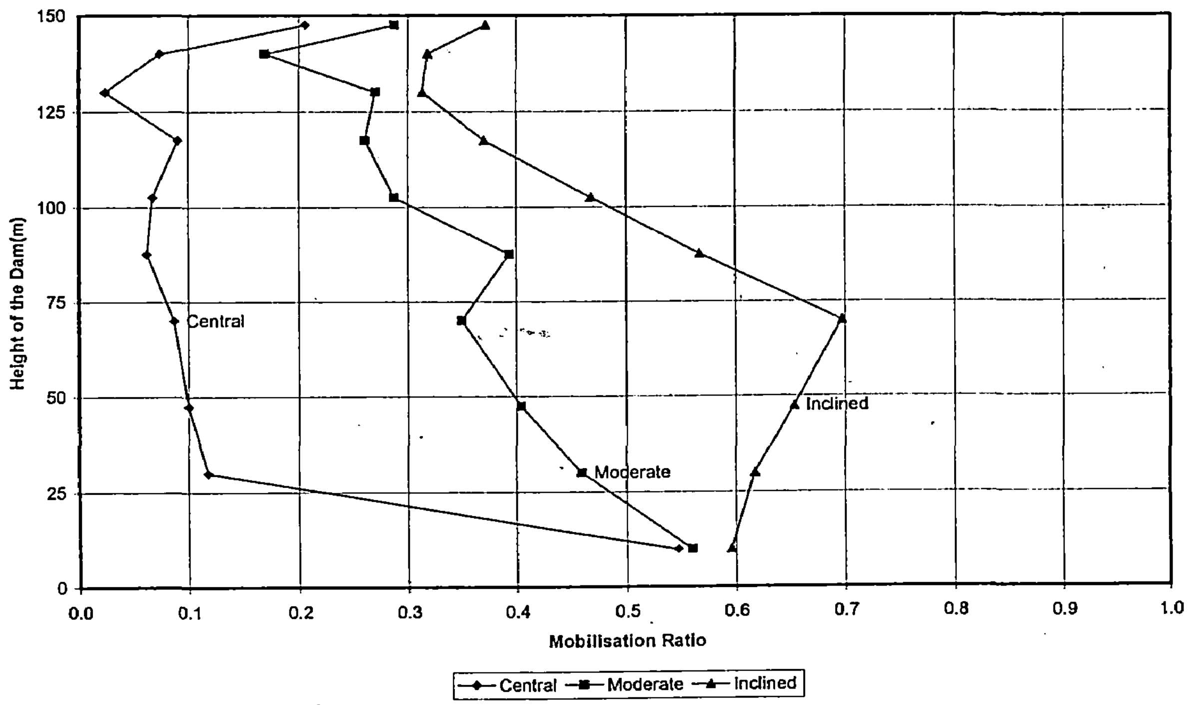


Fig:5.14(b) Mobilisation ratio in core at the full reservoir condition

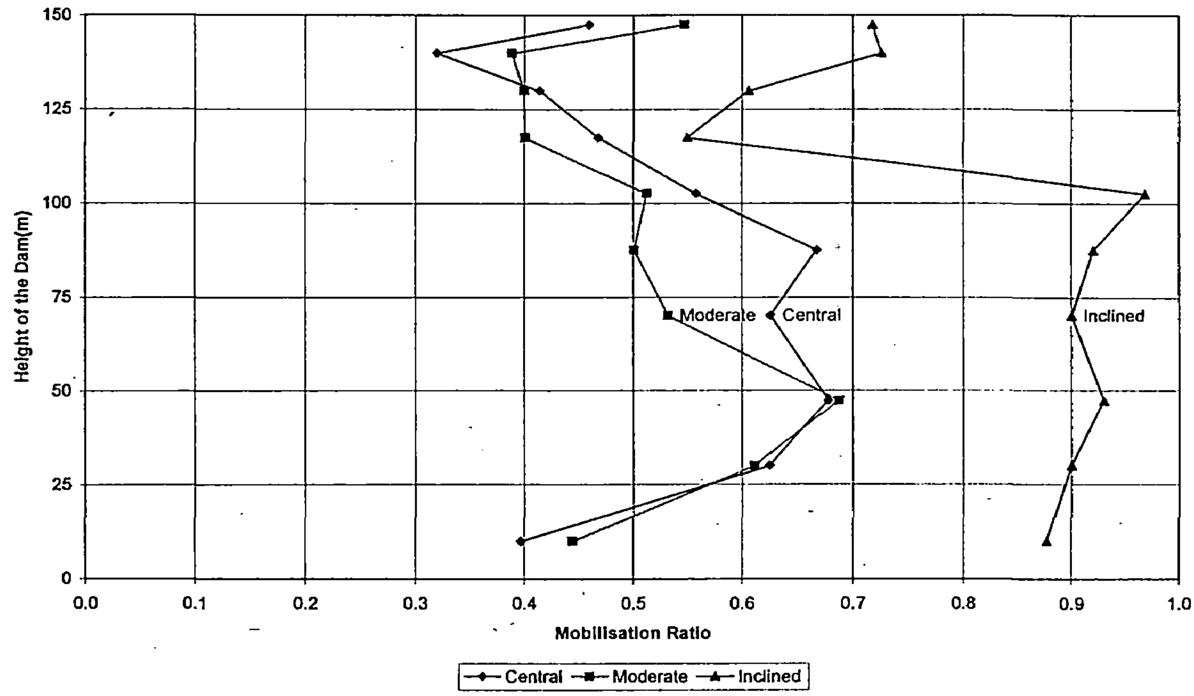


Fig:5.14(c) Mobilisation ratio in u/s shell at the end of construction

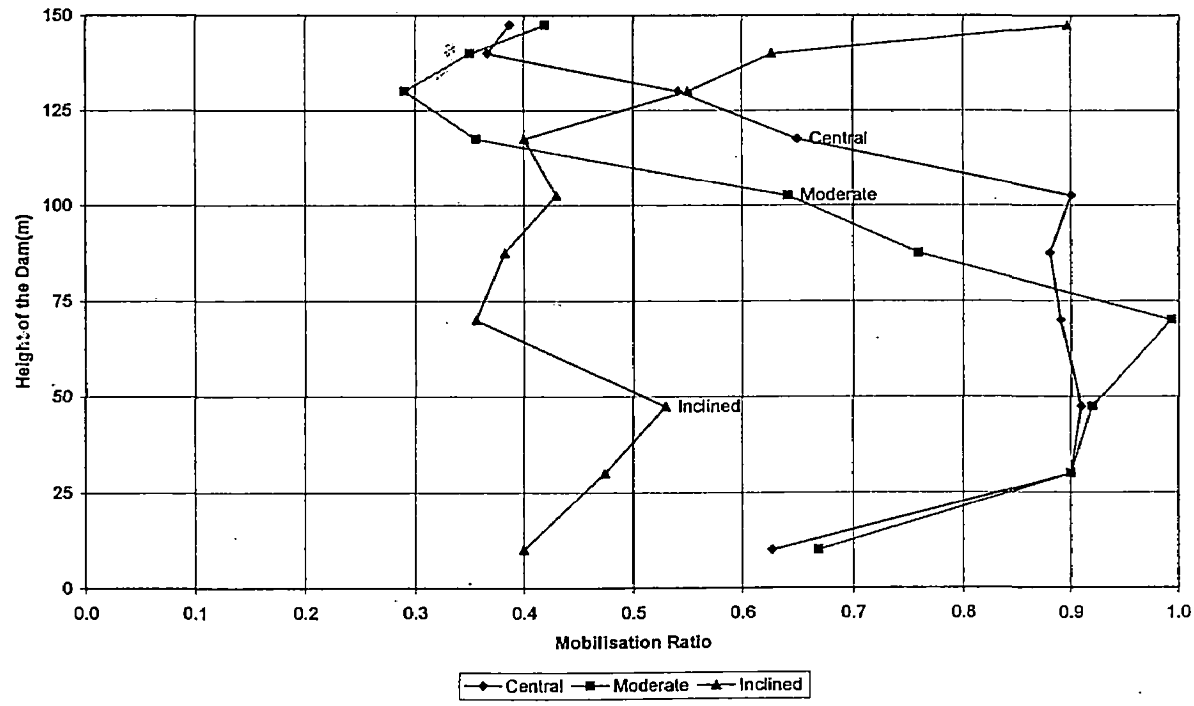


Fig:5.14(d) Mobilisation ratio in u/s shell at full reservoir condition

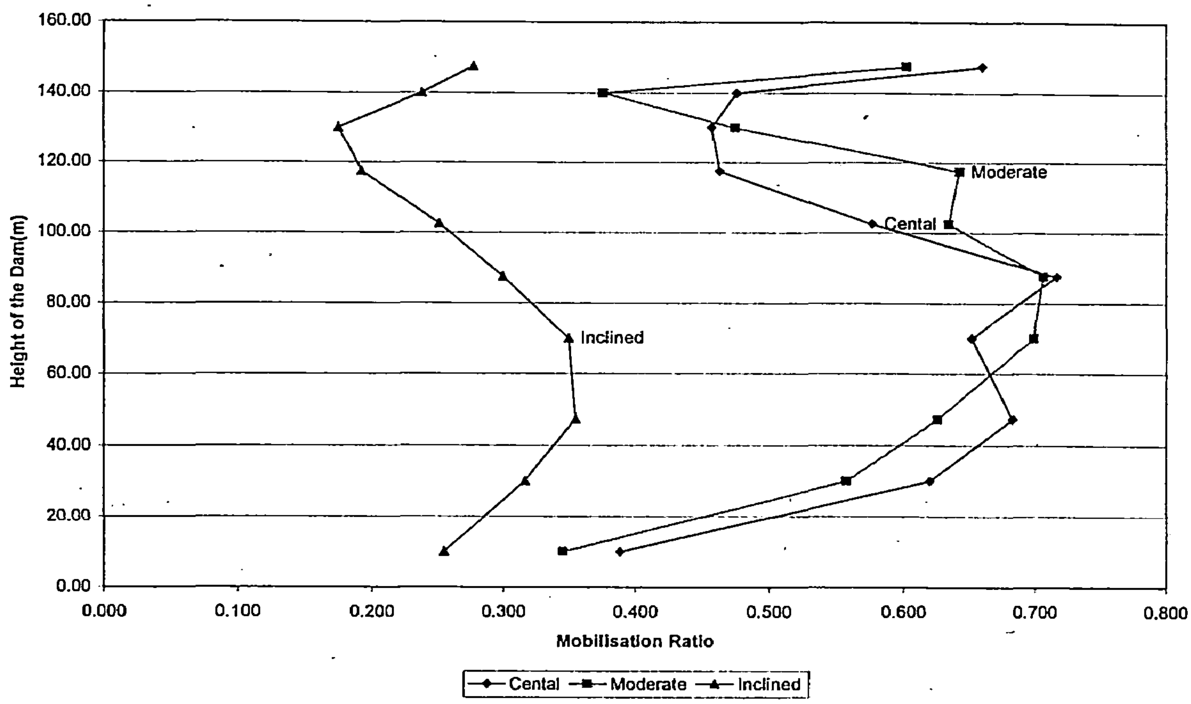


Fig:5.14(e) Mobilisation ratio in d/s shell at the end of construction

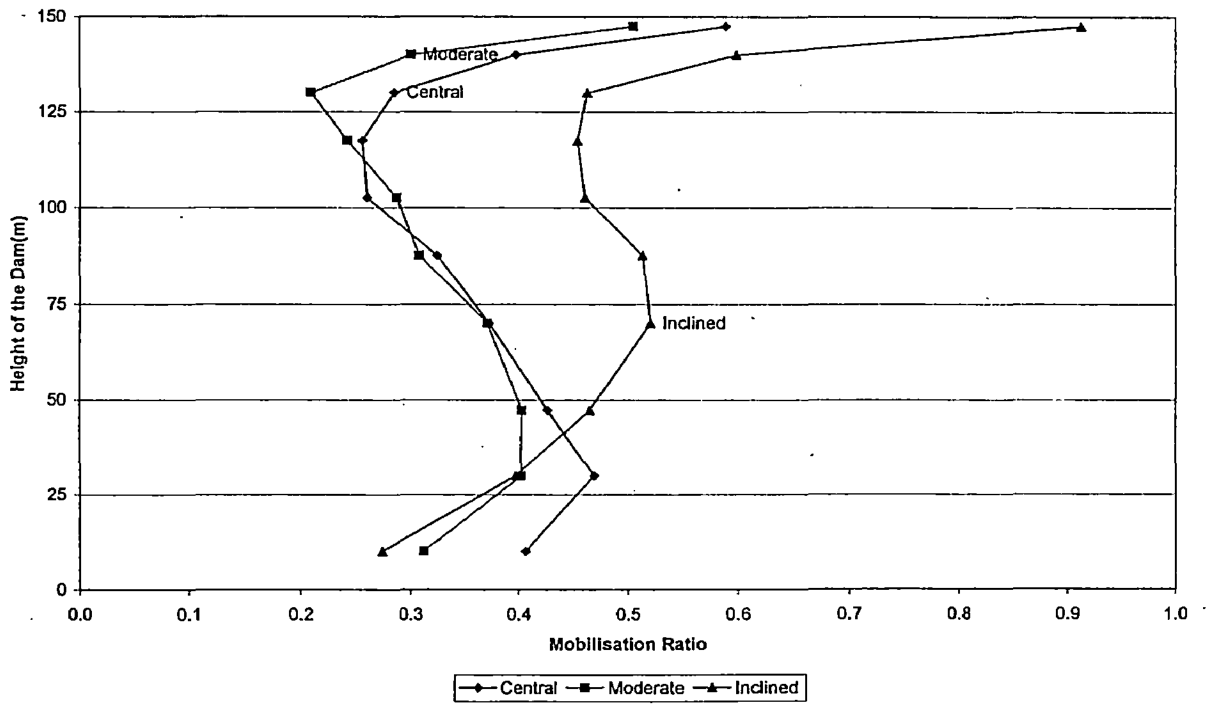
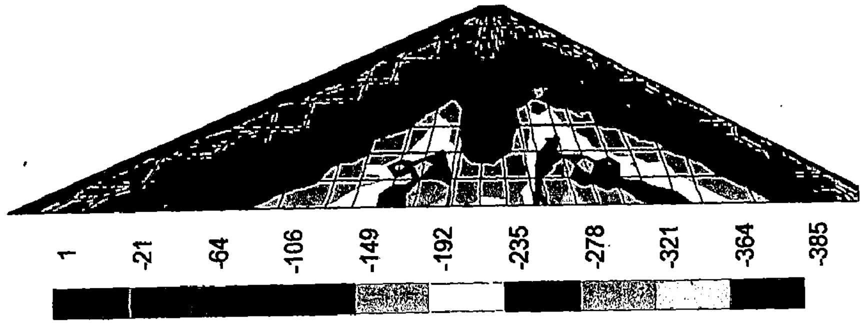
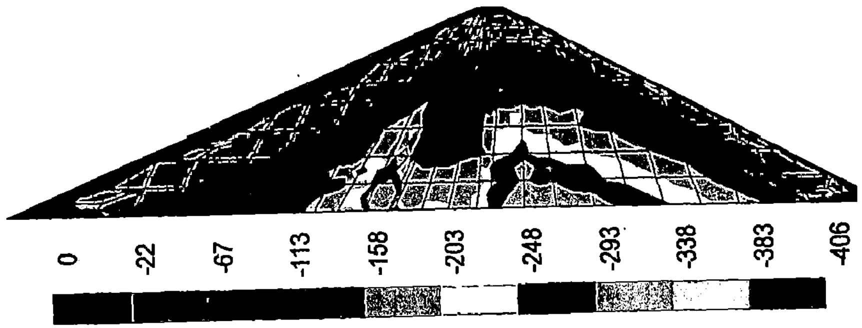


Fig:5.14(f) Mobilisation ratio in d/s shell at the full reservoir condition

Central Core Dam





Moderately inclined Core Dam

Inclined Core Dam

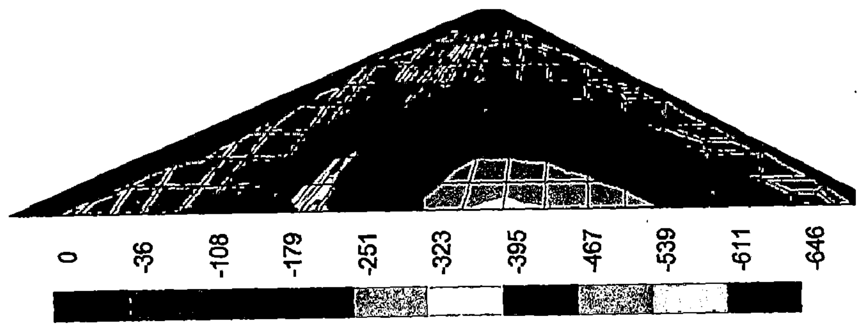
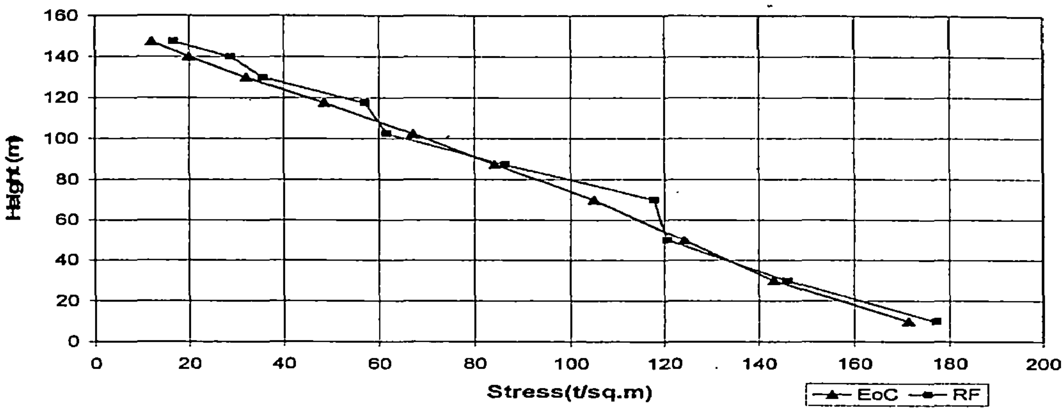
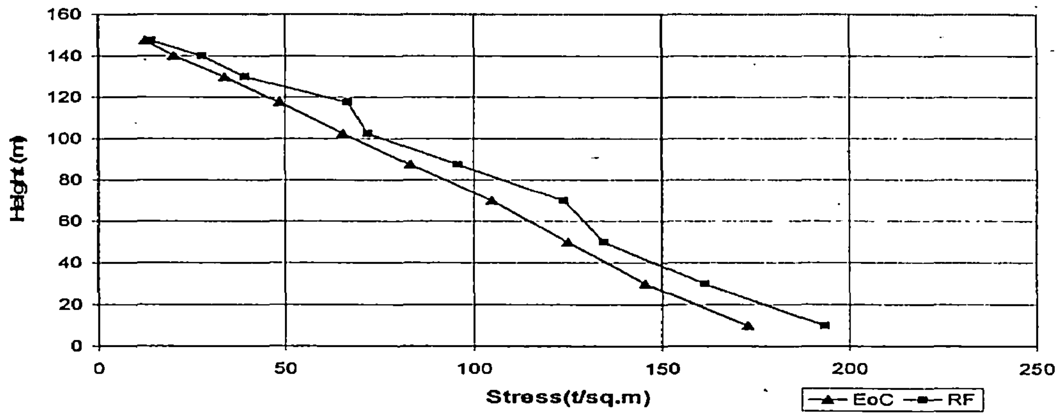


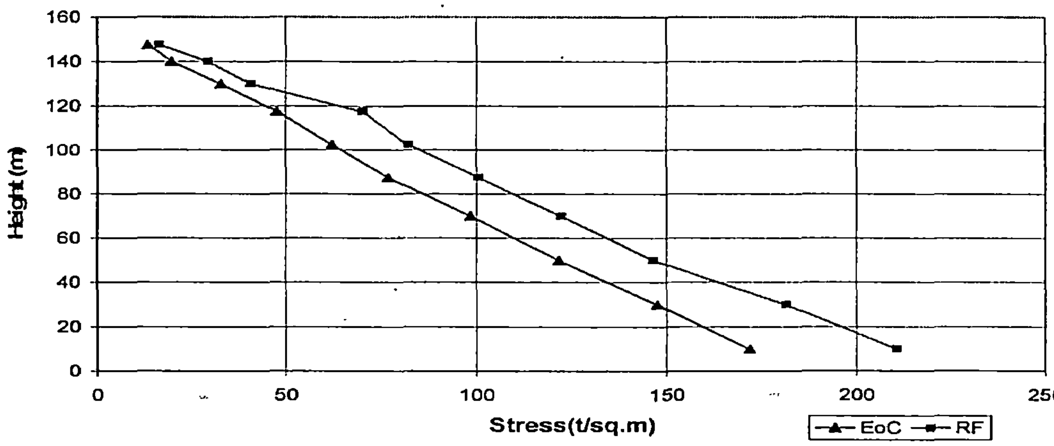
Fig 5.15 Major principal stress contours at Full Reservoir Stage



Central Core dam

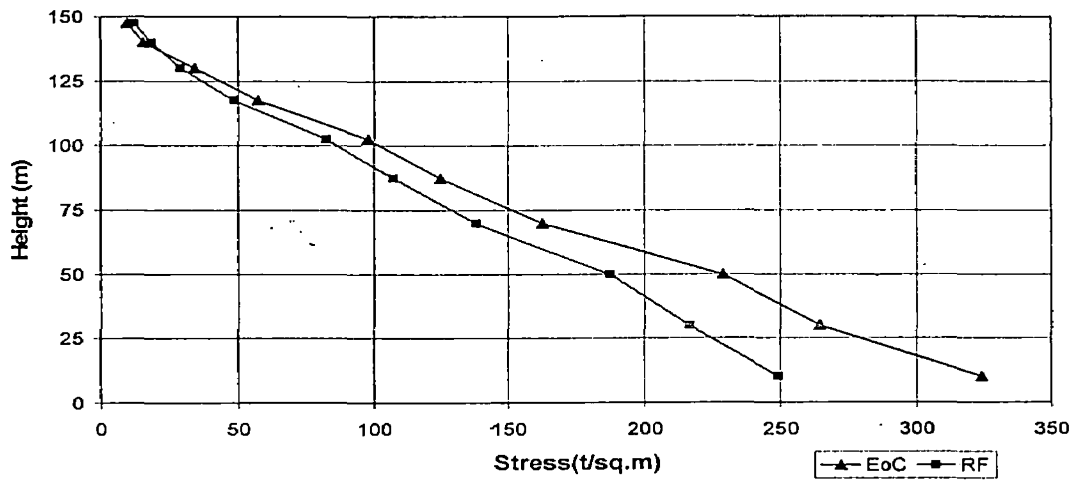


Moderately inclined core dam

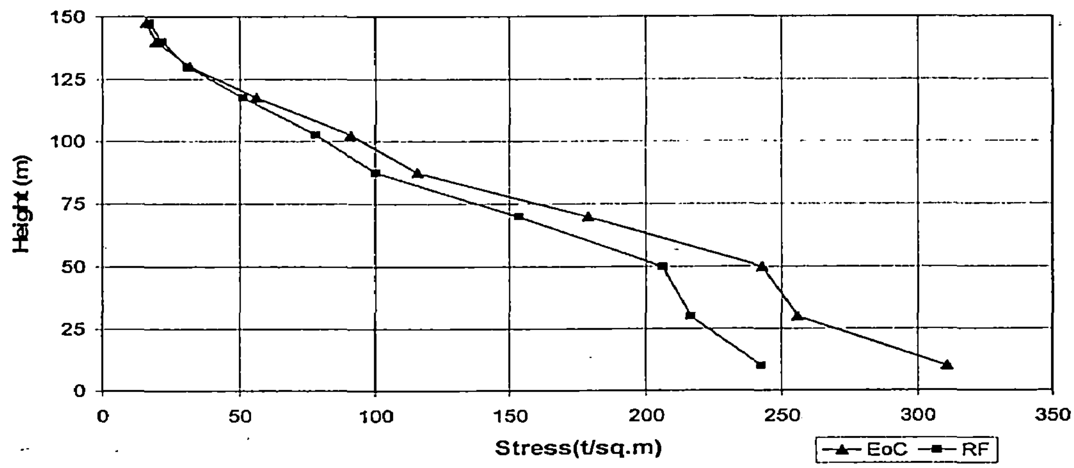


Inclined core dam

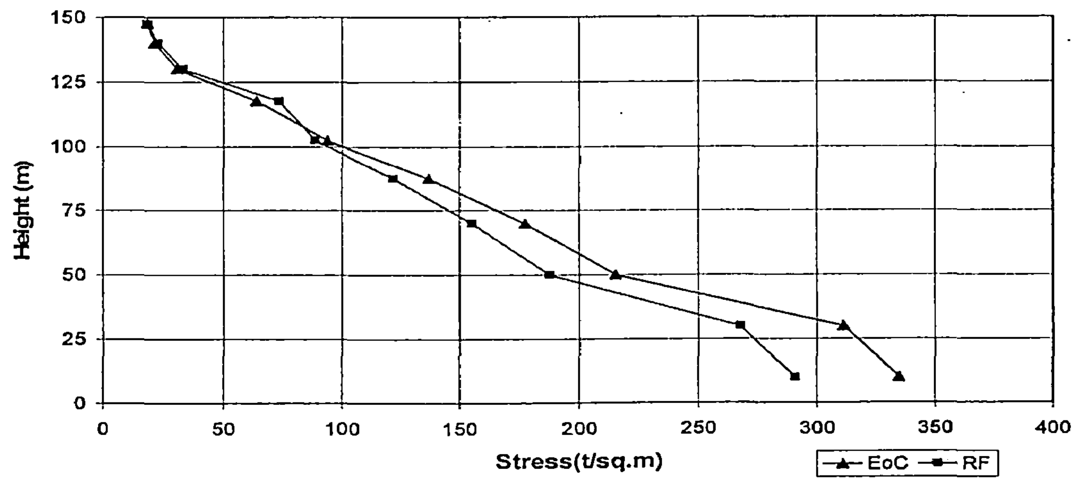
Fig:5.16(a) Variation of major principal (compressive) stress from end of construction to reservoir full stage (core portion)



Central Core

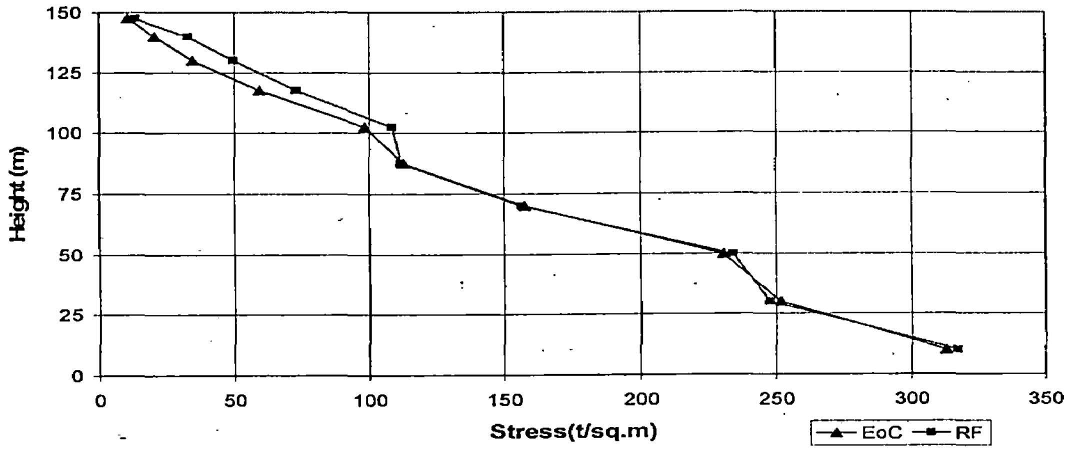


Moderately inclined core

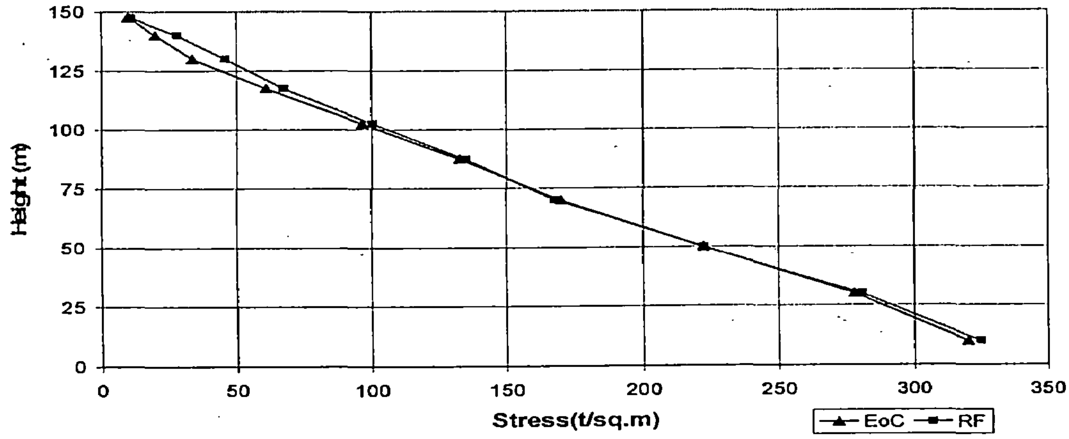


Inclined core

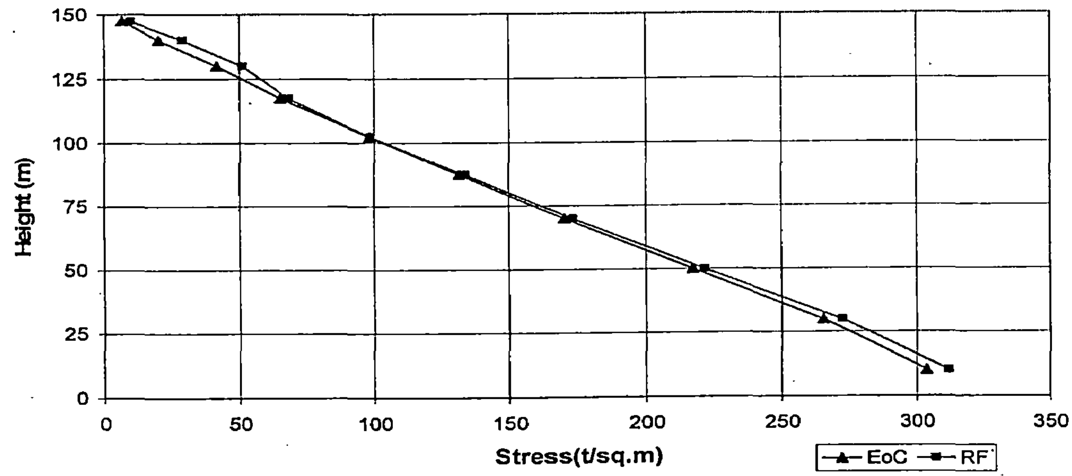
Fig:5.16 (b) Variation of major principal (compressive) stress from end of construction to reservoir full stage (upstream shell)



Central Core

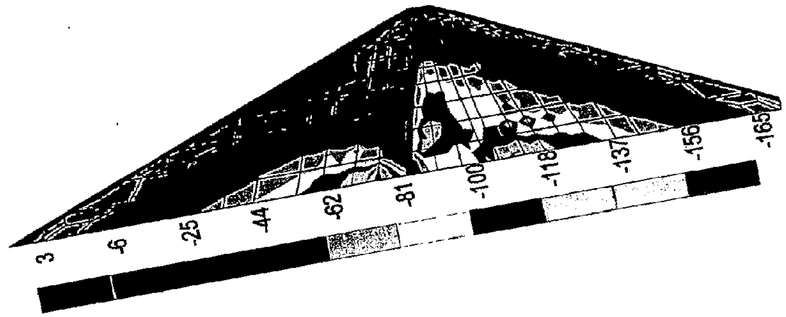


Moderately inclined

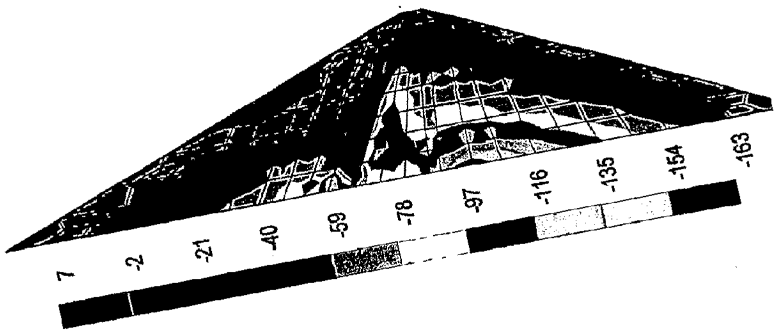


Inclined core

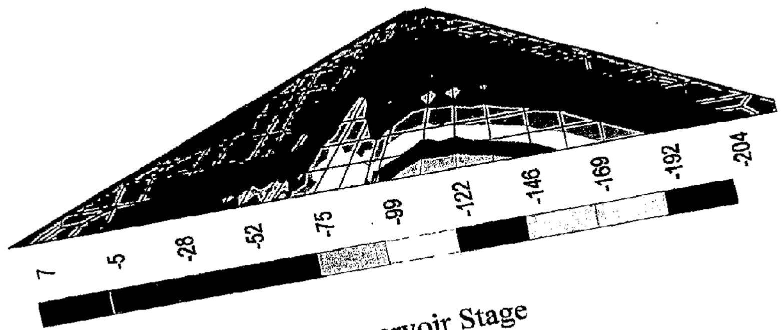
Fig:5.16 (c) Variation of major principal (compressive) stress from end of construction to reservoir full stage (downstream shell)



Central Core Dam

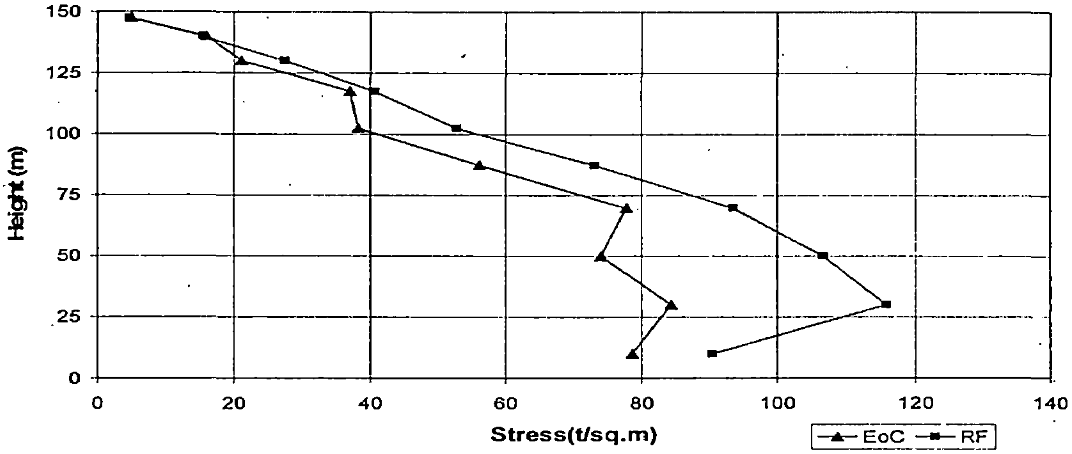


Inclined Core Dam

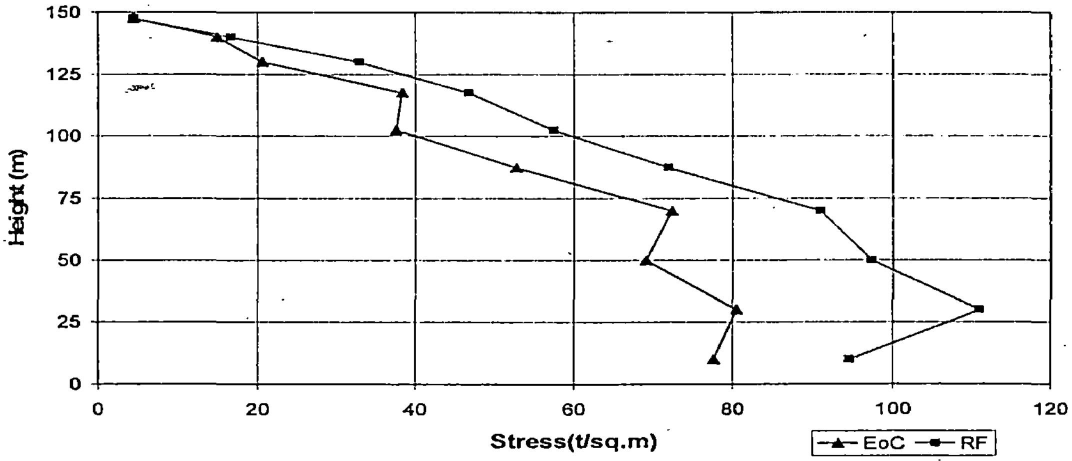


Moderately inclined Core Dam

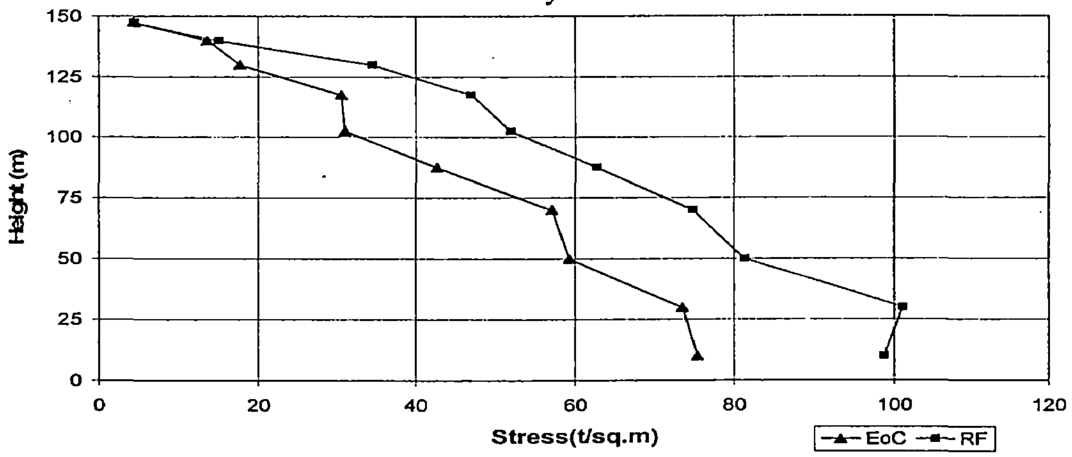
Fig 5.17 Minor principal stress contours at Full Reservoir Stage



Central core

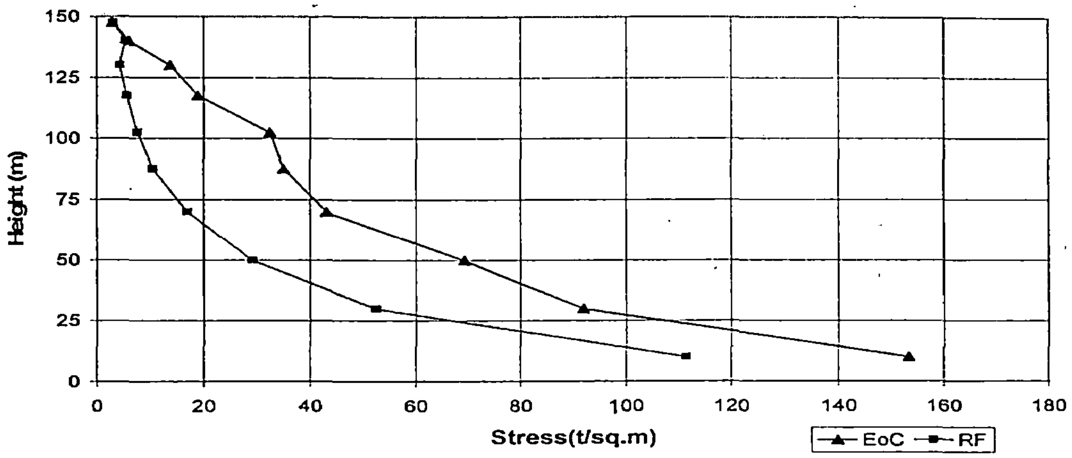


Moderately inclined

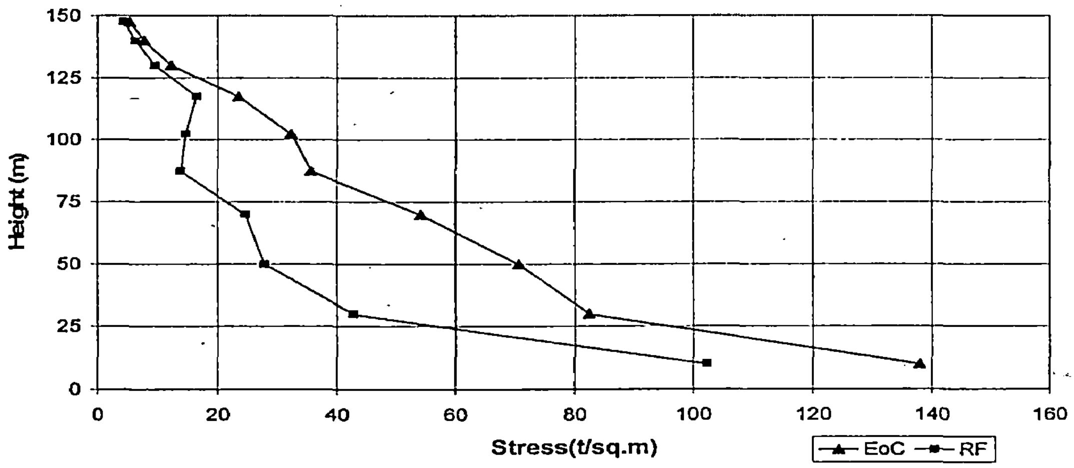


Inclined core

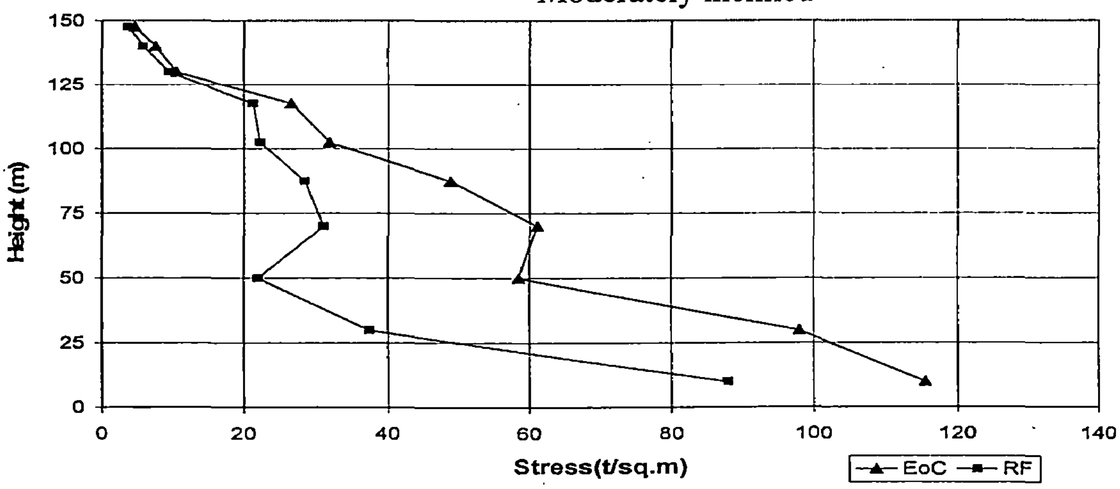
Fig:5.18 (a) Variation of minor principal (compressive) stress from end of construction to reservoir full stage (core portion)



Central core

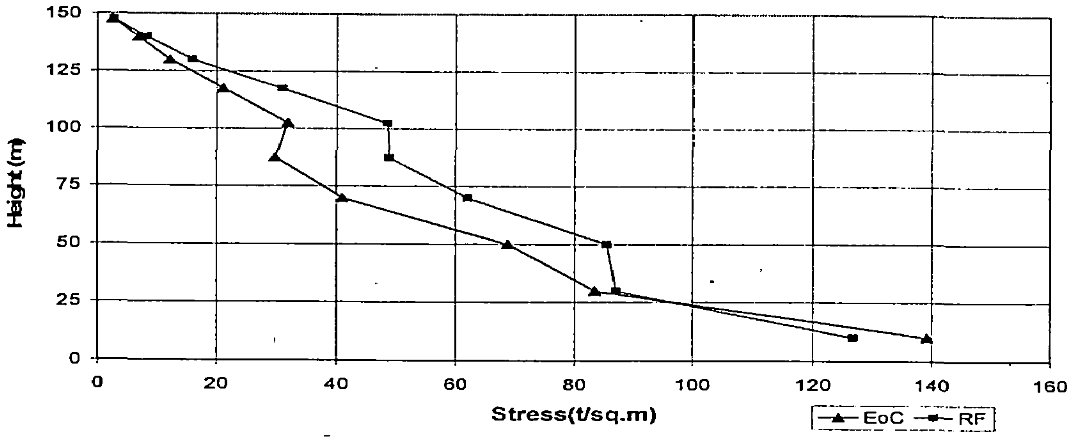


Moderately inclined

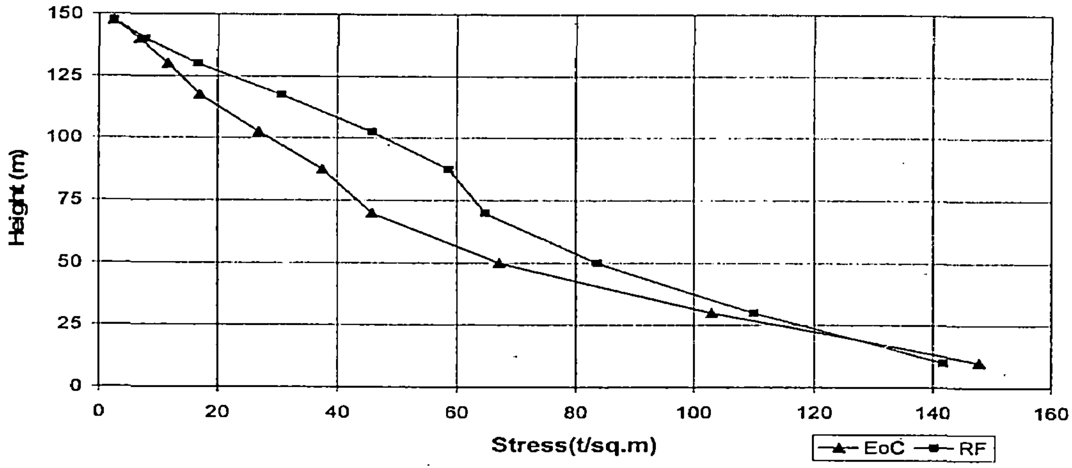


Inclined Core

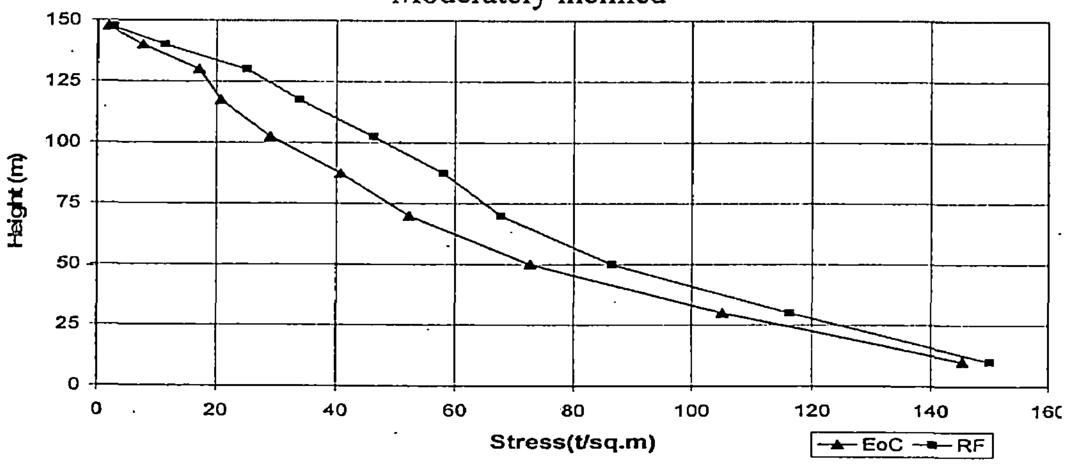
Fig:5.18 (b) Variation of minor principal (compressive) stress from end of construction to reservoir full stage (upstream shell)



Central core



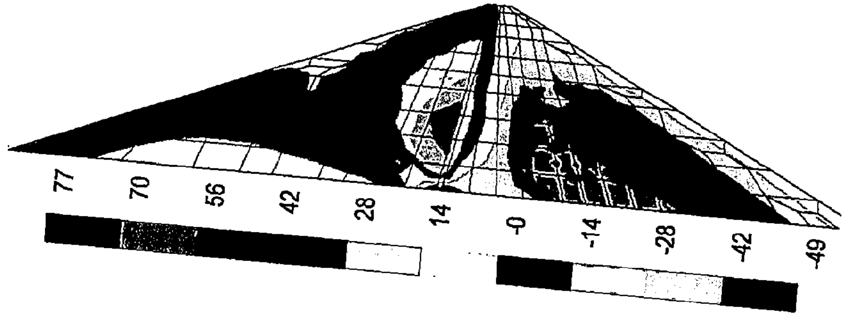
Moderately inclined

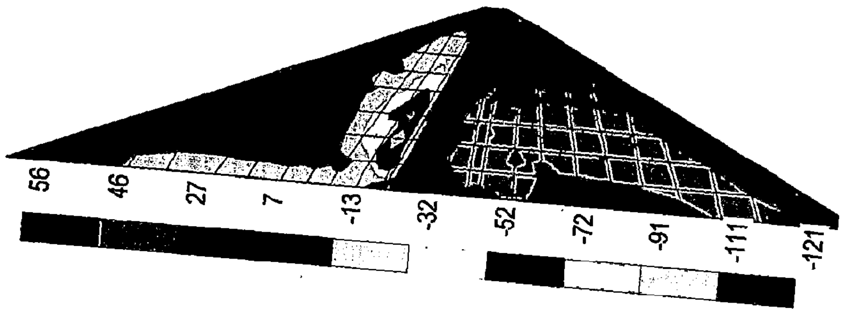


Inclined core

Fig:5.18 (c) Variation of minor principal (compressive) stress from end of construction to reservoir full stage (downstream shell)

Central Core Dam





Moderately inclined Core Dam

Inclined Core Dam

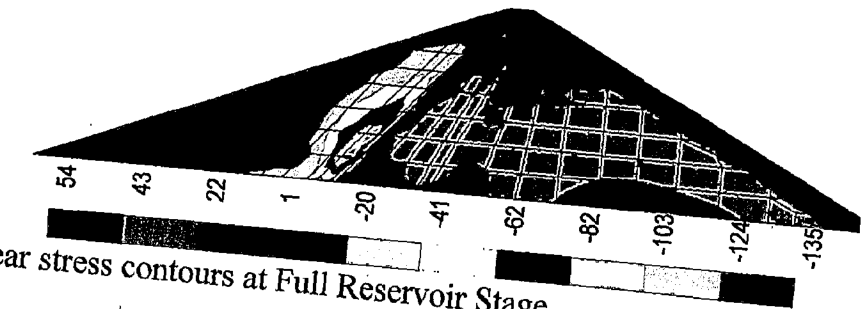
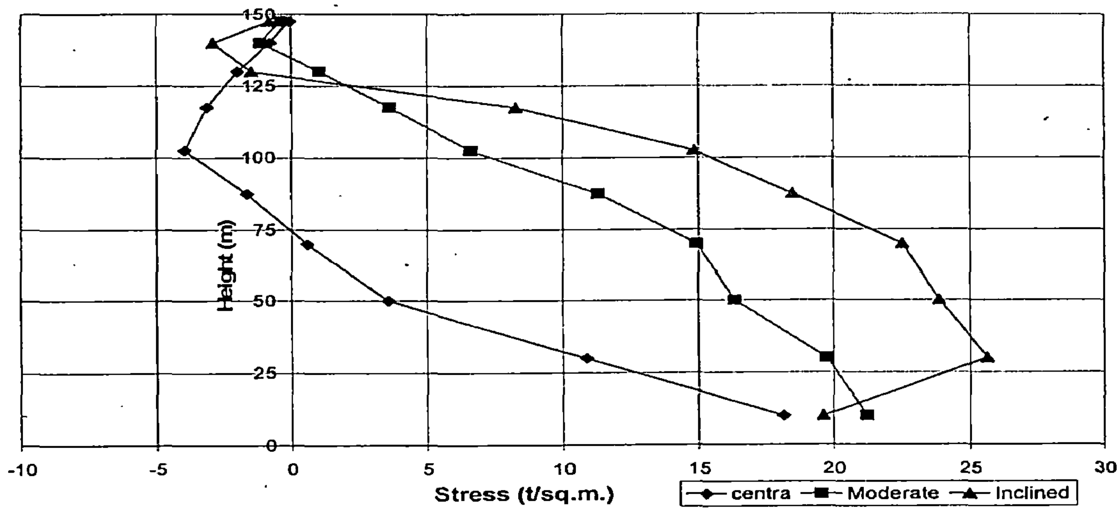
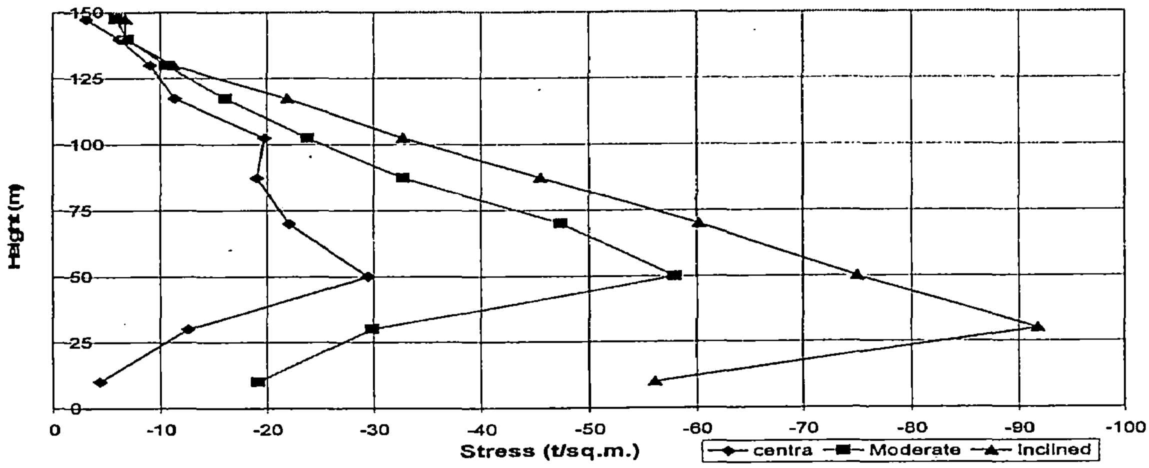


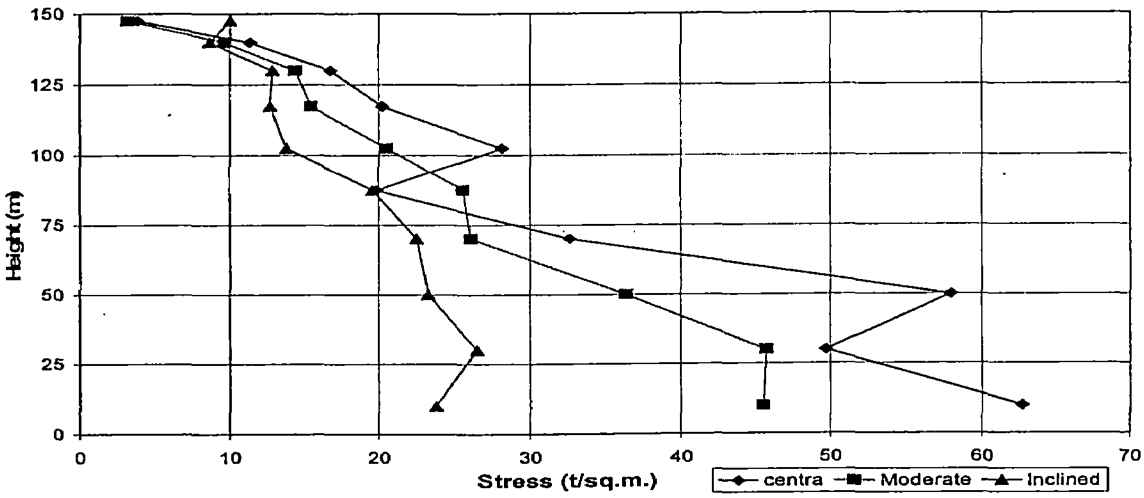
Fig 5.19(a) Shear stress contours at Full Reservoir Stage



Core



Upstream shell



Downstream shell

Fig:5.19(b) Shear stress variation in three sections at reservoir full stage

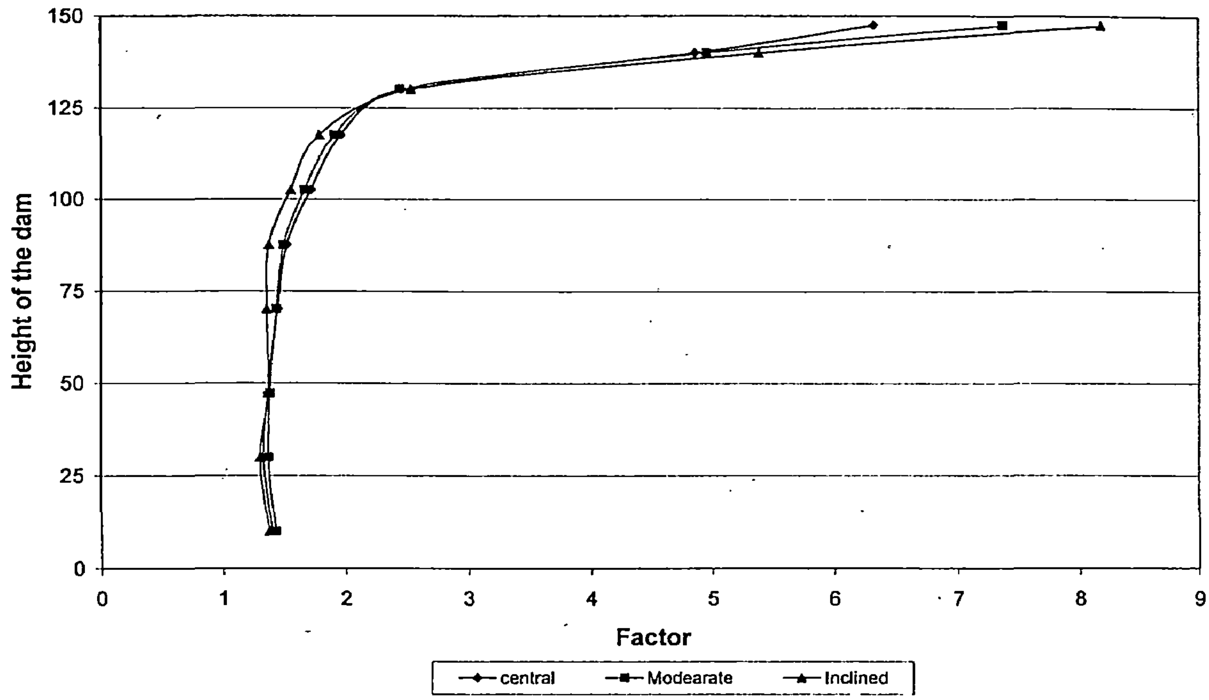


Fig:5.20(a) Variation of hydraulic fracture potential ratio for horizontal crack

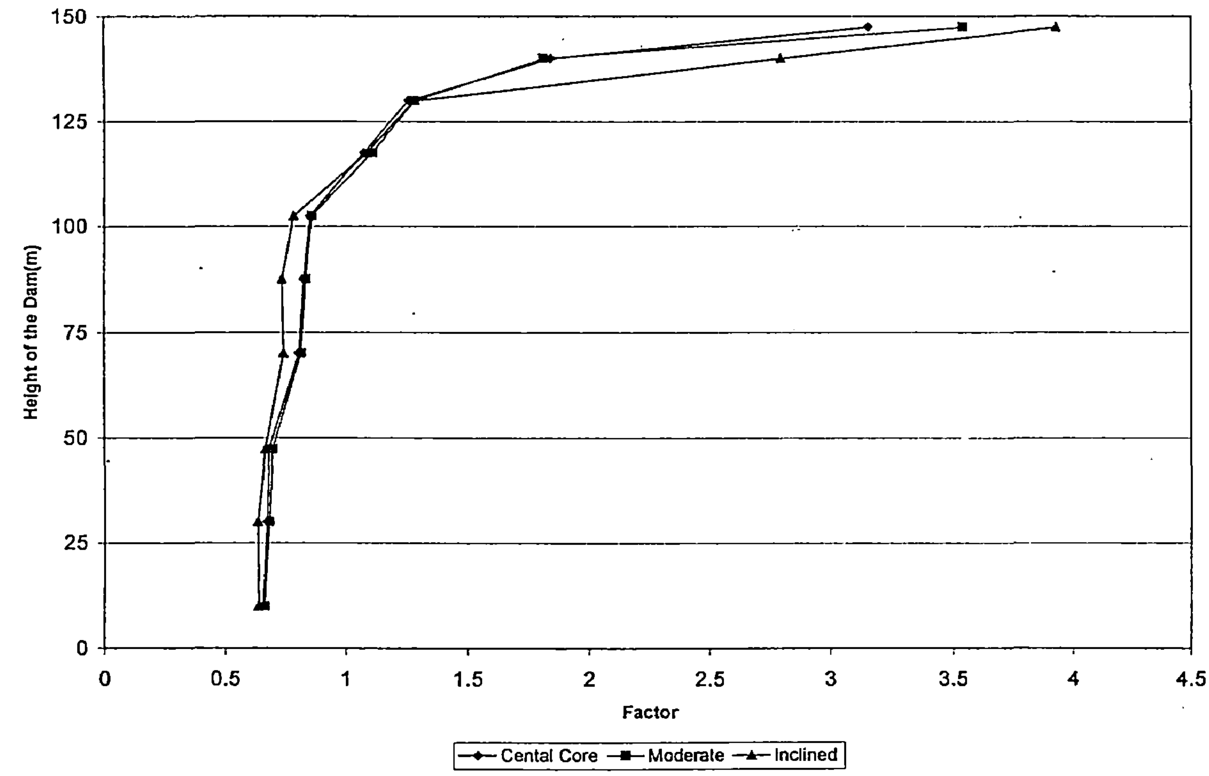
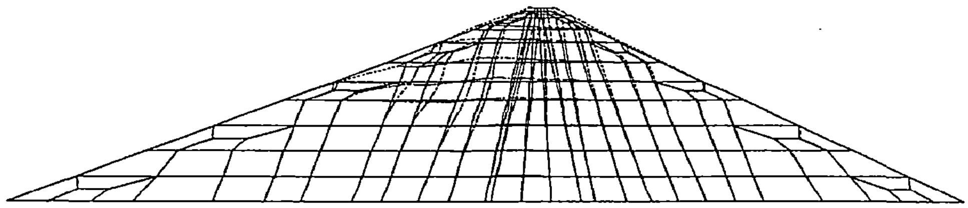
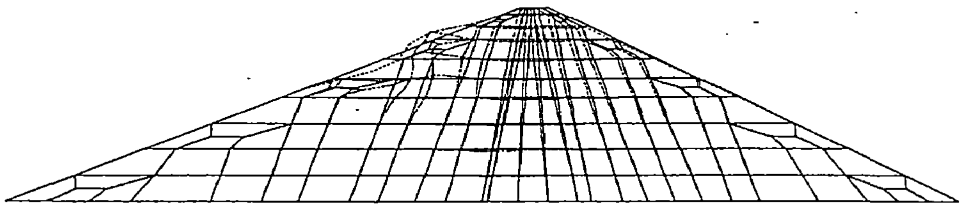


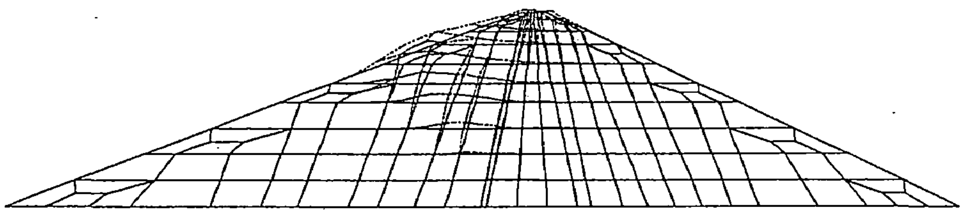
Fig: 5.20(b) Variation of hydraulic fracture potential ratio for vertical crack



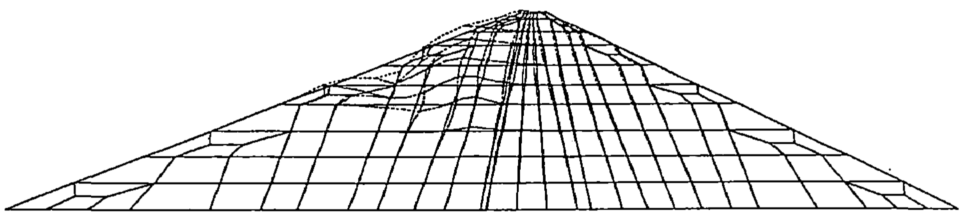
Mode 1 Time period = 1.7 sec



Mode 2 Time period =1.41 sec

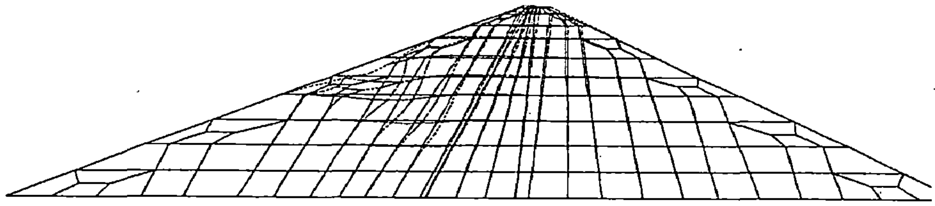


Mode 3 Time period = 1.16 sec

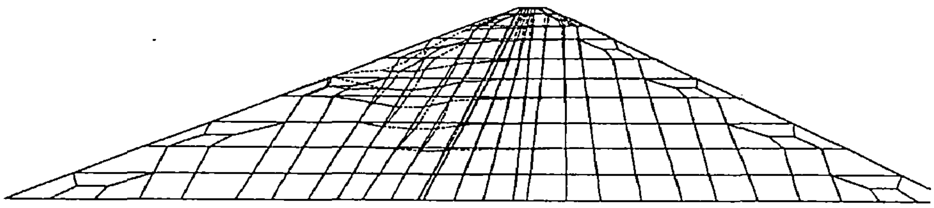


Mode 4 Time period = 1.13 sec

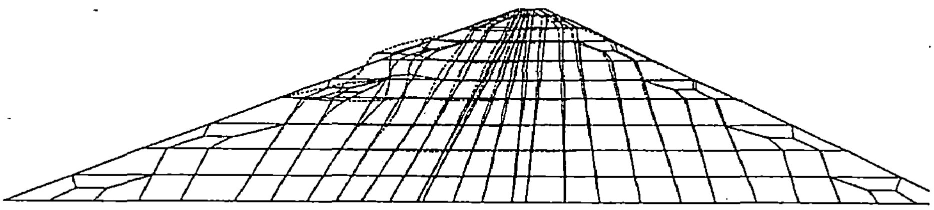
Fig:5.21(a) First four mode shapes of the central core dam



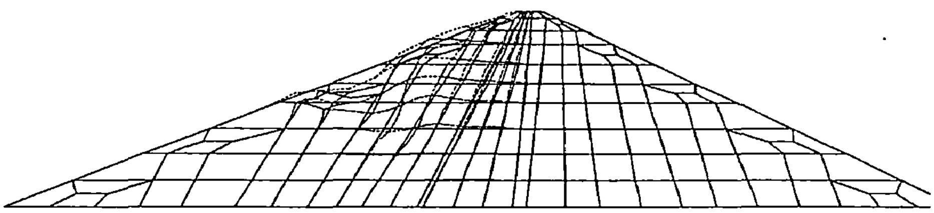
Mode 1 Time period = 1.53 sec



Mode 2 Time period = 1.47 sec

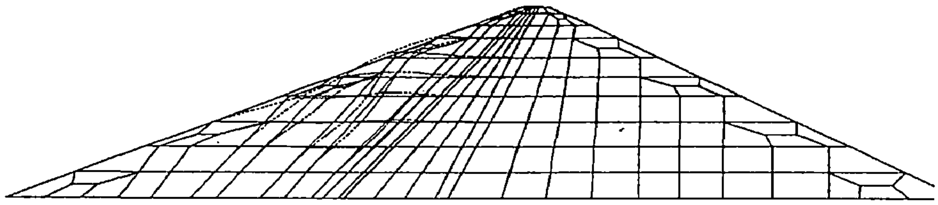


Mode 3 Time period = 1.00 sec

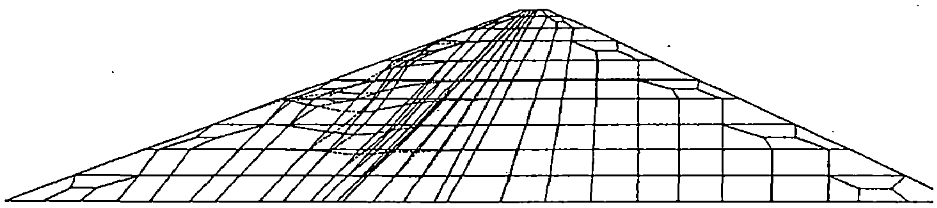


Mode 4 Time period = 0.92 sec

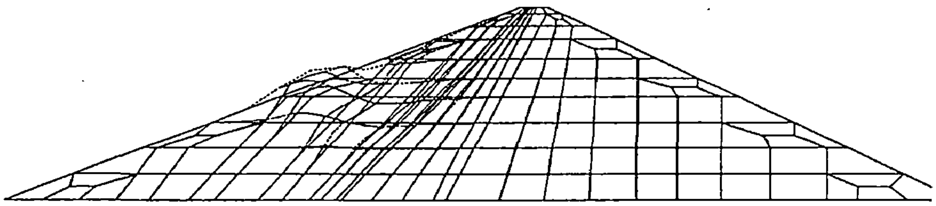
Fig:5.21(b) First four mode shapes of the moderately inclined core dam



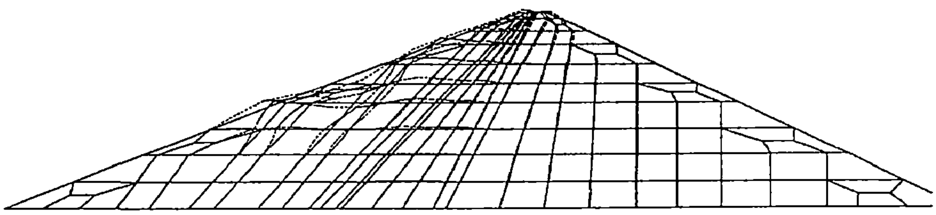
Mode 1 Time period = 1.53 sec



Mode 2 Time period = 1.47 sec

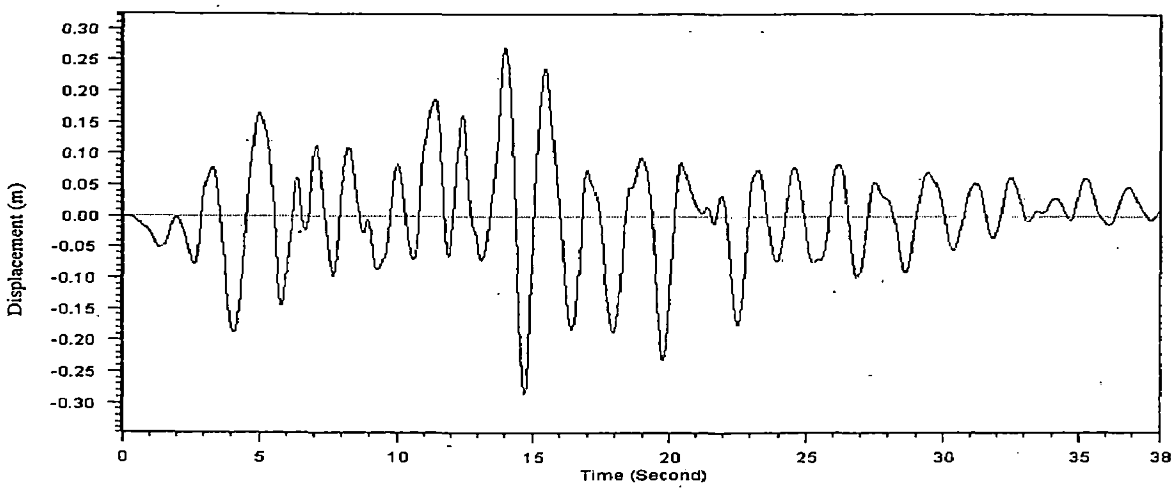


Mode 3 Time period = 1.00 sec

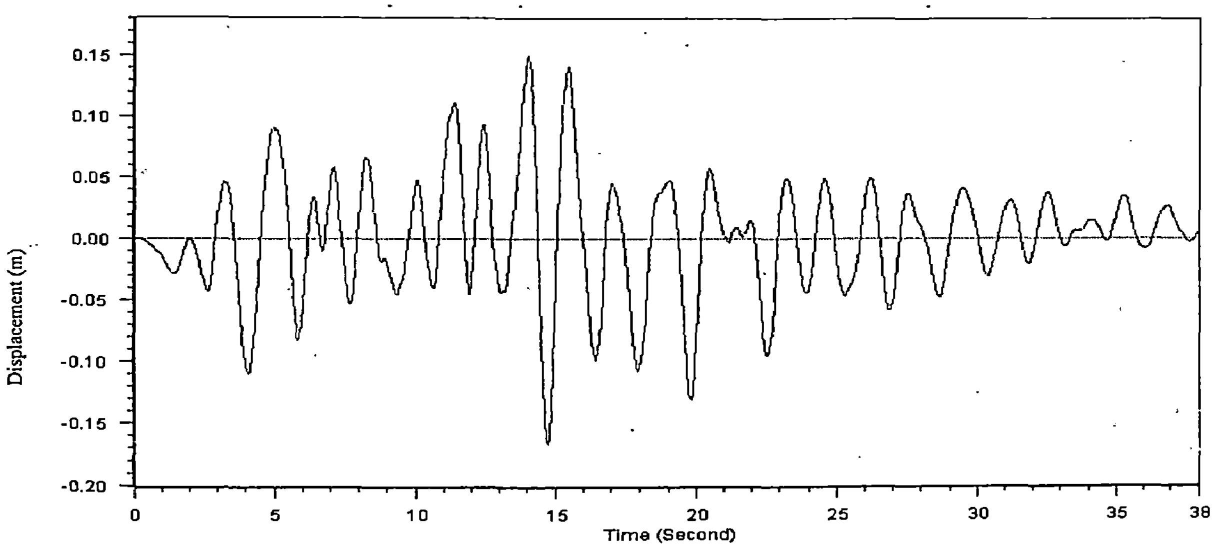


Mode 4 Time period = 0.95 sec

Fig:5.21(c) First four mode shapes of the inclined core dam

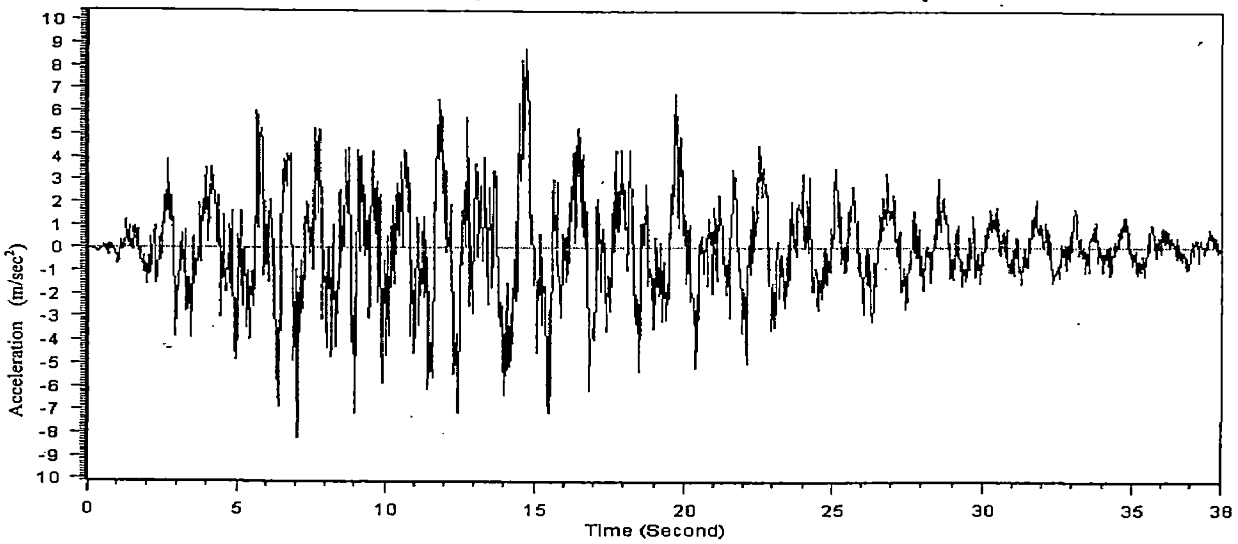


Horizontal displacement
(0.27 m to -0.28m)

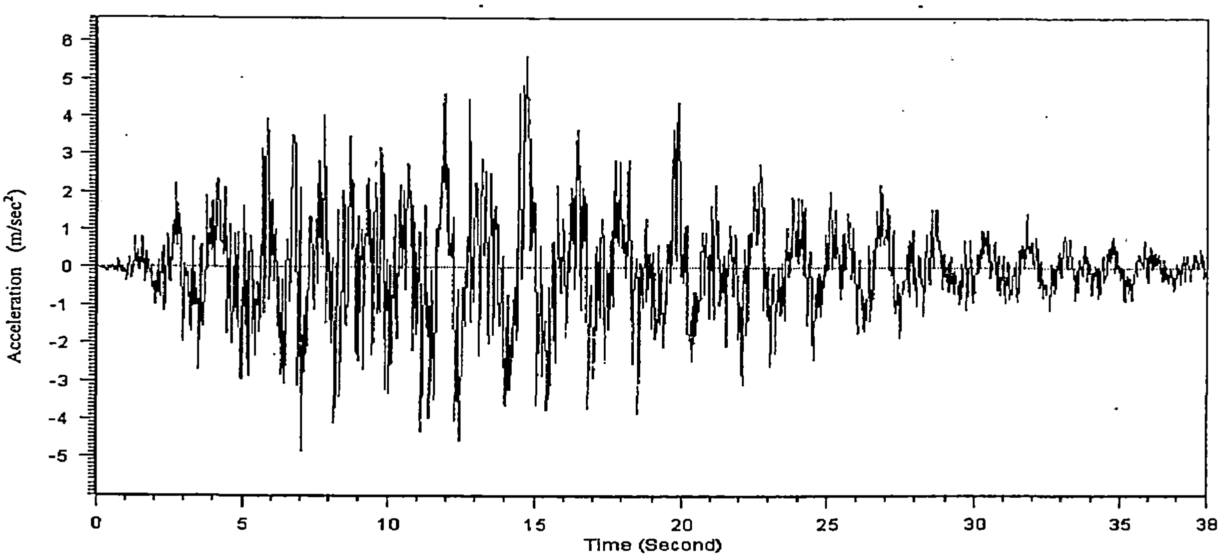


Vertical displacement
(0.15m to -0.166m)

Fig: 5.22(a) Time history of displacement at the crest of the Central core dam

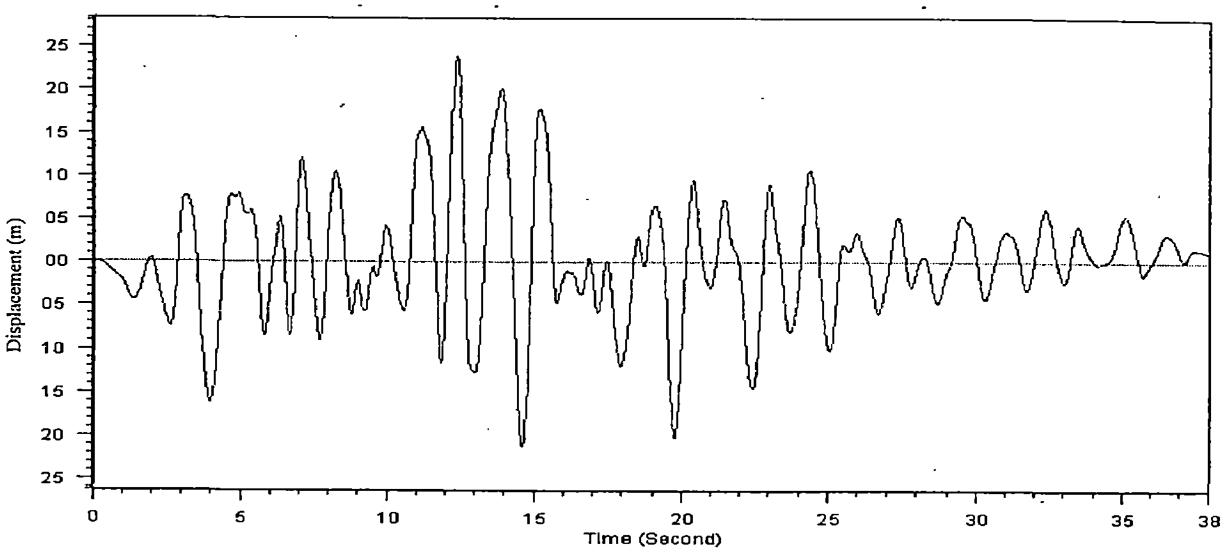


Horizontal acceleration
(8.77 m/sec^2 to -8.19m/sec^2)

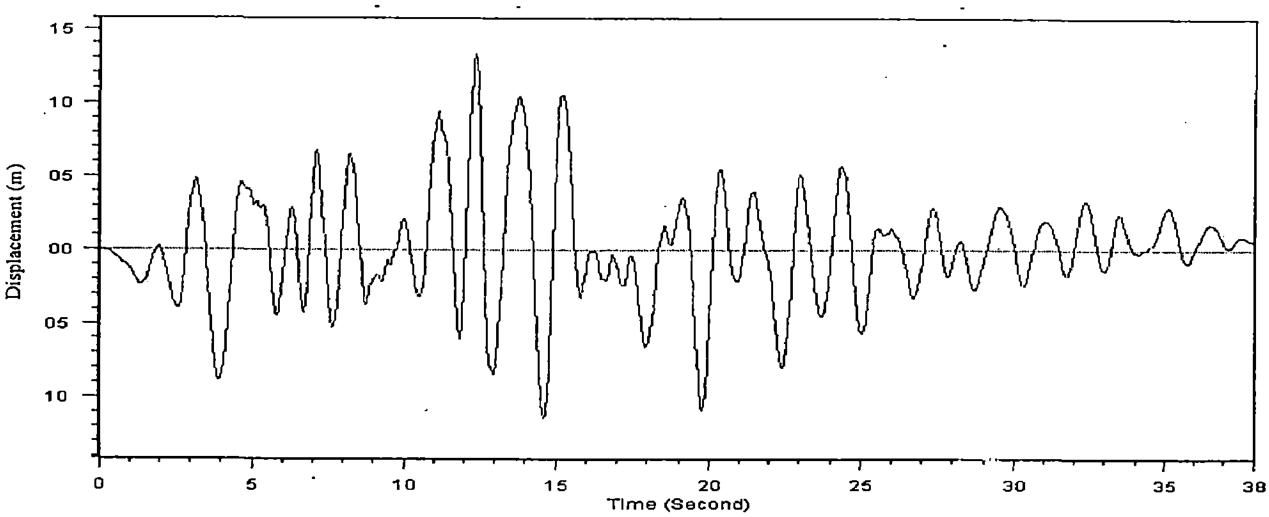


Vertical acceleration
(5.6 m/sec^{2}to -4.85 m/sec^2)

Fig:5.22(b) Time history of acceleration at the crest of the central core dam

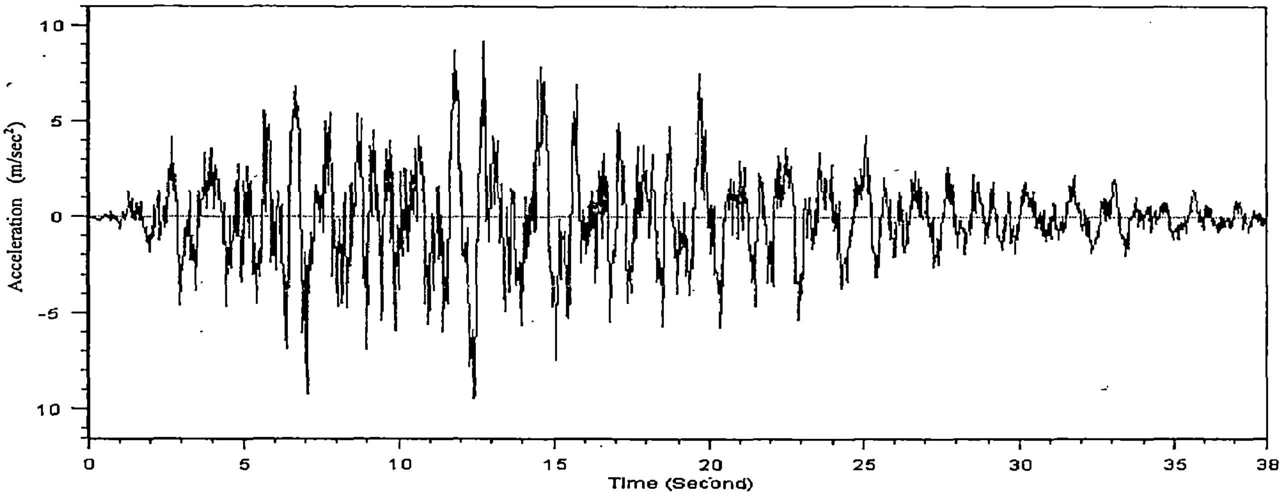


Horizontal displacement
(0.24m to -0.21m)



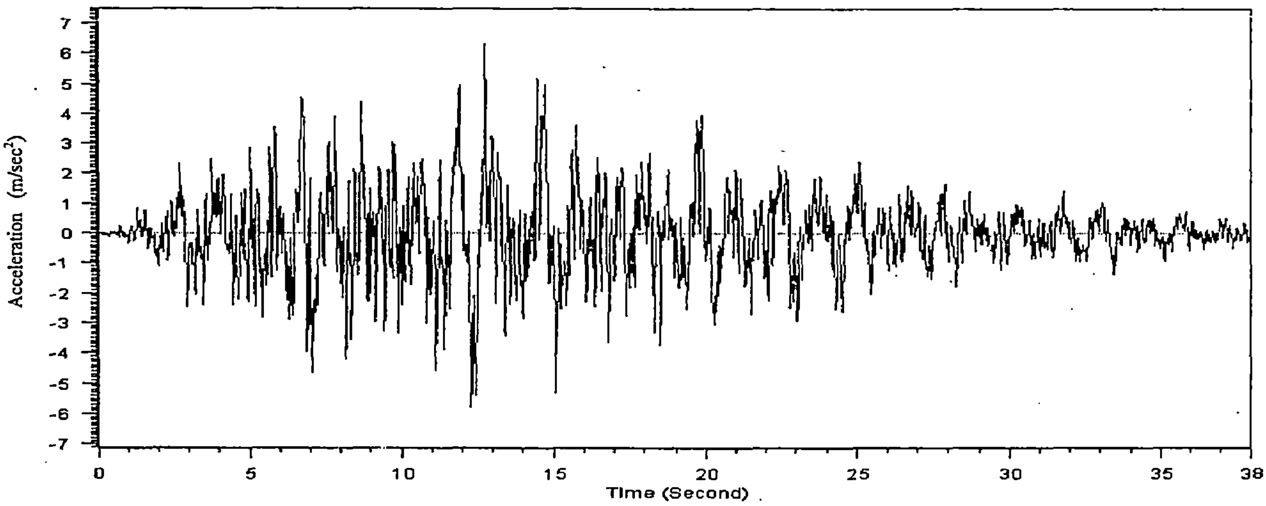
Vertical displacement
(0.13m to -0.11m)

Fig: 5.22(c) Time history of displacement at the crest of the moderately inclined core dam



Horizontal acceleration

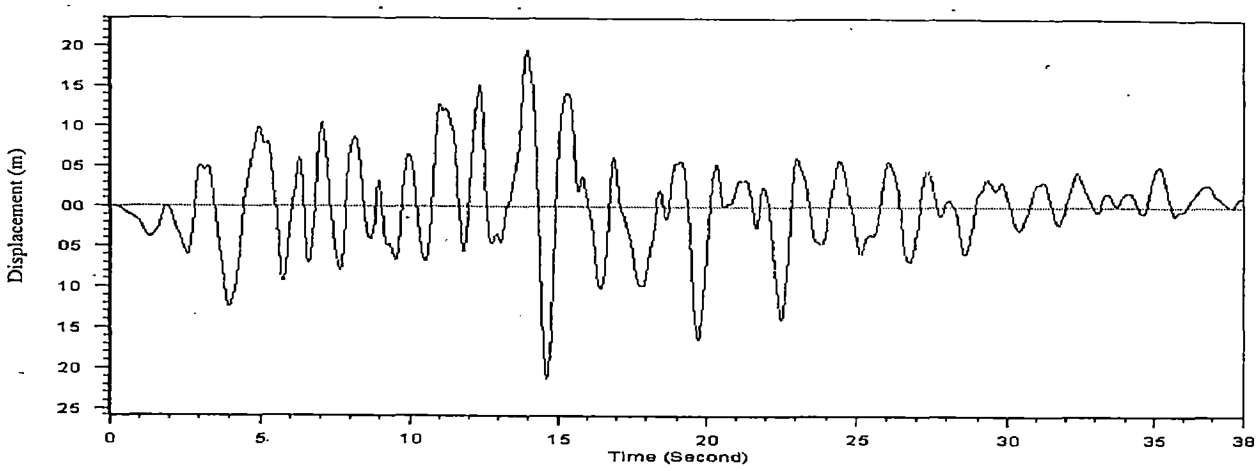
(9.17 m/sec^2 to -9.46m/sec^2)



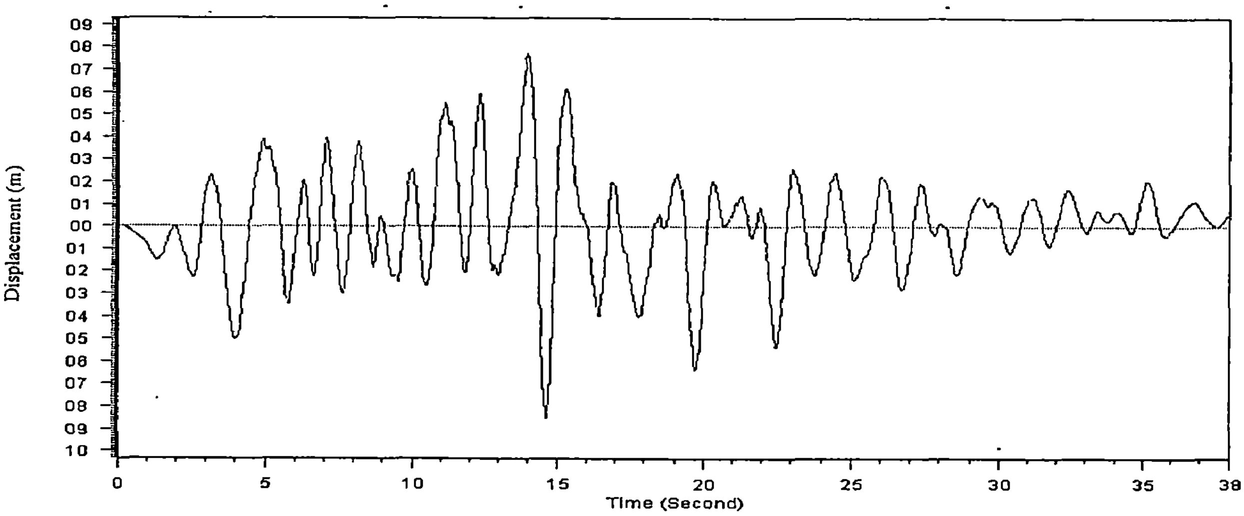
Vertical acceleration

(6.31m/sec^2 to -5.76m/sec^2)

Fig: 5.22 (d) Time history of acceleration at the crest of the moderately inclined core dam



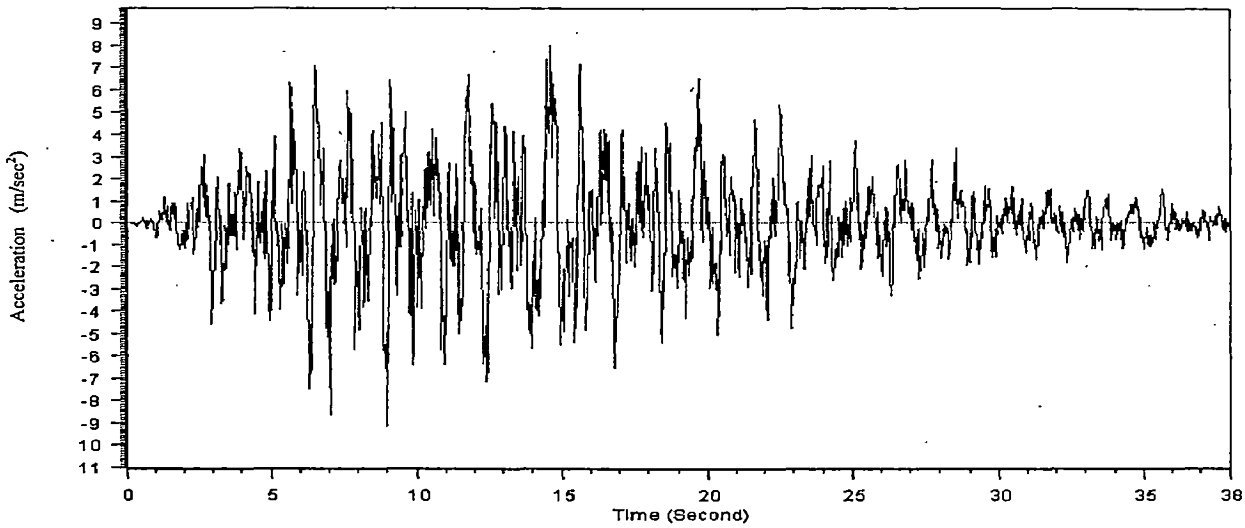
Horizontal displacement
(0.20 m to -0.21m)



Vertical displacement

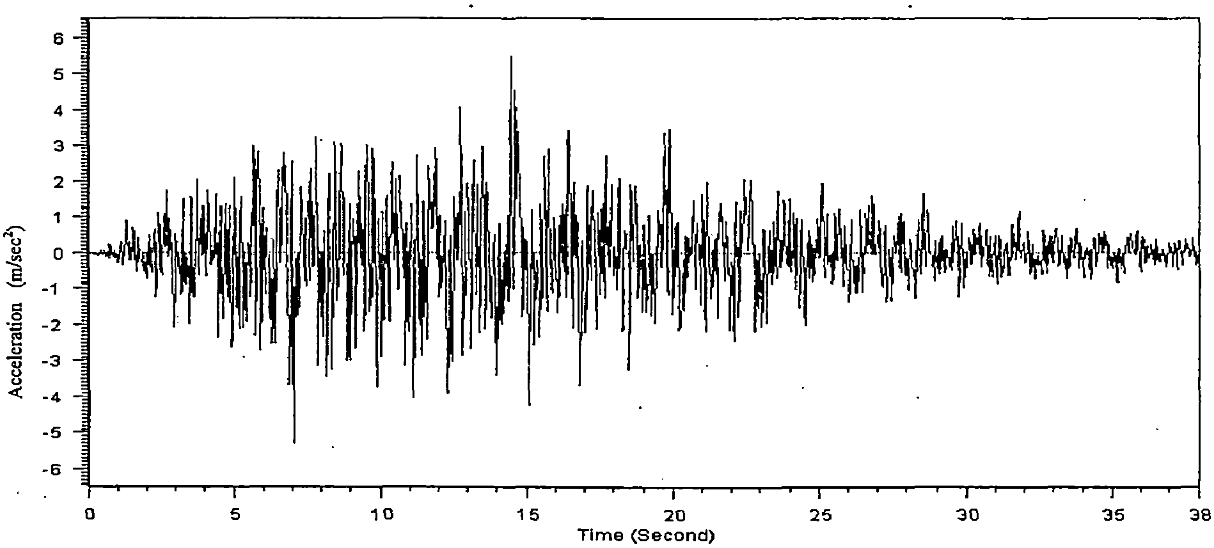
(0.07 m to -0.085m)

Fig: 5.22(e) Time history of displacement at the crest of the inclined core dam



Horizontal acceleration

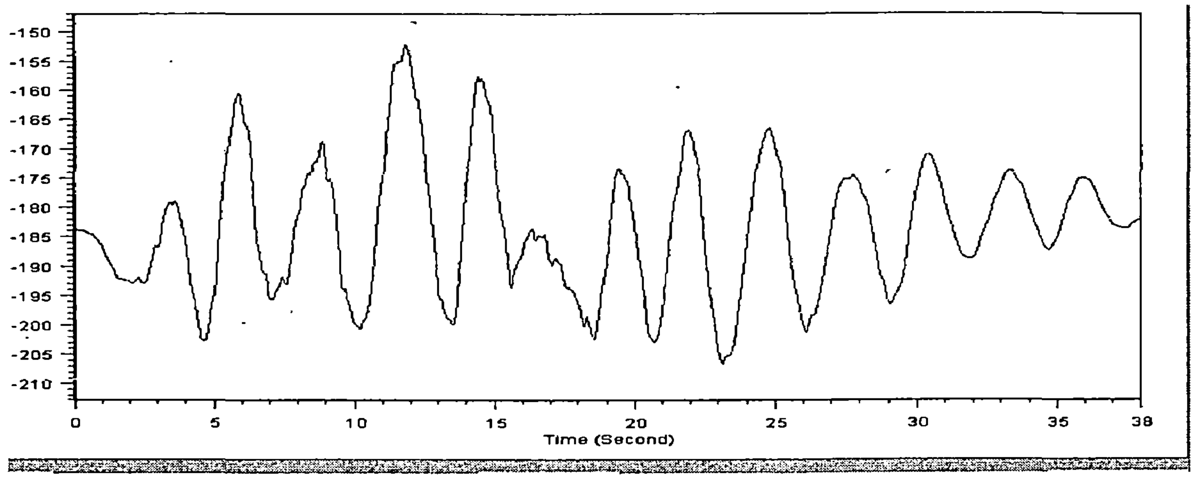
(7.8 m/sec^2 to -9.14 m/sec^2)



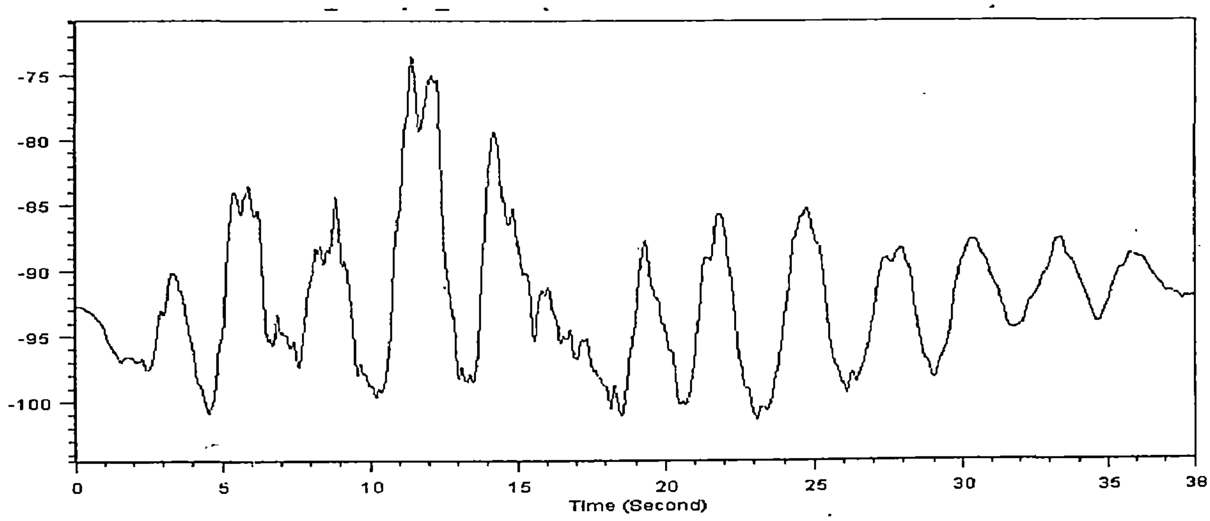
Vertical acceleration

(5.48 m/sec^2 to -5.27 m/sec^2)

Fig: 5.22(f) Time history of acceleration at the crest of the inclined core dam

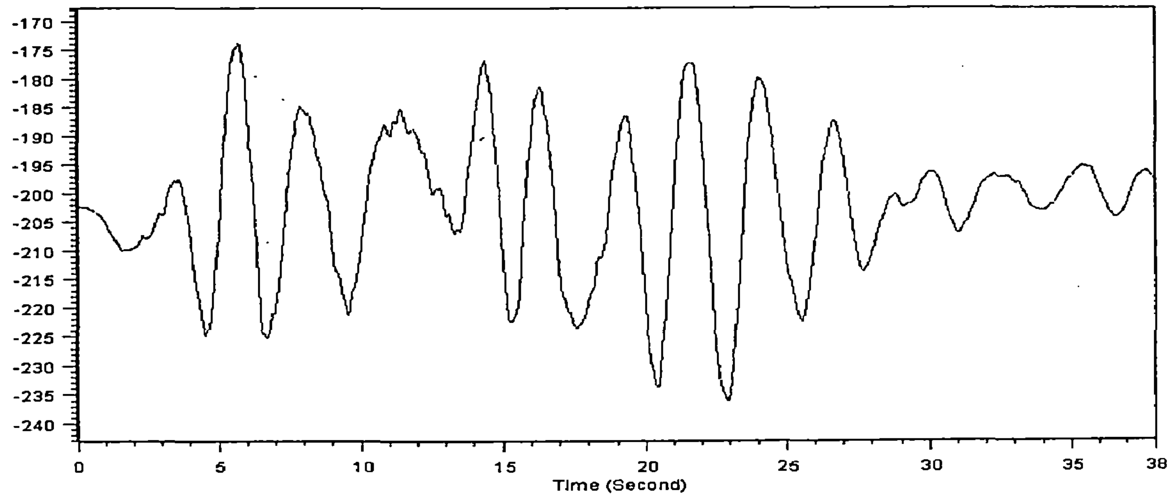


Major Principal Stress
(-152.3 to -206.6 t/m^2)

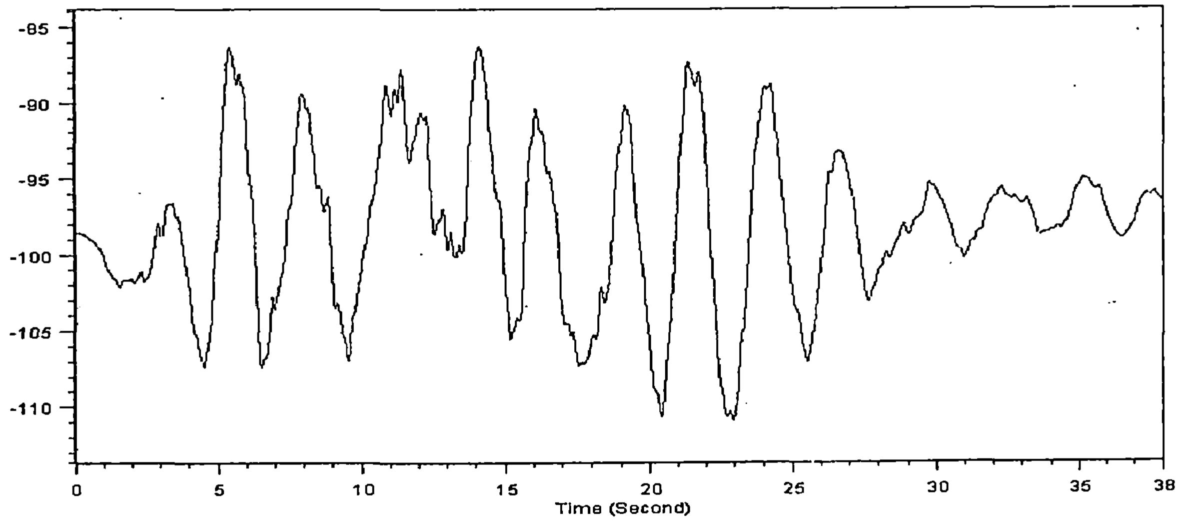


Minor Principal Stress
(-73.53 to -101.4 t/m^2)

Fig: 5.23(a) Time histories of combined principal stress at the base of the core in Central Core Section

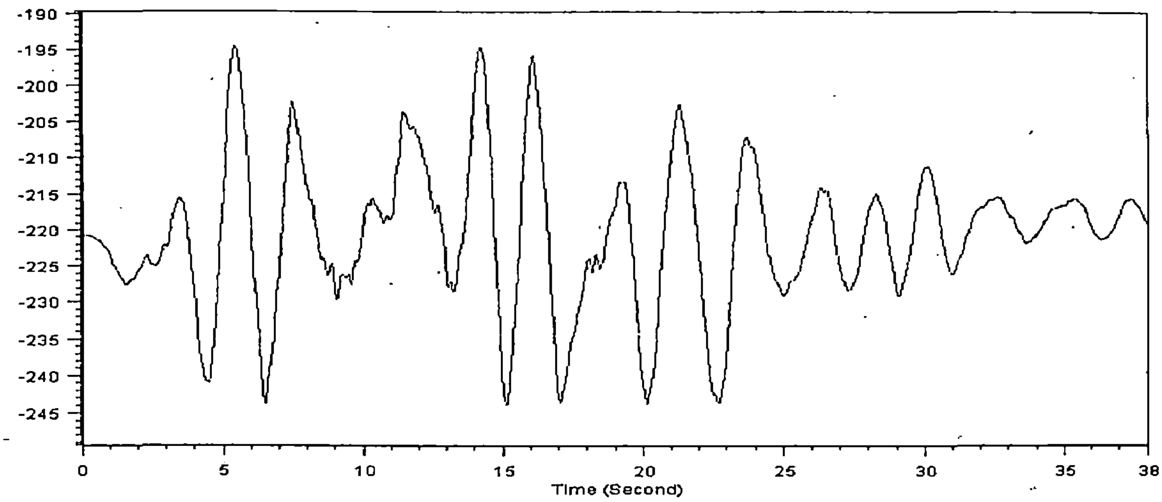


Major Principal Stress
(-173.8 to -236 t/m²)

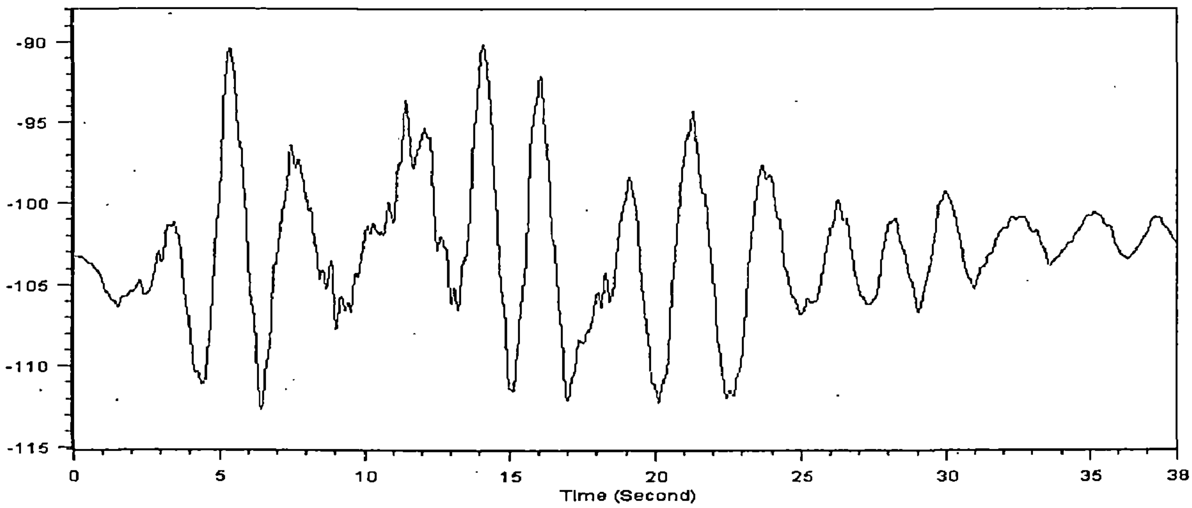


Minor Principal Stress
(-86.24 to -110.9t/m²)

Fig: 5.23(b) Time histories of combined principal stress at the base of the core in Moderately inclined Section

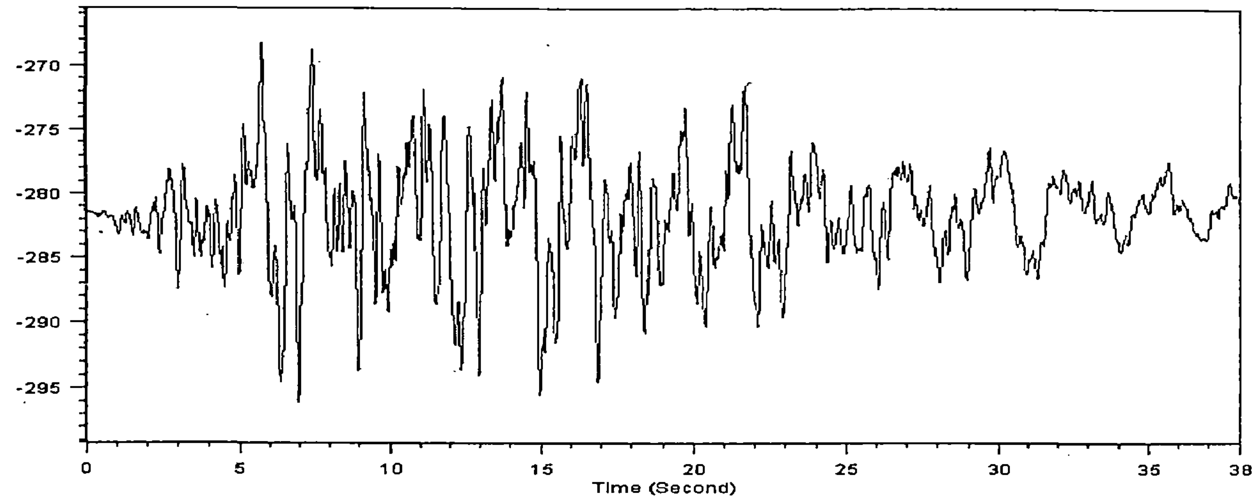


Major Principal Stress
(-194.6 to -243.8 t/m²)



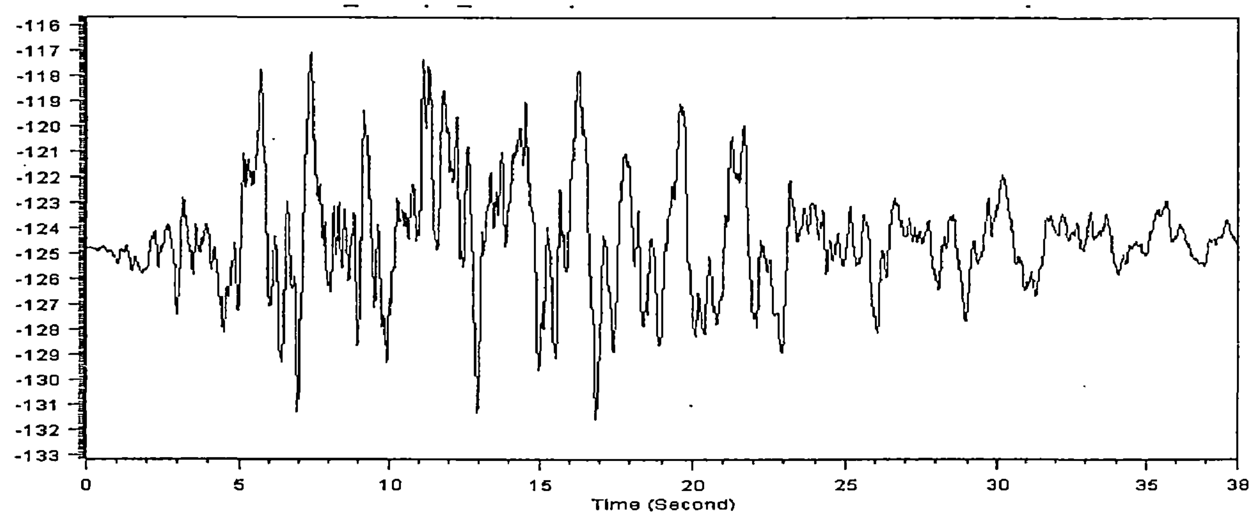
Minor Principal Stress
(-90.17 to -112.6 t/m²)

Fig: 5.23(c) Time histories of combined principal stress at the base of the core in inclined Core Section



Major principal stress

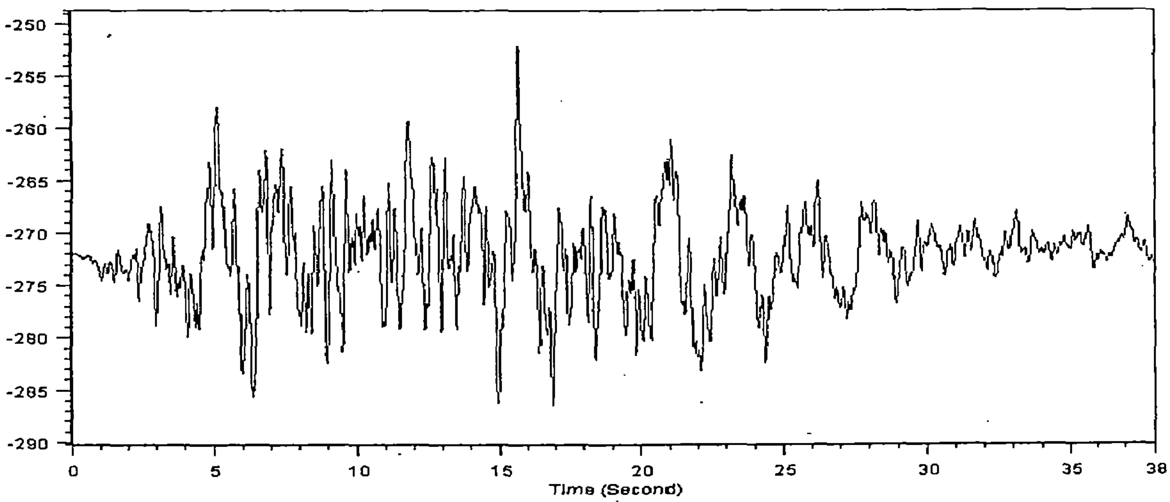
(-268.3 t/m^2 to -296.1 t/m^2)



Minor principal stress

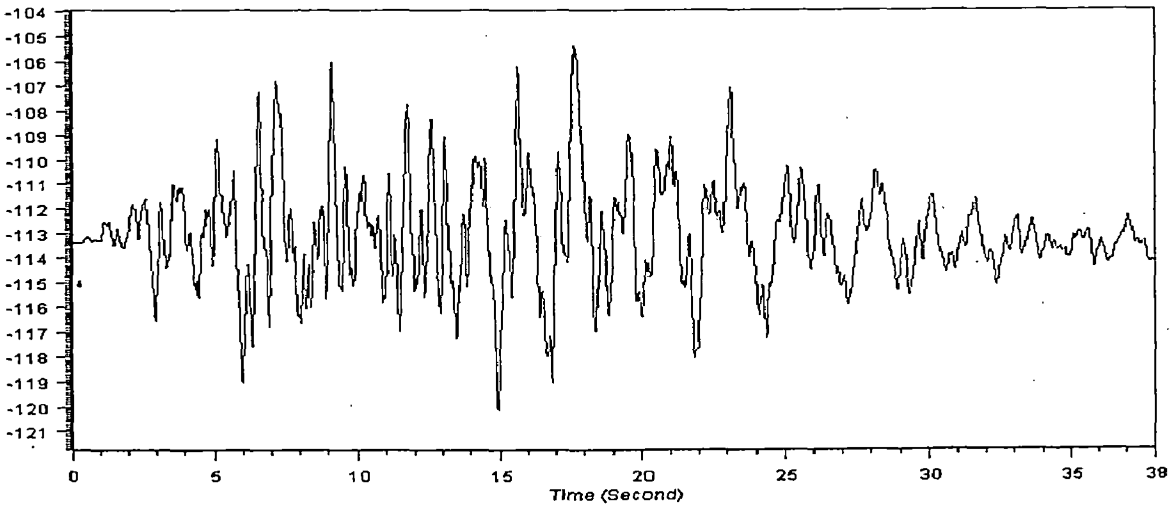
(-117.1 t/m^2 to -131.5 t/m^2)

Fig :5.24(a) Time histories of combined principal stress in element no.11 of the Central Core Section



Major principal stress

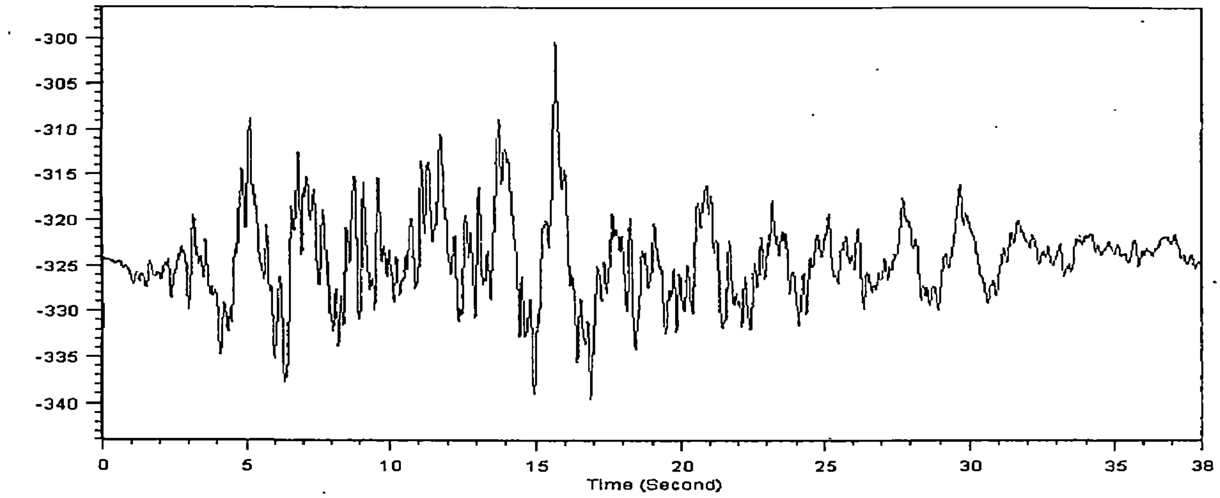
(-252.1 t/m^2 to -286.4 t/m^2)



Minor principal stress

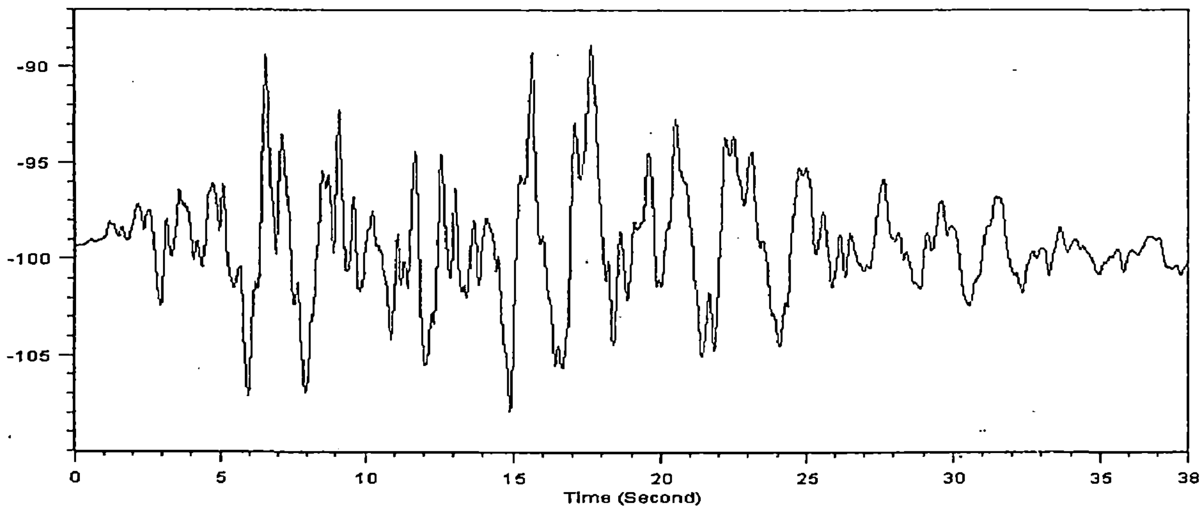
(-105.4 t/m^2 to -120.1 t/m^2)

Fig: 5.24(b) Time histories of combined principal stress in element no.11 of the Moderately Inclined Core



Major principal stress

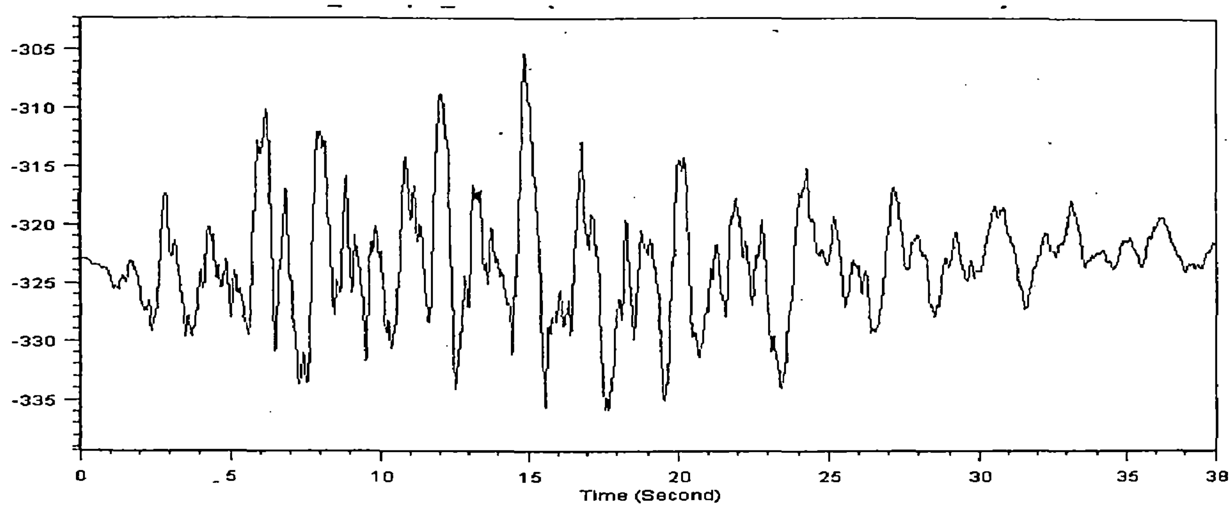
(-300.5 t/m^2 to -339.5 t/m^2)



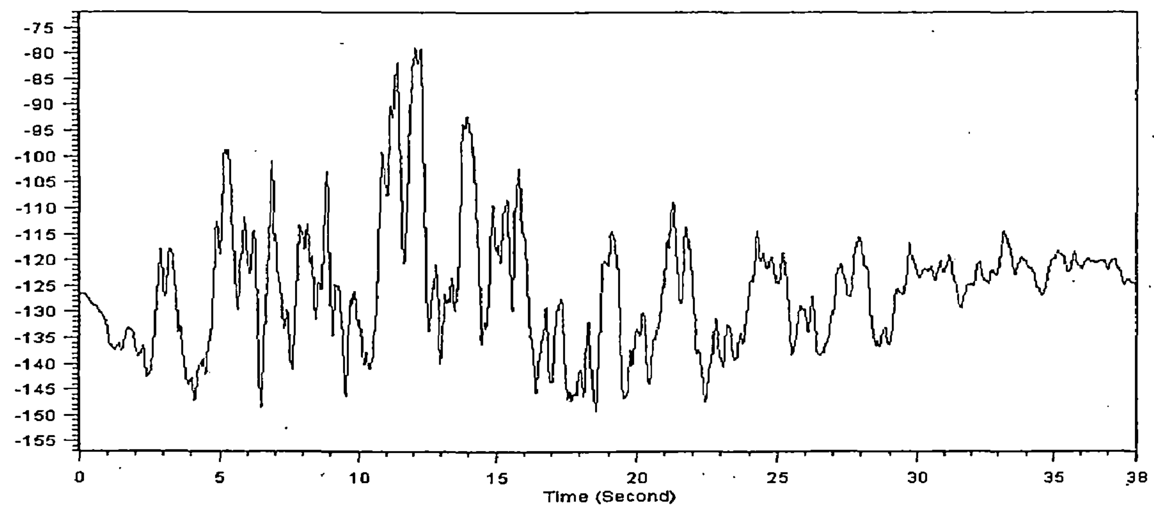
Minor principal stress

(-88.85 t/m^2 to -107.9 t/m^2)

Fig: 5.24 (c) Time histories of combined principal stress in element no.11 of the inclined core section

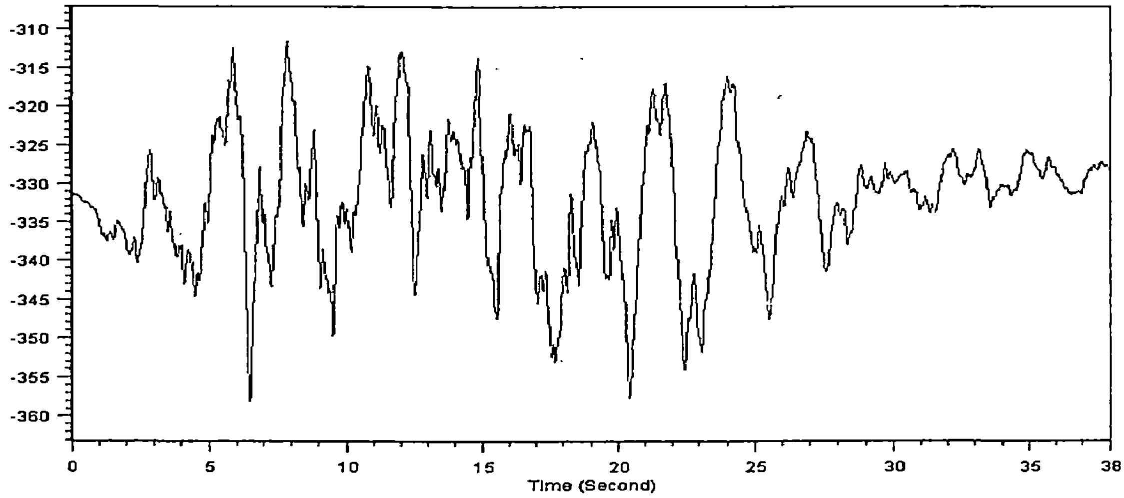


Major principal stress
(-305.3 t/m^2 to -335.9 t/m^2)

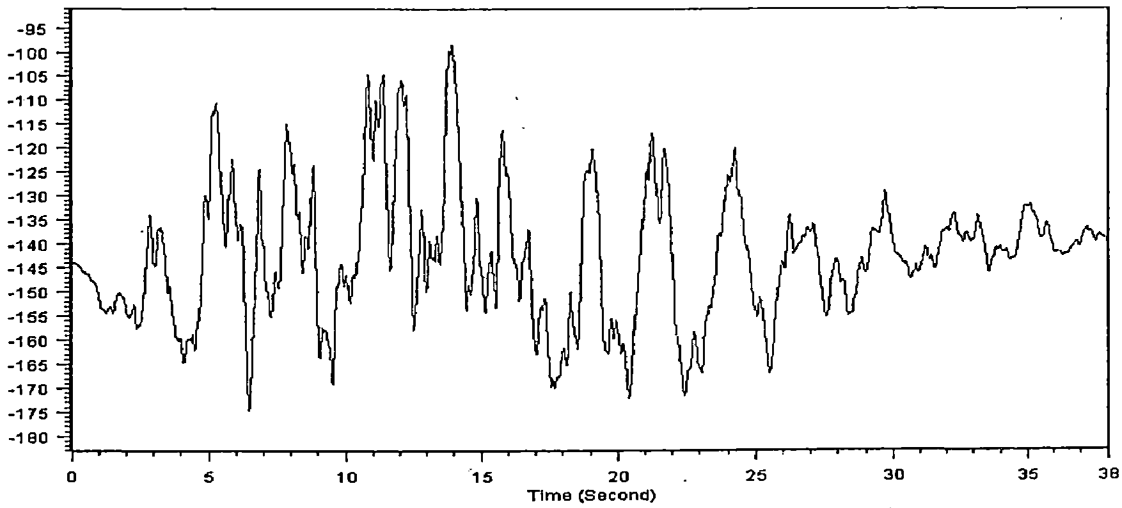


Minor principal stress
(-78.9 t/m^2 to -149.1 t/m^2)

Fig : 5.25(a) Time histories of combined principal stress in the element no.67 of the Central core dam

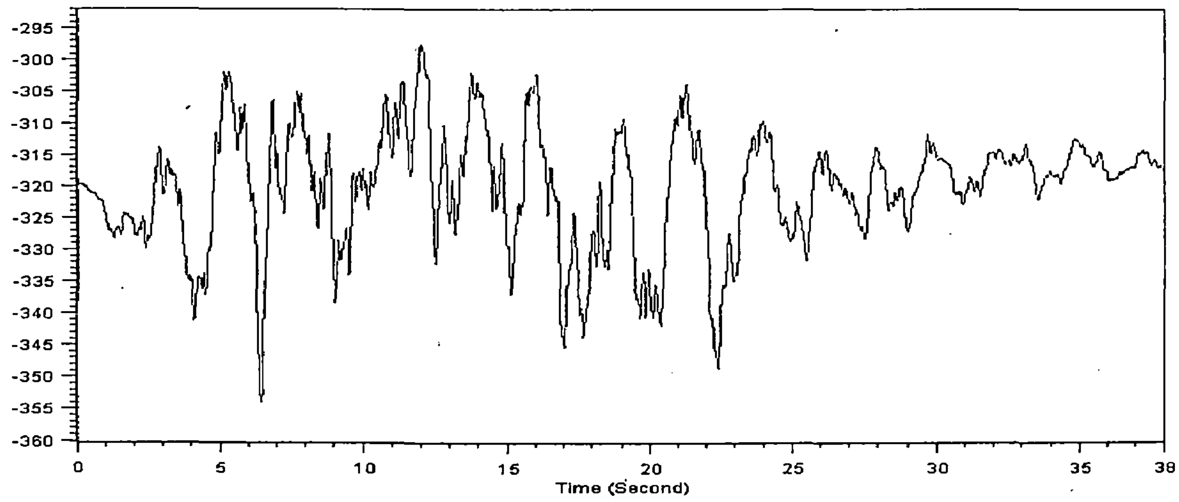


Major principal stress
(-311.7 t/m^2 to -358 t/m^2)



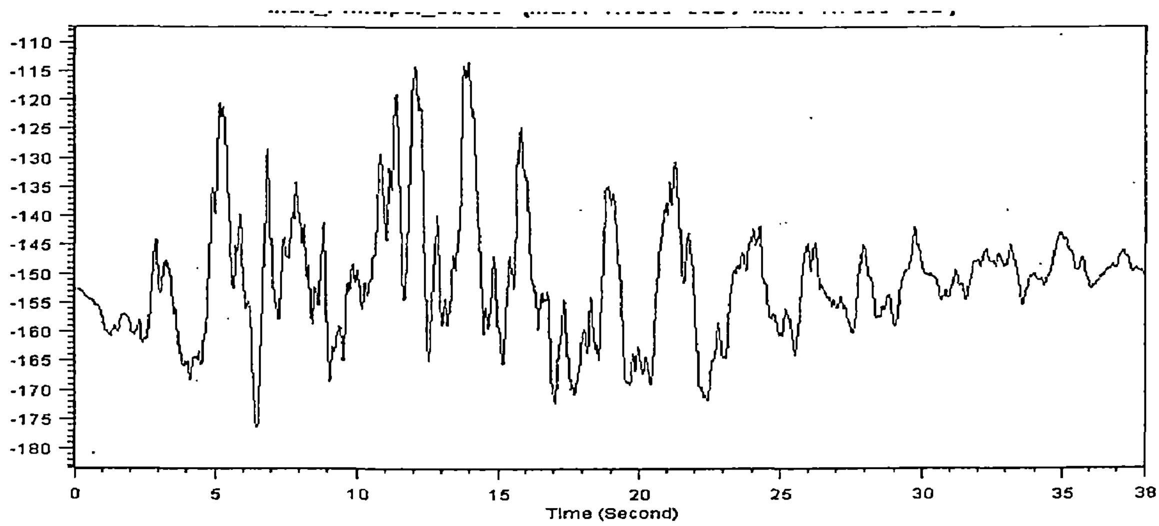
Minor principal stress
(-98.5 t/m^2 to -174.5 t/m^2)

Fig :5.25(b) Time histories of combined principal stress in the element no.67 of the Moderately inclined core dam



Major principal stress

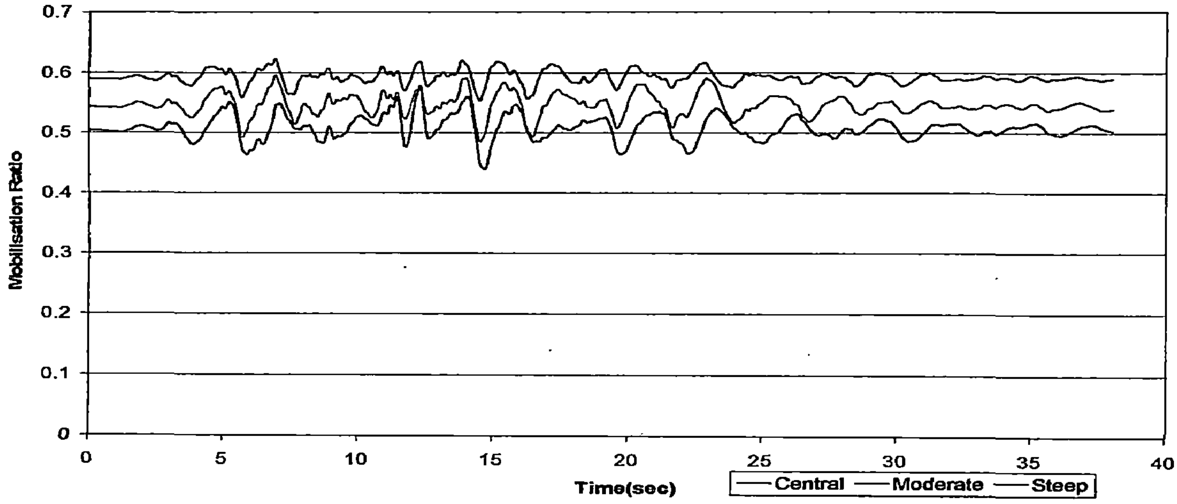
(-297.5 t/m^2 to -354 t/m^2)



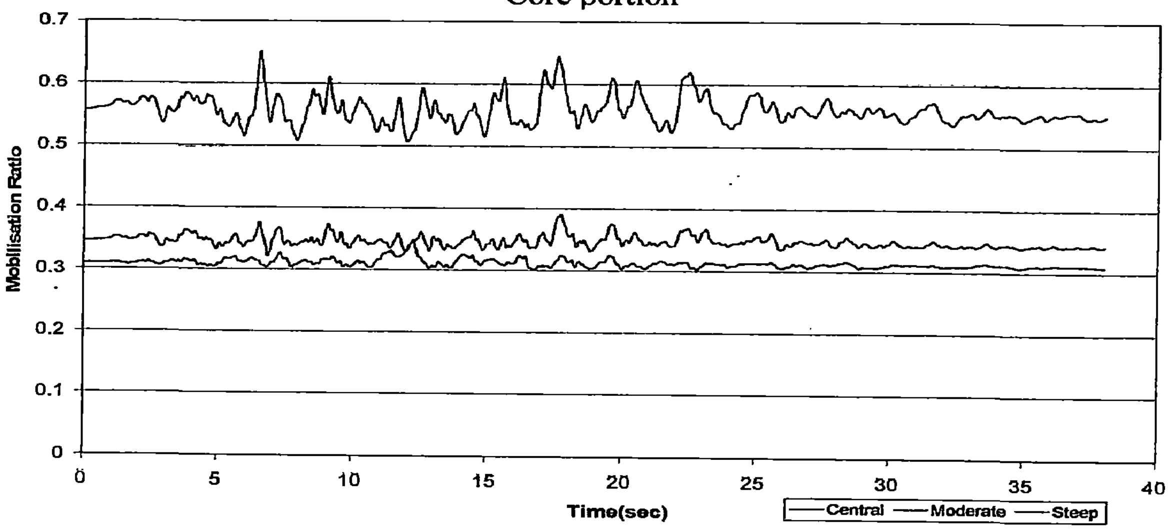
Minor principal stress

(-113.5 t/m^2 to -176.3 t/m^2)

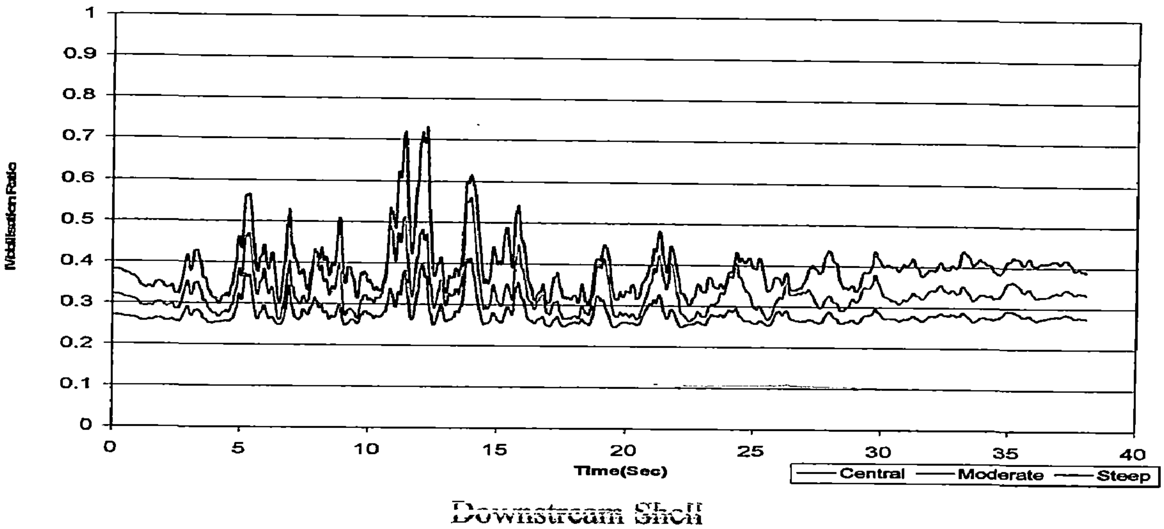
Fig : 5.25(c) Time histories of combined principal stress in element no.67 of the inclined core dam



Core portion



Upstream shell portion



Downstream Shell

Fig: 5.26 Variation of Mobilisation ratios in three dam sections based on combined stress

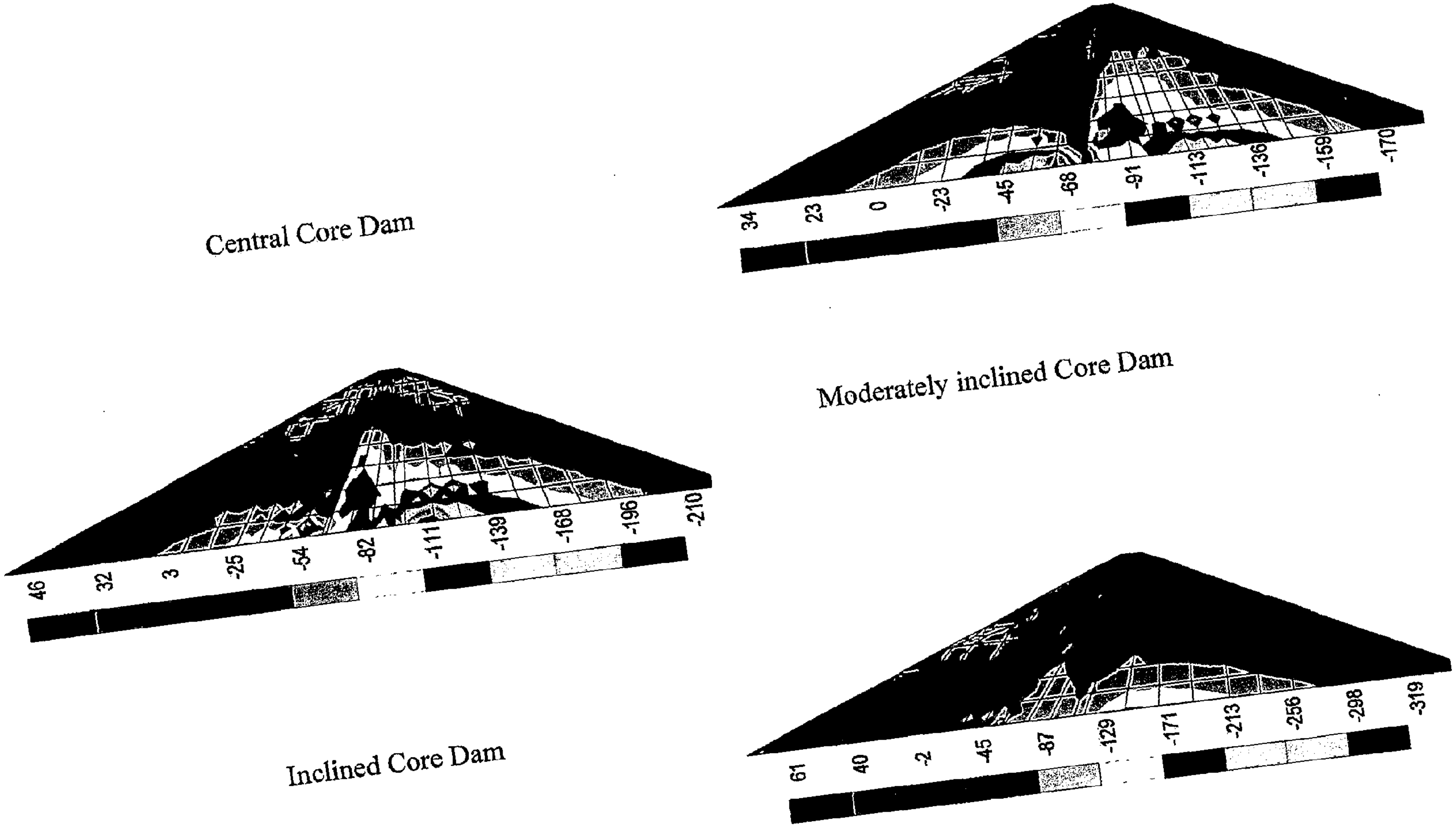


Fig 5.27 Combined Stress contours at the time of maximum tensile stress during earthquake

CHAPTER 6

CONCLUSIONS AND SCOPE FOR FUTURE STUDY

As part of the present study, single stage static analysis of the central core dam, nonlinear multi stage static analysis, free vibration analysis and linear dynamic analysis of all the three alternative dams were carried out. The inferences on the variation of the stress and deformation pattern between single stage and nonlinear multi stage analysis and the study wise conclusions are presented here under:

6.1 SINGLE STAGE ANALYSIS & MULTI STAGE ANALYSIS

1. The settlement distribution pattern was quite distinctive in the case of gravity turn-on analysis from that of the sequential analysis. At the end of construction as well as at the full reservoir stages, the maximum settlement occurred at the crest of the dam in single stage analysis while it occurred at a height of 80 m (mid height) from the base in the later. In both the stages, the maximum vertical settlements were more in case of single stage analysis.
2. The horizontal displacement distribution pattern in the gravity turn analysis as well as multi lift analysis was found to be similar. Higher displacements were observed in the multi lift analysis.
3. The principal stress pattern remained same in two analyses and the single stage analysis showed more stresses than the later.
4. Settlement in the core remained more as compared to shell at the same level due to high compressibility of the core in both the analyses.

6.2 CONCLUSIONS ON STATIC MULTI STAGE ANALYSIS

6.2.1 Static Displacements

As far as static vertical displacements are concerned the deformations were almost equal at the end of construction stage in the central, moderately inclined core dams but higher deformations were observed in the inclined core dams. At the full reservoir stage, the deformations in vertical core dam were minimum.

In case of horizontal displacements, the vertical core dam experienced little bit higher deformations than the other two dams though the variation was marginal.

6.2.2 Static Stresses

The principal stresses in the core are of the same order irrespective of the inclination of the core in reservoir empty condition. The maximum principal stresses in the shell portion increased from central core to the inclined core dam. Out of the three sections, the central core dam experienced the least shear stresses in the core as well as in upstream shell portion. However, higher stresses were observed in the downstream shell portion of the central core dam. In load transfer from core to shell perspective, the central core dam has slight advantage over the other two alternatives. In the core portion, the mobilisation ratio also remained low in case of central vertical core dam as compared with other two alternatives.

For the reservoir full case, the stresses in the central core sections were the lowest. The major principal stress reduced in the upstream shell and increased in the downstream shell portion in all the sections due to buoyancy of the upstream shell and water pressure on the impervious core. The mobilisation ratio in the core portion remained low in the central core dam section. Tension zone appeared in all the three alternatives in the reservoir full stage, but the tensile stress values show increasing trend from central core to inclined core sections. Hence, central core section is preferable.

6.3 CONCLUSION ON DYNAMIC ANALYSIS

6.3.1 Displacements and Acceleration

The inclined core section experienced minimum horizontal and vertical displacements under seismic loading. The variation of acceleration at the crest of the three sections differed marginally.

6.3.2 Stresses

The inclined core section experienced higher combined stresses (Dynamic + Static) than the other two sections. Thus for sloping core dams the total stress distribution was less favourable than other two alternatives.

Large tension zones were formed at the junction of the shell and core in vertical core section and at the top of the core in moderately inclined core dams, as compared to the tension zones at the upstream slopes observed in the sloping core section. This makes sloping core section preferable in seismic zone.

6.4 HYDRAULIC FRACTURING

The results reveal that all the three alternative sections are safe against horizontal cracking, but susceptible to cracking along the vertical plane up to $2/3^{rd}$ height above base. Since the tensile strength of the core material was not considered in computations, it is expected that the cracking may not be significant.

6.5 RECOMMENDATIONS

The vertical core dam emerges as the preferable option under the static loading condition in view of less vertical deformations, favourable principal and shear stress distribution in the dam and low mobilisation ratio in the core portion. Under the seismic loading, the inclined core dam is found to be better option over the other two alternatives so for as safety of the core is concerned.

6.5 SCOPE FOR FURTHER STUDY

1. In the present study the foundation is considered as rigid and the nodes at the base were fixed, but in real life situations the foundation conditions may not be so. The study can be extended to take care of other foundation conditions also.
2. Plane 2 D analysis provides an accurate representation of long dams of uniform cross section. However, this may not provide proper representation of dams in narrow valleys where cross valley stress transfer plays an important role. Thus the study can be extended with 3D FEM model.

3. The study has been carried out for one height and fixed dam slope. Furth studies can be carried out for different heights and different dam slopes.

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