AN ASSESSMENT OF MODELING ALTERNATIVES OF CONCRETE GRAVITY DAM FOR UPLIFT PRESSURE

A DISSERTATION

Submitted in partial fulfilment of the

requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

EARTHQUAKE ENGINEERING

(With specialization in Structural Dynamics)

By

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CANDIDATE'S DECLARATION

I, hereby, declare that the study, which is being presented in this dissertation titled **"AN ASSESMENT OF MODELING ALTERNATIVES OF CONCRETE GRAVITY DAM FOR UPLIFT PRESSURE"** for the partial fulfilment of the requirements for the award of the Degree of Master of Technology in Earthquake Engineering with specialization in Structural Dynamics submitted to the Department of Earthquake Engineering, Indian Institute of Technology Roorkee, is an authentic record of my own carried out under the supervision of **Mr. A.D.Pandey,** Assistant Professor, Department of Earthquake Engineering, Indian Institute of Technology Roorkee, Roorkee, India.

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

Date:

Place: Roorkee (**Tiyyagura Brahma Reddy)**

CERTIFICATE

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

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Date:

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ABSTRACT

It is well known fact that massive structures like dam attract lot of external interests other than engineering. It may not be always possible to build a dam on safe locations from engineering and geological point of views. It is, therefore, a necessity to carry out analyses considering unfavourable locations or foundations and study the behaviour and assess the safety of structures like dam, having huge destructive potential.

Uplift Pressure due to seepage is the major problem for most of the Hydraulic Structures like Dams, Weirs and Barrages. It is always a challenge to know the uplift pressure distribution at super and sub structure interface due to seepage. The interaction between the dam wall, the impounded water, the tail water and the foundation results in development of Uplift and pore pressure.

In this dissertation work an attempt is made to determine the uplift pressure head distribution at super and sub structure interface. For this study representative dam models are adopted. As the governing differential equation is same for seepage flow and heat flow, thermal analysis is adopted to determine uplift heads in terms of nodal temperatures. The obtained results from seepage analysis are compared with code based provisions.

In stress analysis the effect of discontinuity at dam foundation interface is simulated using the contact or interface elements in ANSYS. The static as well as dynamic analyses are performed on dam models with different foundation interactions. The analytical results obtained from both static and dynamic cases show that the finite element model can simulate the discontinuity effects due to friction when compared to models which do not consider the friction into account. The analytical results, further, indicate that the continuous and the discontinuous models differ in the static and dynamic responses under the loading considered.

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1 INTRODUCTION

1.1 GENERAL

A Concrete gravity dam is a solid structure that is constructed across a river/lake to create a reservoir on its upstream side. These are the structures those so proportioned such that all the external forces acting on it are resisted by its own weight. Uplift is an active force that exists within both the foundation and the concrete structure itself. Large uplift pressures can compromise the structural adequacy of such structures and serve as the principal cause of failure. Finite element method is used to develop mathematical models for concrete dam profiles. Structural responses of the two dam profiles taken are evaluated using finite element software ANSYS.

1.2 IMPORTANCE OF STUDY

The primary goal is to assess the modelling alternatives for application of uplift pressure. Generally in the analysis of dams for predicting the state of stress the foundation conditions adopting are

(a) Rigid foundation, which can be modelled as fixed base

(b) Dam and foundation considering together as homogenous body, which can be modelled such that no relative displacement between them

But in real field condition, dam and foundation are different structural components placed one on other and having relative displacement between them. The seepage water can enter the intersection between them. The relative displacement can be taken into mathematical model by providing contact elements at the intersection of dam and foundation. The seepage water exerts equal forces in all directions. So, the percolated water through the intersection between dam and foundation exerts equal pressures on base of dam and top of foundation.

In the present study, dam-foundation interaction is modelled in three different ways. They are (i) dam with fixed base (ii) dam and foundation glued together and (iii) dam foundation interaction using contact elements. The uplift pressures were applied differently in each case along with other loads such as gravity, hydrostatic and earthquake. The state of stress and displacements are compared in three models. The prediction of uplift pressure distribution at the dam foundation interface is always a challenge. In this present work an attempt is made to predict the uplift pressure heads below the dam with the help of seepage analysis using flow net concept.

1.3 ORGANIZATION OF DISSERTATION

In chapter 1, the problem is introduced briefly which discusses about the objectives of the dissertation. In chapter 2, the literature review is done which comprises of developments in seepage analysis, modelling developments for uplift pressures and dam foundation intersection modelling and also a brief review of development of intersection elements are presented. Chapter 3 deals with analytical method description which includes description of finite element analysis of seepage problem, structural analysis and brief description regarding contact analysis. Chapter 4 gives the details of the various modelling alternatives adopted. Chapter 5 illustrates the details of the data adopted for the study that gives the various dimensions for the gravity sections, the assumptions, details of mathematical modelling of concrete gravity dam with foundation, material properties and parameters used for this study. In chapter 6, analysis procedures, load combinations and results obtained from seepage analysis and comparison with Indian Standard, the stress analysis and comparison of stress analysis results among the models and its factor of safety against sliding and overturning are done. In chapter 7, the brief discussion of results and conclusions that are drawn from the present study are presented.

2 LITERATURE REVIEW

2.1 INTRODUCTION

Concrete gravity dam is a hydraulic structure consisting of impervious material that is built across a river to create a large natural source on its upstream side for locking up water for various purposes. Site selection is done after topographical and foundation requirements are satisfied and after conducting engineering, geological and hydrological surveys the type of the dam that has to be constructed is arrived at. The dam construction project initially starts by carrying out a preliminary design based on assumed preliminary dimensions. These are checked based on internal stresses and stability considerations and before arriving at the detailed design all the requirements of stability should be satisfied by the proposed design. This is followed by construction, testing and operation. Subsequent section deals with the literature pertaining to the seepage problem development and modelling developments for uplift pressure.

2.2 REVIEW OF PAST CASE STUDIES IN MODELLING OF UPLIFT PRESSURE AND BASE SLIDING IN CONCRETE DAM

There are research papers that deal with problems on sliding of base of dam with respect to foundation on which it is laid and modelling alternatives for inclusion of uplift pressure. Among those the relevant case studies are discussed here

Michael McKay and Francisco Lopez [6] in their paper on "Practical methodology for inclusion of uplift and pore Pressures in analysis of concrete dams" carried out work on inclusion of uplift in analysis of dams and they have proposed techniques to apply the estimated uplift pressure for its use in FEA of concrete dams. The methodology accounts for the inclusion of a prescribed uplift and pore pressure condition in the FE model, in a way that it can be readily combined with the other loads acting on the dam, and a final effective state of stress can be calculated. Being a coupled analysis, both the steady-state flow and the stress analysis are performed in the same model. In the steady-state analysis the water heads corresponding to the reservoir and the tail water are applied and the boundary conditions of the system are calibrated until the resulting hydraulic pressure in the body of the dam and the foundation replicates the prescribed uplift and pore pressure field. In the stress analysis, the hydraulic pressures obtained from the steady-state flow analysis are automatically converted into a load case that can be combined with other conventional loads in the dam such as gravity, temperature, hydrostatic and hydrodynamic solicitations. The results of the example for the steady-state flow analysis showed that the model reproduced the prescribed state of uplift and pore pressures in the dam body. In the example the stress analysis results showed the significant difference between the effective and total stresses, and hence, the importance of including the effect of the uplift and pore pressure in the structural analysis of concrete dams.

FERC proposes a method of application using a thin interface layer of elements, which allows the uplift pressure to be applied to the bottom of the dam and the top of the foundation. While this procedure maybe easily implemented in 2-D models, its application in 3-D models is impractical because it constraints the geometry of the foundation to simple geometries (for instance, abutments with inclined excavations need to be modelled as stepped excavations), and also because it increases the number of elements in the model when the thin layer interface elements are projected to the foundation domain. The FERC guidelines make no reference to the application of pore pressures within the dam body. USBR (2006) mentions that the uplift pressures can be applied as surface pressures to the opposite faces of contact elements. This method can be implemented in 2-D and 3-Dmodels, but it is limited to geometric nonlinear models, where the dam-foundation interface is able to crack.

Djamel Ouzandja et al. [5] in their paper on “Effects of Dam–Foundation Contact Conditions on Seismic Performance of Concrete Gravity Dams” presented the effects of dam–foundation contact conditions on seismic performance of concrete gravity dams including base sliding. The OuedFodda concrete gravity dam which located in Chlef, is selected as example, linear as well as nonlinear seismic analyses were performed. In addition to that a parametric study based on the coefficient of friction is carried out. The contact interface at dam–foundation interaction was modeled by contact elements that represent the frictional contact. Surface-to-surface contact elements which based on the

Coulomb’s friction law were used to describe the friction. These contact elements used target surface and contact surface to form a contact pair.

From the numerical results obtained in their study, they concluded that

(a) The presence of frictional contact along the dam– foundation rock interface can lead to the dam base sliding phenomenon and amplification of horizontal displacements.

(b) When the friction contact is not considerable the sliding displacement at the dam–foundation rock interface will cause instability of the dam.

Anil K. Chopra et al. [4] in their paper on “Earthquake-Induced Base Sliding of Concrete Gravity Dams” they made an attempt to explore the problem of earthquake induced sliding of gravity dams. The following conclusions were drawn from this exploratory investigation of earthquake-induced sliding displacement of concrete gravity dam monoliths

(a) The dam tends to slide in the downstream direction easily because very small ground acceleration is enough to initiate sliding in downstream direction to upstream sliding.

(b) The permanent displacement which is at the end of the earthquake found to increases with intensity of ground shaking which is larger for systems with smaller acceleration, which results from smaller frictional coefficient of sliding, steeper slope of downstream face and increasing the depth of impounded water, or increasing in uplift force.

A.Burman et al. [7] in their study titled “Coupled gravity dam–foundation analysis using a simplified direct method of soil–structure interaction” performed a time domain transient analysis of a concrete gravity dam and its foundation has been carried out in a coupled manner using finite element technique and the effect of Soil–Structure Interaction (SSI) has been incorporated using a simplified direct method. A two dimensional plane strain dam–foundation model has been used for the time history analysis to compute the stresses and displacements against earthquake loading considering the effect of soil–structure interaction. An effective boundary condition has been implemented by attaching dashpots to the vertical boundaries. The material damping effects have also been considered and the dam and foundation have both been modelled as linear, elastic materials. The dam like structure, having the coupling effect due to the under- lying foundation material during earthquake excitations is analysed.

They concluded that the displacements and stresses have increased for the elastic as compared to the rigid base. Hence it is advisable to carry out the interaction analysis for massive structures like dams under flexible base. It is also observed that the neck is the most severely stressed zone; hence one may expect the appearance of cracks around the neck region of the dam.

2.3 BRIEF DESCRIPTION OF DEVELOPMENT OF INTERFACE ELEMENTS

A.J. Crichton et al. [8] in their paper on “Developments in Non-Linear Finite Element Analysis for Dam Safety Management” had described the use of Non-Linear Finite Element Techniques to assess the structural adequacy of gravity dams. The assessment of structural adequacy mainly included using the non-linear frictional contact elements to model the sliding between the adjacent barrels of dam. Contact elements are capable to simulate friction on interface by allowing transfer of force by friction under only compression.

In this study they had modelled an Arch Barrel with three different types of modelling considerations. The models created were “Fully Glued”, “Fully Decoupled” and “with Contact Elements” between the barrel sections. They found that after using contact elements the slip of joints got increased by 0.2 mm and the behaviour of structure is same as that of glued condition. They concluded from their study on safety assessment that the contact elements were employed to more accurately estimate the force transfer by friction between segments of the structures. They concluded that the contact tool is a potential in finite element assessment of structures which have defects and discontinuities such as weak seams in foundations.

Chavez and Fenves in 1995 had carried out study considering the sliding of dam body at the dam-rock interface. The study accounts for the nonlinear base sliding of the dam, frequency dependent response of the impounded water and the flexible foundation rock.

Based on the study of the results, it was necessary to consider the effects of dam-foundation rock interaction to obtain realistic estimates of the base sliding displacements of the dam. Although a dam remains stable after an earthquake, the base sliding deformation may damage the interface zone without a significant isolation effect for the dam body.

Application of finite element method to problem of steady state seepage in which the boundary of the seepage is fully known a priori have been given by Zeinkeiwicz, Mayer and Cheung.

The method was soon applied to problems governed by Laplacian's or Poisson's equations as these equations were related to the minimization of a functional. The first applications were to conduction of heat transfer.

2.4 BRIEF DESCRIPTION OF HISTORICAL DEVELOPMENTS ON STUDY OF SEEPAGE PROBLEM

The basis for subsoil flow was given for first time by hydraulician Darcy (1856). He proposed that the loss of head per unit length is directly proportional to flow velocity.

$$V = K.\frac{H}{L}$$

Where V – Flow velocity

L – Flow path length

K – Hydraulic conductivity

Forchheimer in 1911 has given a geometrical method which involved in plotting of stream lines and equipotential lines. This method called as flow net method which helps in determining potential heads at required points in the flow field.

In 1966 a new tool as substantial finite element software was developed and used in structural mechanics problem field, and now it became essential and being widely used in many different fields like transfer of heat, electromagnetic and fluid flow.

3 NUMERCIAL STUDY

3.1 FINITE ELEMENT ANALYSIS

3.1.1 FEA of Seepage Problem

Seepage problems are often solved for geotechnical engineering problems by sketching floe nets. This method has many problems like soil inhomogeneity, anisotropic nature and uniform permeability. For such cases Zeinkiewicz and Cheung gave a general method with use of electronic digital computer.

The method is based on the concept of representing the continuous distribution of pressure head in the region of flow by values of the head at a finite number of points. The region is divided into a network of triangles called finite element and the head or potential in each element is specified in terms of the values of the potentials at the nodes. Thus, the seepage problem is considered solved when the potentials at the nodes are known. Because the continuum id divided into finite elements, the method has been termed the finite element method of analysis.

The basic differential equation which governs the wide range of physical problems such as flow of seepage, heat conduction, electromagnetic or electrostatic fields etc., in steady state in two dimensions is given by

$$\frac{\partial}{\partial x}\left(Kx\ \frac{\partial \emptyset}{\partial x}\right)+\frac{\partial}{\partial x}\left(Ky\ \frac{\partial \emptyset}{\partial y}\right)=f(x,y)$$

Where f, k_x and k_y are functions independent of Ø, which is the field variable. k_x and $k_y$are constant for homogeneous medium and k_x= k_y=k for isotropic media.

In the 2D stress problem the field variable is displacement, in seepage problem case it is the potential or the total fluid head given by

$$\emptyset=\frac{p}{\gamma}+z$$

Where p is pressure exerted by fluid, γ is density of water and z is the datum head

In cases like flow problem it is the velocity potential given by

$v_x = \frac{\partial \emptyset}{\partial x}$ $v_y = \frac{\partial \emptyset}{\partial y}$

The following are the steps for FEA of seepage problem

Step 1 : Discretise the field

Step 2 : The fluid potential have to select as field variable and a linear model

Ø = [N] {q}

Gradient of Ø is given by

$$\{g\} = \begin{bmatrix} \frac{\partial}{\partial x} \\ \frac{\partial}{\partial y} \end{bmatrix} \emptyset = [L]\emptyset$$

$$\{g\} = \begin{bmatrix} \frac{\partial}{\partial x} \\ \frac{\partial}{\partial y} \end{bmatrix} [Ni \quad Nj \quad Nk]\{q\}$$

{g}= [B] {q}

Step 3 : Define the constitutive law

Hook’s law is used for structural problems, similarly for the case of seepage problems we use Darcy’s law which state that

V= k i and Q= v A

$$\{v\} = \begin{Bmatrix} vx \\ vy \end{Bmatrix} = -\begin{bmatrix} kx & 0 \\ 0 & ky \end{bmatrix} \begin{Bmatrix} \frac{\emptyset}{x} \\ \frac{\emptyset}{y} \end{Bmatrix}$$

= [D] {g}

This is generally written as

{v}= - [R] [B] {q}

This is almost similar to stress equation, where {v} equal to discharge intensity per unit area, [R] equivalent to [D] and which is the permeability matrix

$$[R] = \begin{bmatrix} kx & o \\ 0 & ky \end{bmatrix}$$

Step 4 : Determining the element stiffness or element equations

Step 5, 6, 7& 8 : These steps are exactly same and identical. The secondary unknowns are discharge intensity, velocities and discharge

3.1.2 Contact Analysis

Application of the contact interface element and contact mechanics are very common and essential in mechanical engineering particularly in machine design. These contact problems are nonlinear to most extent and require adequate, significant resources. There are mainly two significant difficulties present with contact problems. First, beforehand the regions where contact exists are not known. Depending on the loads, boundary conditions, material properties and other factors, surfaces may come into and may go out of contact with respect to each other in most unpredictable and abrupt way. Second, in most of the contact problems friction involve. There are several models and frictional laws exist to choose from which all are nonlinear. Frictional response sometimes can be chaotic due to which the convergence will be difficult.

The most commonly used friction law associated with contact elements given by Coulomb's law. The law can be represented as $R_T = \mu_0 R_{in}$

Where R_T is tangential load

R_{in} is normal load and

μ_0 represents coefficient of friction.

A sample segment of contact boundary is shown in figure 3.1

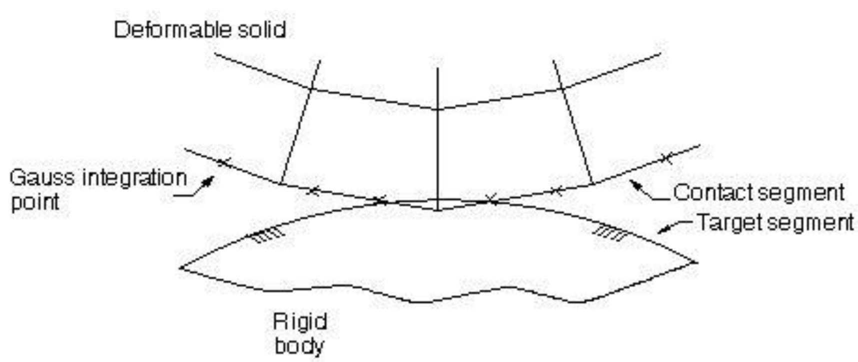


Figure 3.1 Segment of the contact boundary

Actually the micromechanics of the contact interaction exhibit significant three dimensional inhomogeneity (special technological features of different kind for the production of the friction surface, their breaking during operation, non-uniform action of lubricant, etc.,), as a result of which the local friction coefficient may vary along the contact surface. In this case, above mentioned relationship becomes invalid.

For all contact element pairs (over the entire contact boundary), and assuming that the global vectors of the tangential and normal stresses at the contact nodal pairs are liked by the relationship

$$\{P_T\} = [\mu]\ \{Pn\}$$

$$\{Rn\} = [\Omega]\ \{Pn\}$$

$$\{R_T\} = [\Omega]\ [\mu]\ \{Pn\}$$

And $\{R_T\} = [\mu_g]\ \{Rn\}$

Where $[\mu_g] = [\Omega][\mu][\Omega]^{-1}$

Thus it can be noted that when the frictional coefficient does not vary along the contact surface

$[\mu_g]=[\mu_0]\ I$

4 MODELING TECHNIQUES

In this work two different dam sections are considered, for each section three models are created by changing the foundation type and dam foundation interaction. The uplift pressure application is different in each model and described as below

4.1 DAM WITH FIXED BASE

In this model dam is modelled with restraining all degrees of freedom at base i.e fixed base. In this case the uplift pressure is applied on the nodes just above the base of the dam.

4.2 DAM AND FOUNDATION GLUED TOGETHER

In this model dam section and foundation are created separately and glued together in order to make them as homogenous structure. The uplift pressure is applied on the nodes that are at intersection between dam and foundation.

4.3 DAM AND FOUNDATION CONNECTED WITH INTERFACE ELEMENTS

In this model dam section and foundation are created separately and surface to surface contact is defined at the interface of dam and foundation. As the water applies equal pressure in all directions, the uplift pressure is applied upwards on base of dam and downwards on top of foundation.

The uplift pressure distribution is assumed to be an intensity that at the line of drains exceeds the tail water pressure by one-third of the differential between head water and tail water head. The pressure gradient is then extended to head water and tail water respectively in straight lines. In this work it is assumed that drains are chocked during analysis to produce values on conservative side.

Uplift distributions according to the dam foundation interface considered are shown in following page as fig 4.1 to fig 4.3 for dam section 1

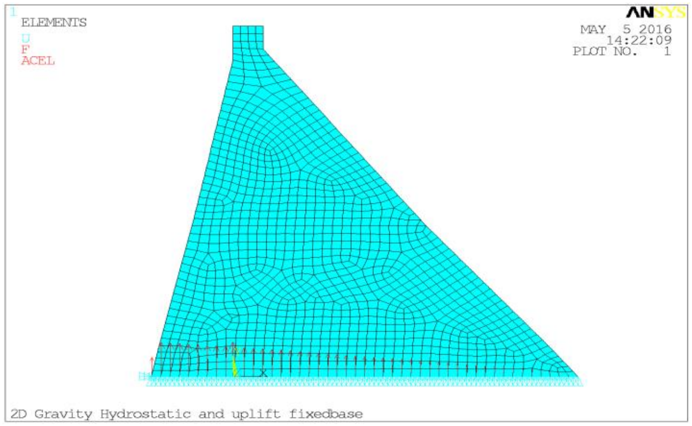


Figure 4.1 Uplift distribution in model with fixed base

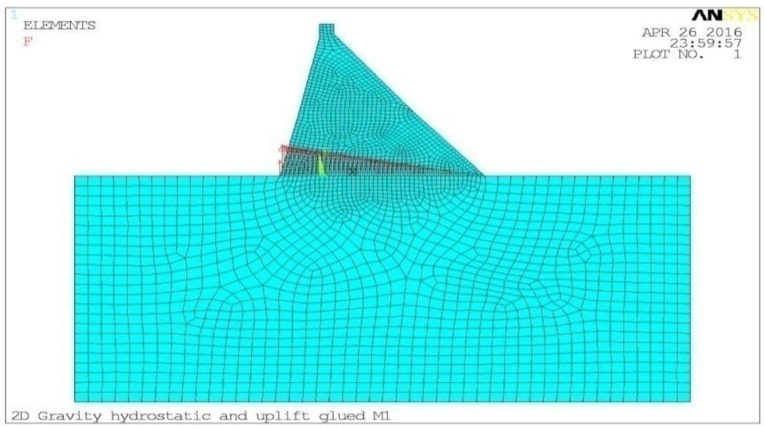


Figure 4.2 Uplift distribution in model with foundation and dam glued together

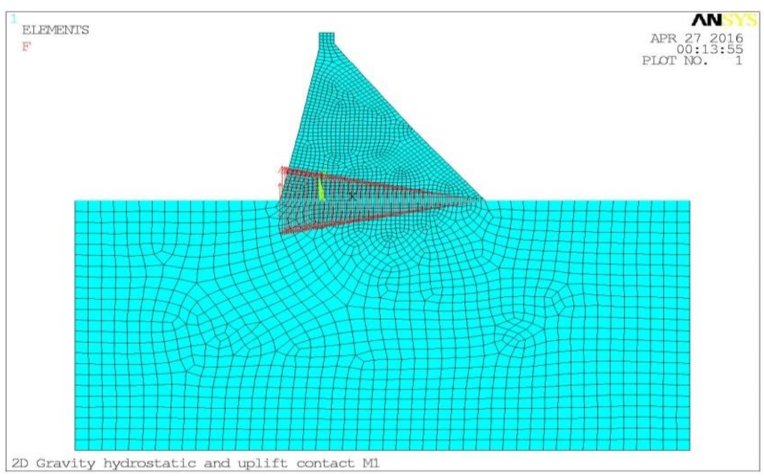


Figure 4.3 Uplift distribution in model with foundation and dam connected with interface elements

5 MATHEMATICAL MODELS ADOPTED

5.1 GENERAL DESCRIPTION OF THE DAM MODEL

The dam and foundation are modelled as 2D plane stress model for finite element analysis. Three types of models were created for each dam section base on dam foundation interaction. One is restricting the displacement at bottom of the dam. Second one is considering the all displacement fields continuous between dam and foundation. The last one in which the discontinuity is modelled using contact interface which obeys Coulomb's law of friction between dam and foundation.

Concrete gravity dams are usually constructed as monoliths with joints normal to the dam axis. The joints may be either a straight joint with grouting or can be a keyed joint. Even though the joints slip at small amplitude of vibration, the different monoliths act together [1]. But during a large vibration problem, the behavior is determined entirely by the inertia force transferred across the joints. The inertia force that develops for such large vibration problems are very high when compared with the shear strength of a simple straight grouted joint [2]. This results in the slipping of joints as a result of which the monoliths vibrate independently. This was also clear from the studies conducted in Koyna dam during the 1967 earthquake [3]. In such cases, a 2D plane stress idealization can be adopted for modeling. But, for dams in which the transfer of force across the individual monoliths are ensured by keyed joints and for roller compacted dams in which the dam is made without any transverse joints, a plain strain idealization is preferred. The same idealization is adopted for embankment dams wherein the entire length of dam is made monolithically. But these idealizations does not hold good if the dam is situated in a V-shaped valley or in a narrow canyon where the monoliths will have a stepped or a tapering cross section. In such cases, the torsional effects may become predominant which the 2D models are not capable of representing.

The element that is used for modelling the non-overflow section for conducting the Convergence Studies in ANSYS is the PLANE183 element. This element is 2-D, 8-

noded element which having quadratic displacement behaviour and it is well suited for modelling irregular meshes.

This PLANE183 element is defined by 8 nodes having 2 degrees of freedom at each node; they are translations in both x and y directions. The geometry and nodes pattern are shown in figure 5.1. This element can be used as a plane element (plane stress or plane strain) or as an axisymmetric element. This element has plasticity, creep, hyper-elasticity, large strain, large deflection capabilities and stress stiffening.

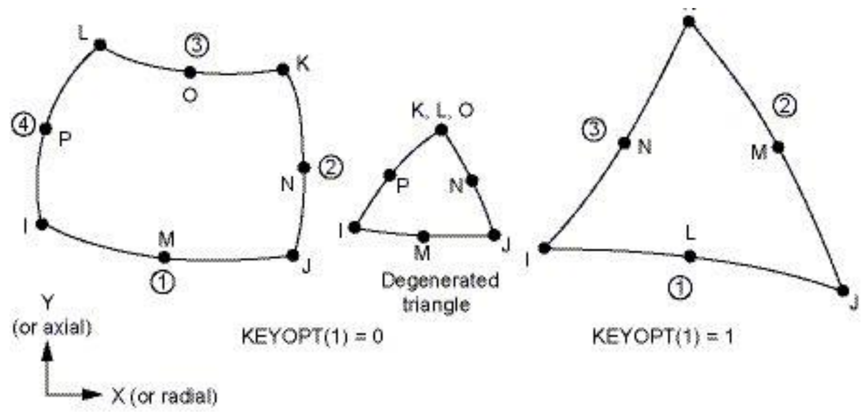


Figure 5.1 Plane 183 Element Degree of freedom

Foundation is considered in two of the modelling alternatives of the dam section. The concrete material used for the dam and foundation is assumed as isotropic, linearly elastic and homogenous. This linear behaviour of concrete is defined by assuming the properties like constant poisons ratio and constant modulus of elasticity to the dam as well as foundation media.

The boundary conditions imposed on the mathematical model are such as to replicate the prototype behaviour with regard to the displacements at the extremities of the finite boundary of the foundation media. The general procedure adopted is to restrain the boundary nodes of the finite foundation media against displacement normal to the boundary face i.e. roller supports are assumed to enforce the required displacement constraints. In effect then, the left and right boundaries are prevented from moving horizontally but are free to move vertically while the base boundary is prevented from any vertical movement but is free to move horizontally.

The width and depth of the foundation for the 2 dam profiles adopted are obtained from convergence study on each individual profile of the dam which is explained in the convergence study section.

CONTA172 and TARGET169 are used as interface or contact elements.

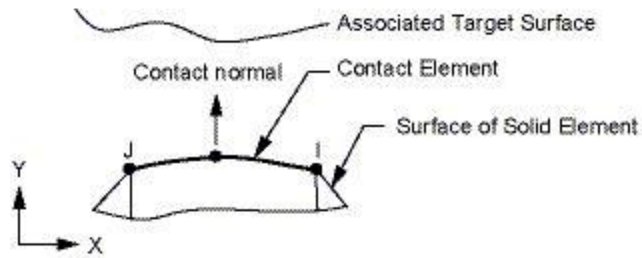


Figure 5.2 CONTA172 geometry

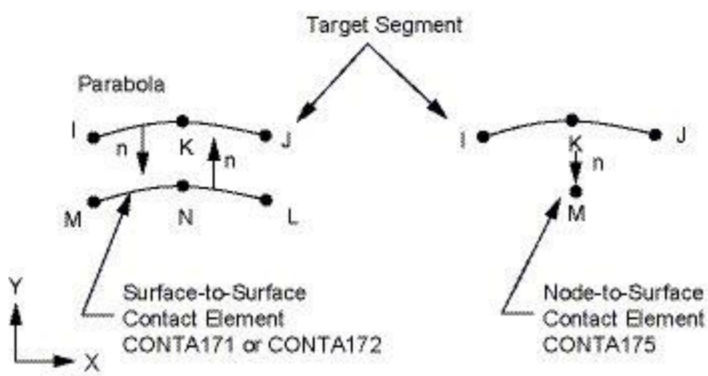


Figure 5.3 TARGE169 geometry

TARGE169 is used to represent various 2-D “target” surfaces for the associated 2D contact elements. The contact elements themselves overlay the solid elements describing the boundary of a deformable body and are potentially in contact with the target surface, defined by TARGE169. This target surface is descritized by a set of target segment elements (TARGE169) and is paired with its associated contact surface via a shared real constant set. Any translational or rotational displacement, temperature, voltage, and magnetic potential on the target segment element can be imposed. Forces and moments on target elements can be also applied. The typical geometry and behaviour of TARGE169 are shown in figure 5.3.

CONTA172 is used to represent contact and sliding between 2-D “target” surfaces (TARGE169) and a deformable surface, defined by this element. The element is applicable to 2-D structural and coupled field contact analysis. This element is located on the surfaces of 2-D solid, shell, or beam elements without mid side nodes. It has same goemetric characteristics as the solid, shell, or beam element face with which it is connected. Contact occurs when the element surface penetrates one of the target segment elements on a specified target surface. Coulomb and shear stress friction is allowed. The typical geometry and behaviour of CONTA172 are shown in figure 5.2.

5.2 GEOMETRICAL DESCRIPTION OF THE DAM MODEL

To understand the behaviour of interface elements and to compare the results with other kinds of foundation interactions two existing non-overflow sections of dams are adopted and they are termed as profile 1and profile 2 accordingly. The sectional details of the adopted dam sections are as shown in the table 1.

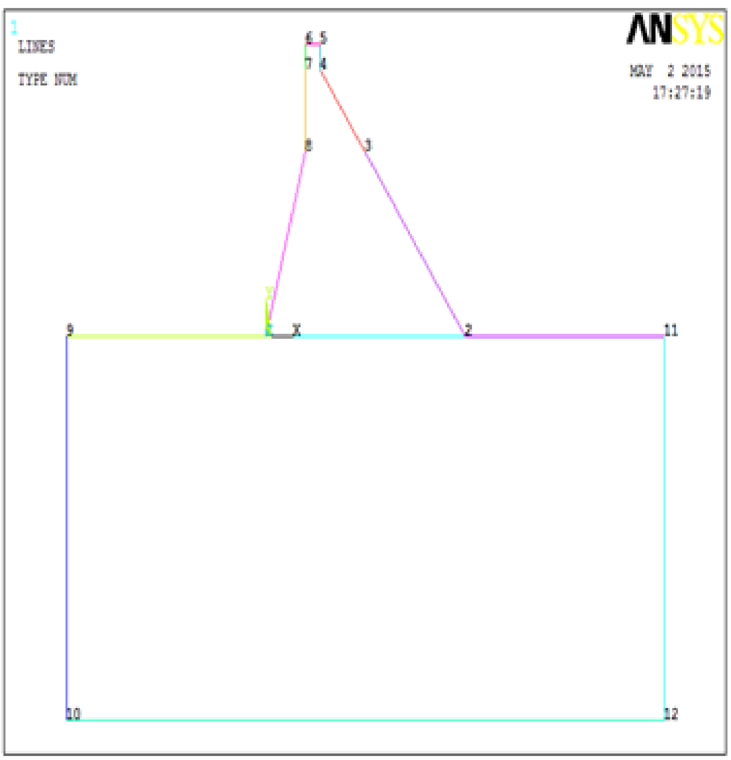
Table 1 Details of Two Different Dam Profiles adopted

S.No	Parameter	Dam profile 1	Dam profile 2
1	Total height of dam	114.024 m	150.6 m
2	Base width of dam	113.024 m	141 m
3	Free board	5 m	5 m
4	Crest width	8 m	10 m
5	Upstream slope	0.2090	0.300
6	Downstream slope	0.7830	0.74545
7	Volume of dam (unit thick)	6342.59 m^3	9906.625 m^3

The modelling, application of loads and load cases considered are similar in two profiles. The outlines of two models are shown in following fig 5.4 and fig 5.5 with key points

Key.no	x (m)	y (m)
1	-21.605	0.00
2	91.419	0.00
3	51.20	49.024
4	8.00	106.532
5	8.00	114.024
6	0.00	114.024
7	0.00	109.024
8	0.00	103.024
9	-10.80	49.024

Figure 5.4 Line Representation for Dam profile 1 with key points



Key.no	x (m)	y (m)
1	0.00	0.00
2	141	0.00
3	70.182	95.00
4	38.5	137.5
5	38.5	150.6
6	28.5	150.6
7	28.5	137.5
8	28.5	95.00
9	-141	0.00
10	-141	-197.4
11	282	0.00
12	282	-197.4

Figure 5.5 Line Representation for Dam profile 2 with key points

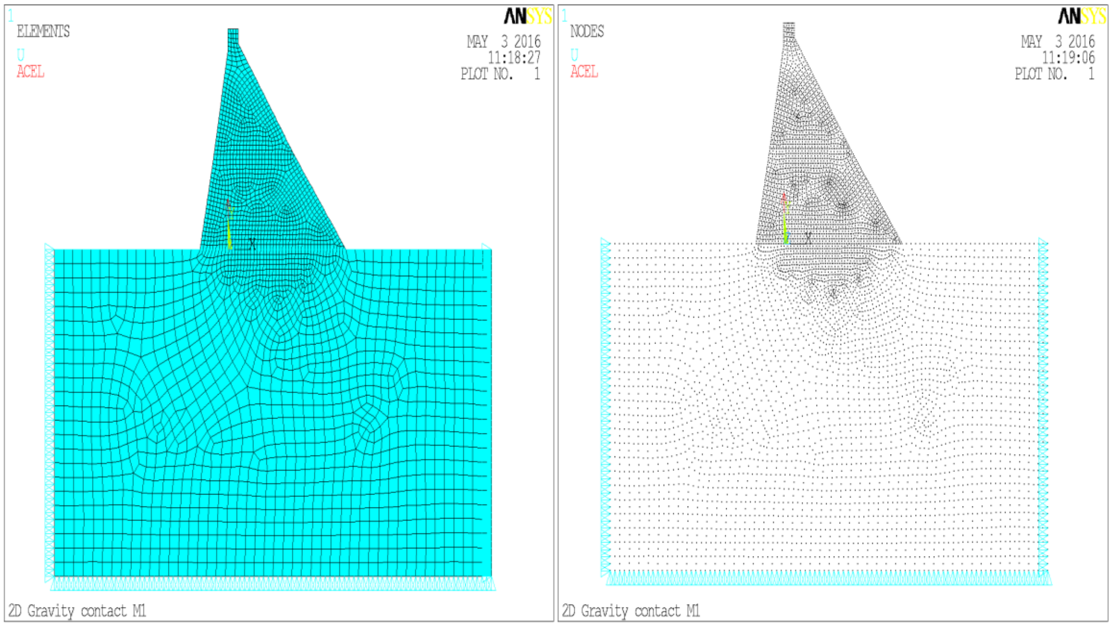


Figure 5.6 Elements and Nodes of dam profile 1

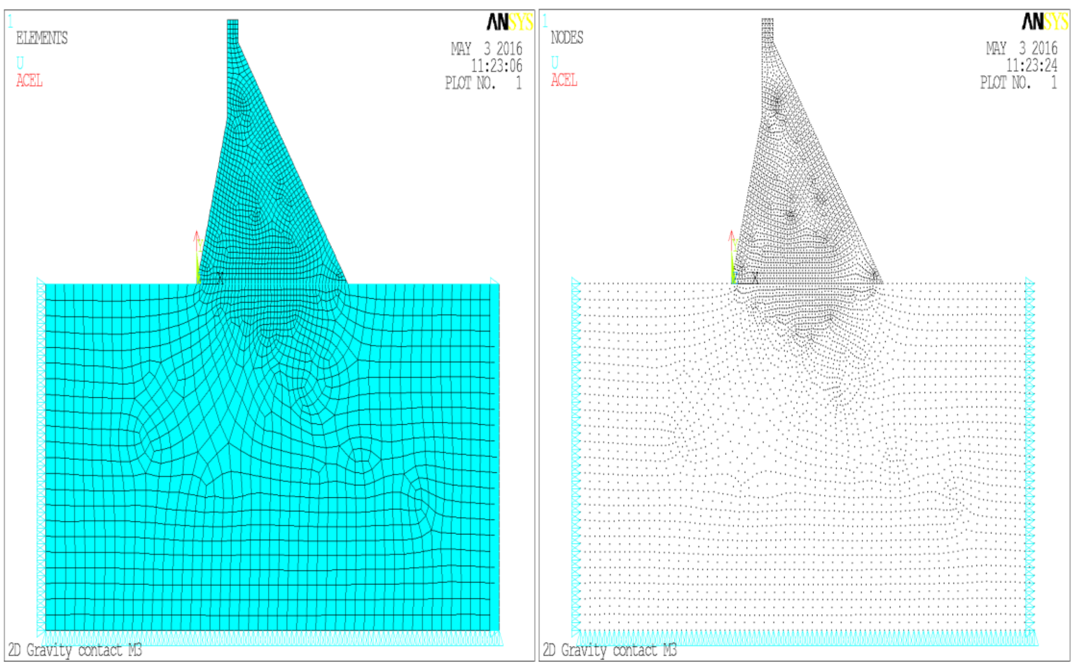


Figure 5.7 Elements and Nodes of dam profile 2

The nodes and elements after meshing for all two dam profiles are as shown in figures 5.6 and 5.7.

5.3 DESCRIPTION OF MATERIAL PROPERTIES

The concrete material which used in the analysis for dam is assumed to be isotropic, homogenous and linearly elastic.

The Non Overflow sections adopted for analysis is mainly composed of the materials properties those are given in tables 2, 3 and 4:-

Table 2 Material properties used in Non-Overflow Section1

	M15	RCC	Foundation
Mod. of Elasticity(N/m^2)	1.94E10	1.5E10	0.41E10
Poisson Ratio	0.2	0.2	0.25
Density(Kg/m^3)	24000/9.81	24000/9.81	2.7e-10/9.81

Table 3 Material properties used in Non-Overflow Sections 2

	M15	Foundation
Mod. of Elasticity(N/m^2)	1.94E10	0.41E10
Poisson Ratio	0.2	0.25
Density(Kg/m^3)	24000/9.81	2.7e-10/9.81

Table 4 Deployments of Materials in Non-Overflow Section for Dam profile 1

S. No.	Material	Description
1	Concrete M15	Below EL.1140 m (49.024m) for the Non-overflow portion.
2	RCC	AboveEl.1140 m (49.024m) for the Non-overflow portion.

5.4 BOUNDARY CONDITIONS

For the seepage analysis which performed with the help of thermal analysis, the boundary condition is applied as the boundary contains less coefficient of thermal conductivity. That is because; the heat flow should be avoided at the boundaries inorder to get the exact distribution of nodal temperatures. The boundary conditions for structural analysis model were explained in modelling foundation part.

5.5 CONVERGENCE STUDIES

The Finite Element Method (FEM) is a method in which an entity is can be discretised into finite segments or elements which are interconnected at the nodes. The nodes are capable of displacements as defined by independent components also known as the degrees of freedom i.e. horizontal and vertical displacements in a 2D planar body. Interpolation functions are then established for the variation of the displacement parameters across the domain of each individual element. The governing equations are developed for each element in the form of element stiffness matrices and assembled into the overall or system stiffness matrix to yield the load displacement characteristics of the entity. The application of constraints on the displacements would permit the solution of the problem due to any load case incident upon it.

Evidently then an issue which arises is the degree of refinement which would lead to an acceptable solution with regard to accuracy. In the earlier years of FEM, limited resources in terms of computing power and graphical support, a reasonable looking mesh was accepted and the solutions were never critically examined from the point of accuracy given by that particular mesh configuration. However, with the increase in computational power there now stands a need to establish the credibility of the solution with regard to accuracy provided by the adopted mesh. Convergence studies are therefore carried out before conducting any analysis to ascertain that the mesh configuration adopted yields the desired results within the limits of acceptable values in terms of permissible error. Meshes are adopted with varying degree of refinement progressively and the displacements and/or stresses at typical points/sections are monitored till two successive refinements of the mesh yield solution which vary within acceptable error limits. The coarser mesh is then adopted for the analysis with such refinement, if necessary, as not to influence the final outcome at those sections and/or parameters which were used for the convergence studies.

The task of carrying out the convergence studies has been divided into different phases as follows:-

a) Convergence study on Dam body
b) Convergence study on Foundation width
c) Convergence study on Foundation depth
d) Convergence study on Discretization of Foundation

The entire body of the non-overflow section was treated as a single area entity resulting in a random orientation of elements since mapped meshing could not be achieved due to the irregular shaped geometry. The SIZE was varied to ensure a reasonable mesh and the vertical stress distribution at the base was used as a benchmark for establishing the convergence criteria. For the dam profile 1, on the basis of the convergence study conducted adoption of SIZE=2.50 seems to be in order with regard to idealization of the Non Overflow section without compromising accuracy to any significant extent. The idealization of the foundation media for the convergence studies for dam profile 1 was based on the assumption that the width and the depth of the foundation media to be considered for analysis can be expressed as:-

Width of Foundation = $226\alpha + 113.024$ ------------------- (4)

Depth of Foundation = $114.024*(1 + \beta)$ ------------------- (5)

The recommendations of the convergence studies carried out indicate a value of $\alpha = 1.00$ and $\beta = 0.50$ which leads to a total width of 339.072 m and a depth of 169.536 m. The element edge sizes to be adopted on the basis of studies conducted are 2.50 m for the superstructure and 7.50 m for the foundation media.

For the dam profile 2, on the basis of convergence study conducted adoption of SIZE=3.00 seems to be in order with regard to idealization of the Non Overflow section without compromising accuracy to any significant extent.The idealization of the foundation media for the convergence studies for dam profile 2 was based on the assumption that the width and the depth of the foundation media to be considered for analysis can be expressed as:-

Width of Foundation = 282α + 141 ------------------- (8)

Depth of Foundation = 141*(1 + β) ------------------- (9)

The recommendations of the convergence studies carried out indicate a value of α = 1.00 and β = 0.40 which leads to a total width of 423 m and a depth of 197.4 m. The element edge sizes to be adopted on the basis of studies conducted are 3.00 m for the superstructure and 9.00 m for the foundation media. To have an idea on the convergence study adopted to obtain the size of the width and depth of foundation, line representation for dam profile 1 with parameters is shown in following figure 5.8

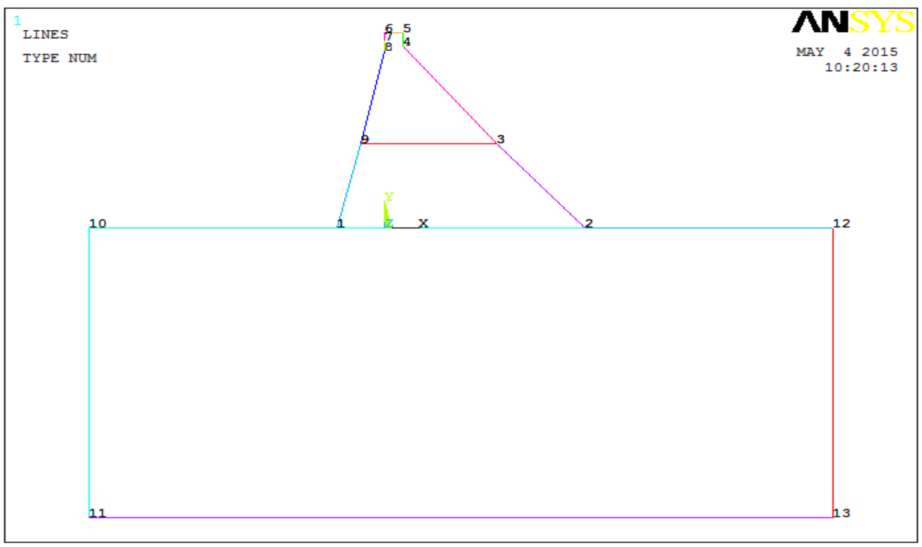


Figure 5.8 Line Diagram showing the extent of Foundation for Dam profile 1

Table 5 Values adopted in analysis of different dam profiles

Profile No	α	β	Foundation width B×(1+2α)	Foundation depth H× (1+β)	E size	F size
Dam profile 1	1.00	0.50	339.072 m	169.532 m	2.50 m	7.50 m
Dam profile 2	1.00	0.40	423.00 m	197.4 m	3.00 m	9.00 m

Where, B and H are Base width and Total height for the different Dam profiles

α, β are Multiplying factor for B in considering width and depth of foundation respectively

E size and F size = Element size and Foundation size adopted for meshing.

α, β are determined from the Convergence studies conducted for different dam profiles respectively

Usually the element size is adopted for the super structure and f size is for substructure which is the foundation part. The dimensions of the foundation as established from the above parameters may be rounded off nominally, if necessary, to facilitate the mathematical modelling without affecting the accuracy of the solutions obtained to any significant extent. After convergence study the width, depth and mesh sizes for all two dam profiles are presented in table 5.

5.6 ANALYTICAL PARAMETERS ADOPTED

In ANSYS, both continuous and contact the models have been analysed and especially for contact analysis some parameters involved on which the stability and convergence as well as accuracy of the analysis depends. The values of those parameters are presented in the table 6 given below

Table 6 Values of different parameters used in contact analysis

	Values	
Target Circle Radius (R1)	0	(Default)
Super element Thickness (R2)	1	(Default)
Normal Penalty Stiffness Factor (FKN)	10	(User)
Penetration Tolerance Factor (FTOLN)	0.0001	(User)
Initial Contact Closure (ICONT)	0	(Default)

Pinball Region (PINB)	2	(Default)
Upper Limit of Initial Penetration (PMAX)	0	(Default)
Lower Limit of Initial Penetration (PMIN)	0	(Default)
Max. Friction Stress (TAUMAX)	2.0e10	(User)
Contact Surface Offset (CNOF)	0	(Default)
Contact Opening Stiffness (FKOP)	1	(Default)
Tangent Penalty Stiffness (FKT)	10	(User)
Contact Cohesion (COHE)	0	(Default)
Static/Dynamic Ratio (FACT)	1	(Default)

ANSYS also has the option to choose specify the type of bonding exists between the contact surfaces. For this analysis no separation (always) bond with sliding permitted is applied to ensure the friction between the two surfaces takes its effect.

ANSYS also permits choice of several equation solvers like direct solver, Frontal solver and Iterative solver. However it is advisable to use a direct solver like sparse solver for the contact analysis if convergence is slow.

5.7 FREE VIBRATION CHARACTERISTICS

A modal analysis is carried out in ANSYS of the dam body to understand the free vibration characteristics and as well as the predominating frequencies of the structure. The first 10 modal frequencies corresponding to each dam section are given in following tables 7 and 8

Table 7 Natural Frequencies and Natural Periods of First 10 Modes for Section 1

	Fixed Base		Foundation Glued		Foundation Contact	
Mode	Frequency (Hz)	Time Period (Sec)	Frequency (Hz)	Time Period (Sec)	Frequency (Hz)	Time Period (Sec)
1	3.5745	0.27976	1.59268	0.62787	1.5919	0.62818
2	7.50146	0.13331	2.59184	0.38583	2.59149	0.38588
3	8.0373	0.12442	4.07566	0.24536	4.07536	0.24538
4	12.3516	8.10E-02	8.11079	0.12329	8.11043	0.1233
5	17.5356	5.70E-02	11.6067	8.62E-02	11.6061	8.62E-02
6	17.9253	5.58E-02	13.2387	7.55E-02	13.2384	7.55E-02
7	19.0613	5.25E-02	13.6886	7.31E-02	13.6879	7.31E-02
8	21.0128	4.76E-02	15.4846	6.46E-02	15.4834	6.46E-02
9	22.6326	4.42E-02	16.7343	5.98E-02	16.7333	5.98E-02
10	24.6669	4.05E-02	19.3732	5.16E-02	19.3718	5.16E-02

Table 8 Natural Frequencies and Natural Periods of First 10 Modes for Section 2

	Fixed Base		Foundation Glued		Foundation Contact	
Mode	Frequency (Hz)	Time Period (Sec)	Frequency (Hz)	Time Period (Sec)	Frequency (Hz)	Time Period (Sec)
1	2.8857	0.34654	1.29944	0.76956	1.29859	0.77007

2	5.46596	0.18295	2.13581	0.46821	2.13537	0.4683
3	6.93908	0.14411	3.17123	0.31534	3.17099	0.31536
4	8.99747	0.11114	5.42781	0.18424	5.4275	0.18425
5	13.0822	7.64E-02	9.04893	0.11051	9.04839	0.11052
6	14.4616	6.91E-02	10.2007	9.80E-02	10.2005	9.80E-02
7	16.7817	5.96E-02	12.0926	8.27E-02	12.0917	8.27E-02
8	17.8971	5.59E-02	12.5188	7.99E-02	12.5178	7.99E-02
9	18.2965	5.47E-02	14.3672	6.96E-02	14.3659	6.96E-02
10	20.3592	4.91E-02	15.0366	6.65E-02	15.0361	6.65E-02

5.8 PERMISSIBLE STRESSES FOR CONCRETE

The concrete used in the dam is M15, so the safe compressive stress is 15MPa and the tension of 2.25MPa can be allowed. The permissible tensile stresses are in accordance with recommendations of Dr. A. K. Chopra in his paper [10]

Table 9 Permissible Tension allowed in a Design mix

Mix	Permissible Tens.(KPa)
M15	2250
M20	3000
M25	3750

5.9 LOADS AND LOAD CASES

In the design of concrete gravity dams, it is essential to find out the loads that are required for the stability of the dam and stress analysis. The forces that mainly affect the design are:

(a) Gravity load (Dead load)

(b)Hydrostatic pressures

(c) Uplift pressure

(d) Seismic forces

These forces fall into two categories as:

i) Forces, such as water pressure and weight of the dam, which can be directly found from the unit weights of the materials and fluid pressures. These act as stabilising forces against all the external forces.

ii) Forces, such as uplift, earthquake loads, hydrostatic pressure which can only be found on basis of the assumption of varying degree of reliability and these acts as destabilising forces. It is in the estimation of this second category of the forces that care has to be taken based on the experience, available data and judgment.

(a) Gravity load- The self-weight of the dam which is the major stabilising force. For analysis purpose of view, generally unit length of dam is considered. The unit weight of concrete generally taken as 24000/9.81 Kg/m^3 as specified.

(b) Hydrostatic pressures – The water pressure acting on the upstream side of the dam are found from the meteorology, hydrology and reservoir regulation studies. The rate at which the different upstream water levels occurs need to be determined to find about which has to be used in the various load combinations used in the design. The hydrostatic pressure of the dam is a function of the water depth times the unit weight of water. The unit weight should be taken at 1.0 t/m^3, even though the weight varies slightly with temperature. Tail water is not considered in this analysis for all the three dam profiles. The hydrostatic pressure is applied similar to the distribution shown in figure 5.9

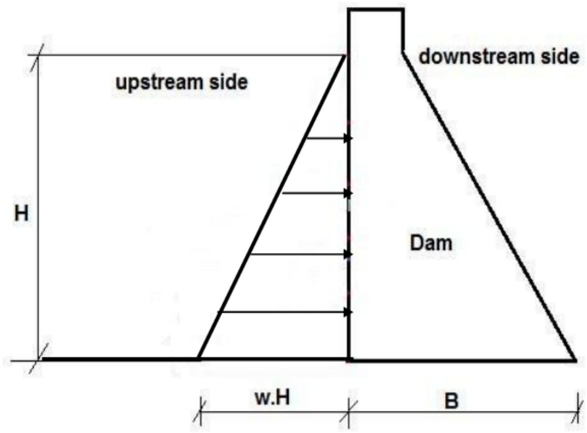


Figure 5.9 Hydrostatic Pressures on Dam

(c) Uplift pressure - Uplift pressure resulting from headwater and tail water penetrates through the cross sections within the dam body, within the foundation below the base of the dam and at the interface between the foundation and the dam. This pressure is present within the pores, joints, cracks that are present in the foundation and dam body. This uplift pressure is also an active force along with the hydrostatic force and these must be included in the stress and stability analysis to ensure that the structure is adequate. Uplift pressures are assumed to have no effect due to earthquake load. The different uplift pressure distributions considered are as given in figure 5.10

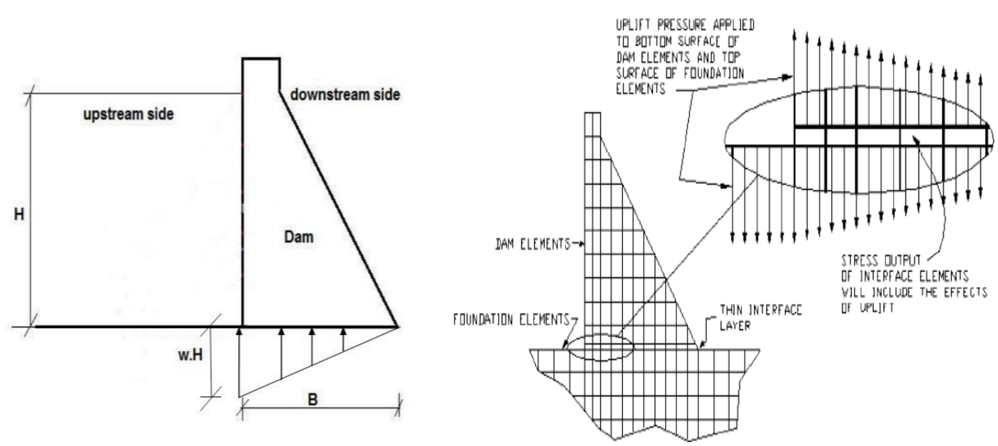


Figure 5.10 Uplift Pressures on Dam

The uplift pressure distribution is assumed to have an intensity that at the line of drains exceeds the tail water pressure by one-third the differential between head water and tail water head. In this work it is assumed that drains are chocked during analysis to produce values on conservative side.

(d) Earthquake forces - The response of the dam body to the vibration of the ground is mainly a function of the characteristics of foundation media, structural configuration of the dam, intensity and the duration of ground motion. The waves impart accelerations to the foundation under the dam and cause its movement. In order to avoid rupture the dam must move along with the foundation. This acceleration induces inertial force in the dam. So at the time of earthquake inertial forces are induced in the dam body. Inertial forces act at the centre of gravity in horizontal and vertical directions. It destabilizes the dam body significantly. Final designs are to be based on dynamic analysis using site dependant seismic parameters for which the Response Spectrum Analysis is employed.

5.9.1 Load Combinations

Based on IS Code 6512:1984, following load combinations have been considered in the present analysis:

a) Load Combination A (Construction Condition):- Dam completed but no water in the reservoir and there is no tail water.

b) Load Combination B (Normal Operating Condition):- Full reservoir elevation at FRL, normal dry weather, Normal uplift, tail water level.

c) Load Combination C (Flood Discharge Condition):- Reservoir at maximum flood pool elevation, all gates open, Normal uplift, tail water at elevation.

d) Load Combination D: - Combination A, with earthquake.

e) Load Combination E: - Combination B, with earthquake.

f) Load Combination F: - Combination C, but with extreme uplift (drains inoperative).

g) Load Combination G: - Combination E, but with extreme uplift (drains inoperative).

6 SEEPAGE AND STRESS ANALYSIS OF TWO DAM PROFILES

6.1 SEEPAGE ANALYSIS DESCRIPTION

The differential equation governing the flow of heat and seepage water is similar. The temperature and coefficient of thermal conductivity of heat flow are exactly similar with head and coefficient of permeability of seepage flow. In order to perform seepage analysis using ANSYS thermal analysis is adopted. The steps followed in seepage analysis in ANSYS are as follows

Step 1 : Creating the model

The element adopted in this analysis is PLANE77 which is a thermal solid element. The model dimensions considered for the two dam sections are defined in table 1.

Step 2 : Defining Material Properties

The structural material properties adopted for the two dam sections are defined in tables 2, 3 & 4. The thermal properties adopted in analysis are K_{xx}, which values are equal to 1e-10 and 0.864 for dam section and foundation respectively for both the sections.

Step 3 : Meshing into Elements and Nodes

The mesh sizes are adopted as values those came from convergence studies. The mesh sizes are defined in table 5.

Step 4 : Applying the Boundary Conditions

The water heads present on upstream and downstream of the dam are applied in the form of nodal temperatures. Those are shown as potentials in the below given figures 6.1 and 6.2. The boundaries of foundation are provided with less coefficient of thermal conductivity in order to avoid the flow of heat into surroundings.

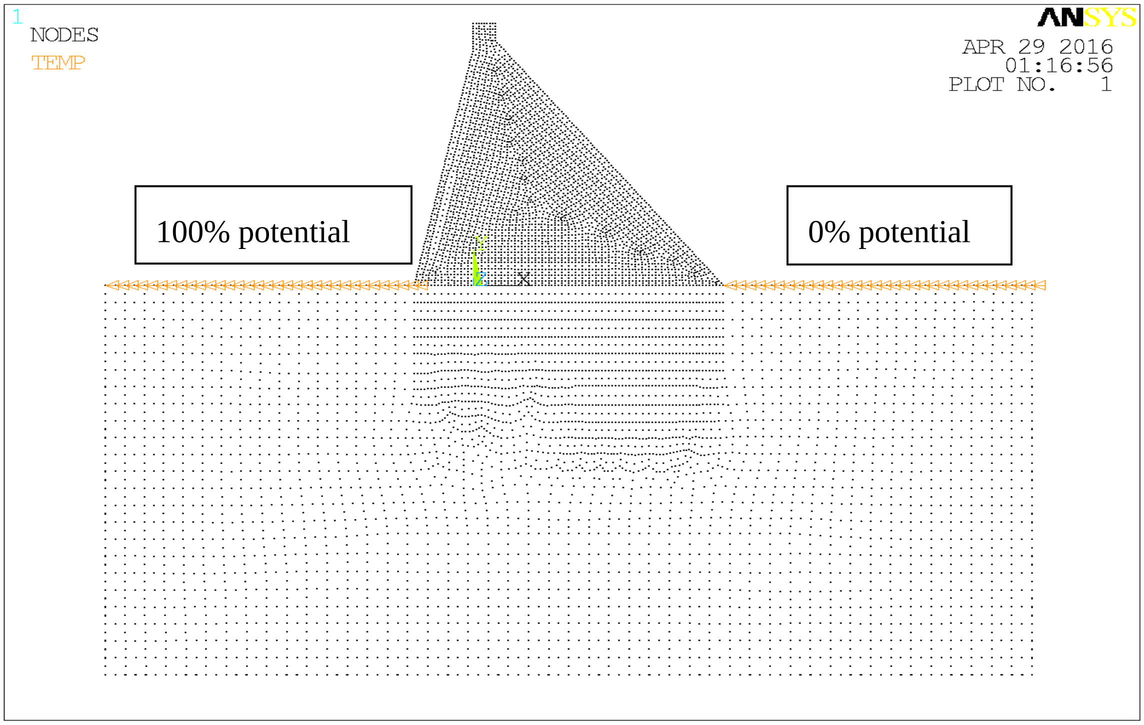


Figure 6.1 Meshed 2D seepage model for section 1

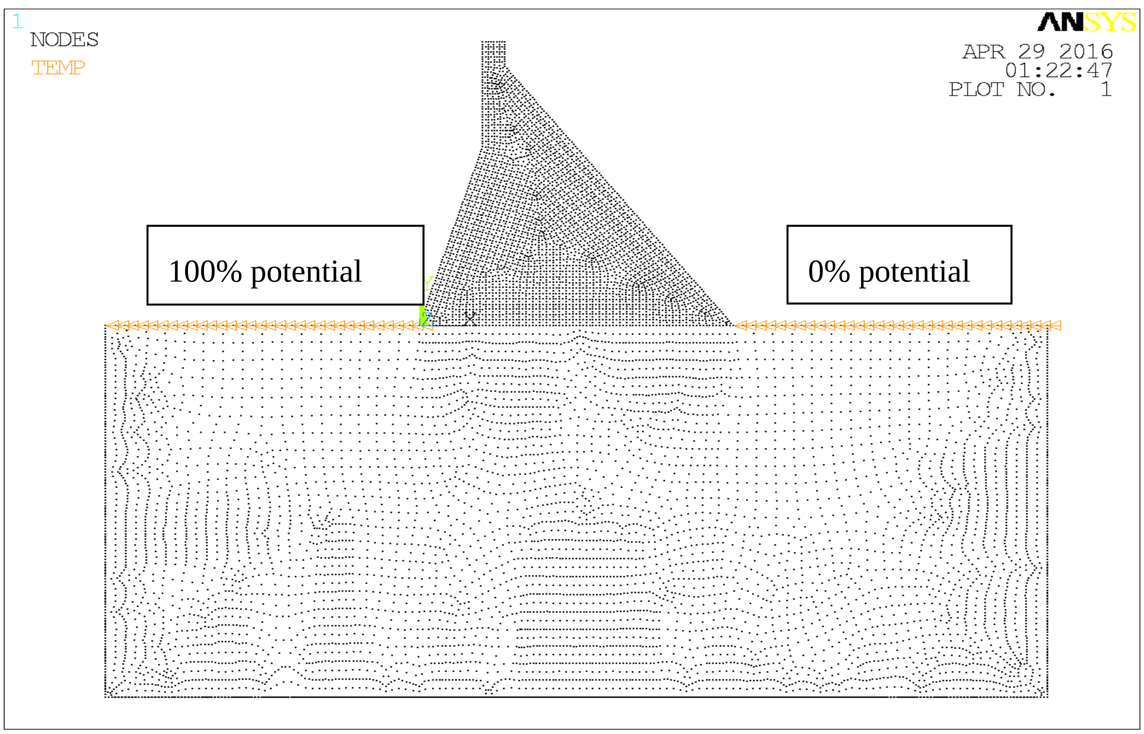


Figure 6.2 Meshed 2D seepage model for section 1

Step 5 : Obtaining the solution

Seepage analysis is performed as thermal analysis, the obtained pattern of uplift contours in terms of nodal temperatures are shown in below given figures 6.3 and 6.5

For Dam profile 1

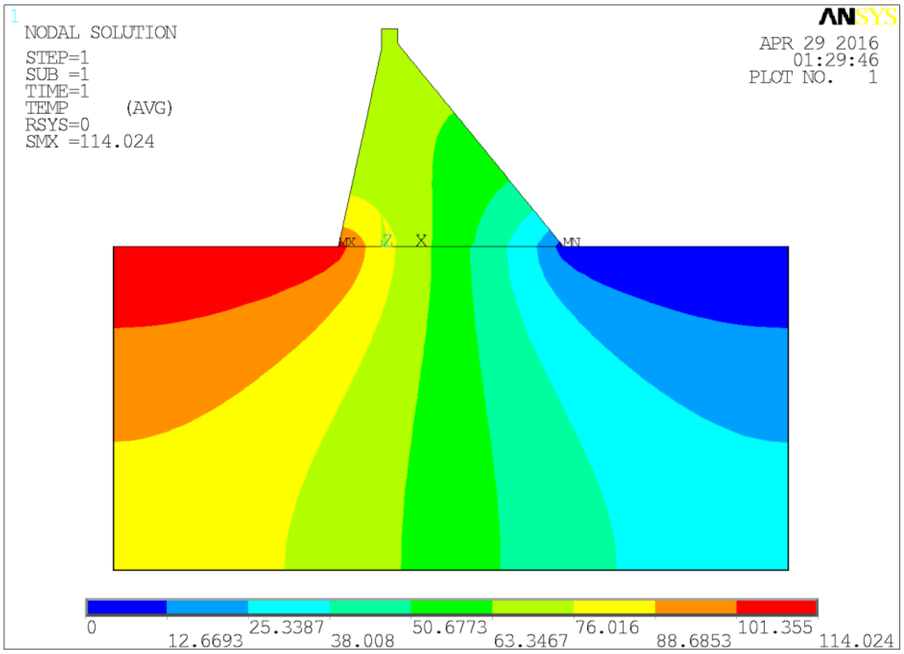


Figure 6.3 Uplift head distribution contours for profile 1

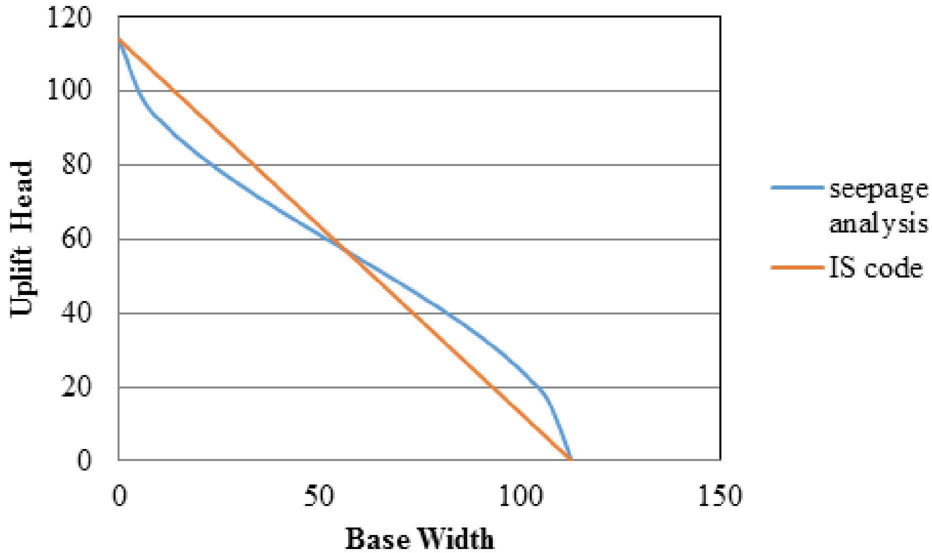


Figure 6.4 Comparison of uplift distribution at the base with IS code for profile 1

For Dam profile 1

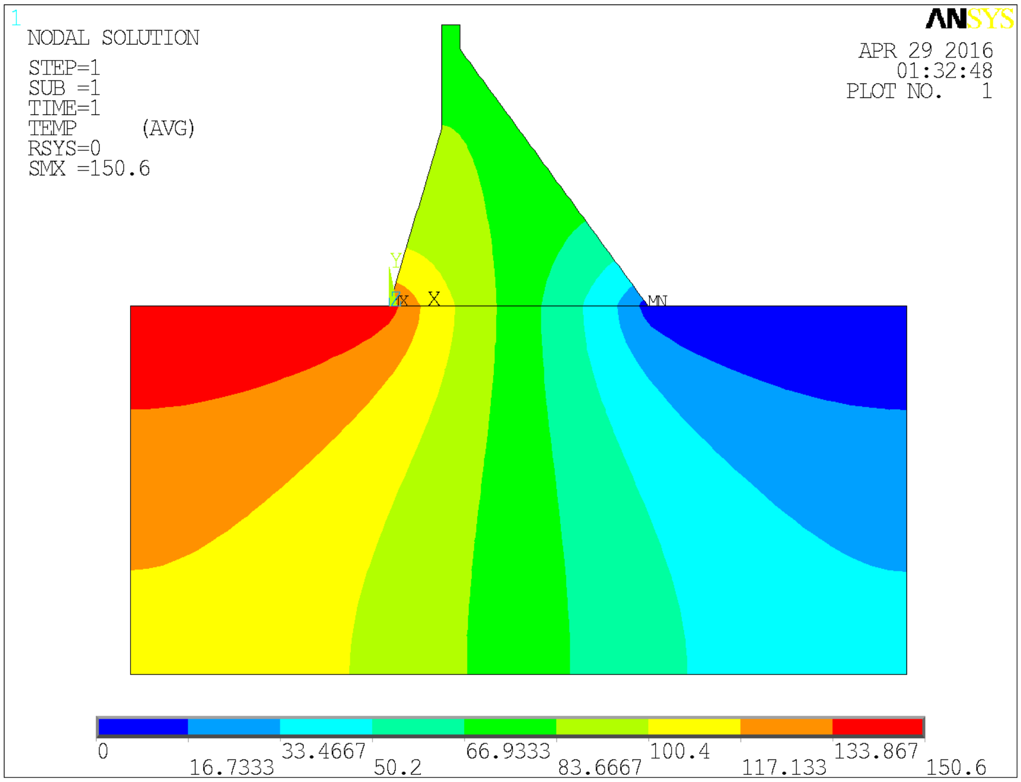


Figure 6.5 Uplift head distribution contours for profile 2

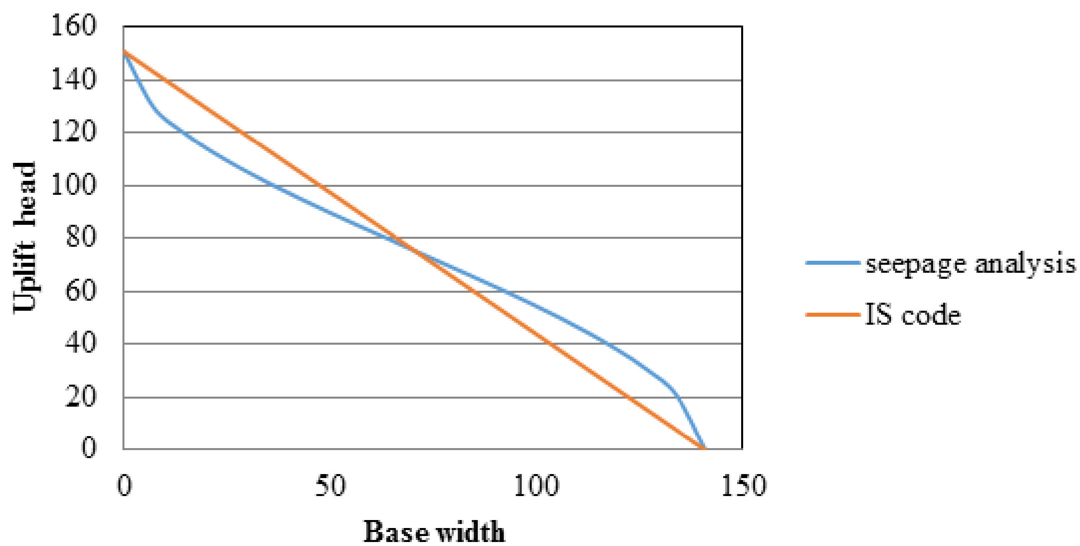


Figure 6.6 Comparison of uplift distribution at the base with IS code

6.2 STRESS ANALYSIS DESCRIPTION

The Static and Dynamic Analysis are done in two cases

a. Linear analysis

To perform analysis under the conditions of fixed base and dam foundation glued together, linear analysis can be used.

b. Non-Linear analysis

To perform analysis under the conditions of dam foundation interaction using contact elements, Non-linear analysis can be used.

6.2.1 Static Analysis

In static analysis linear as well as non-linear models are analysed for static loads. The load combinations corresponding to static analysis are

Load Combination **A** (Construction Condition):- Dam completed but no water in reservoir and no tail water.

Load Combination **F**: - Combination C, but with extreme uplift (drains inoperative).

6.2.2 Dynamic Analysis

In dynamic analysis also linear as well as non-linear models are analysed for static as well as seismic loads.

The load combinations corresponding to dynamic analysis are

Load Combination **D**: - Combination A, with earthquake.

Load Combination **G**: - Combination F, with earthquake.

In this dynamic analysis the Earthquake forces are considered in the form of Response spectrum compatible Time History generated for Zone IV, Response spectrum of IS 1893. The analysis is done with large number of sub steps. The hydrostatic and uplift pressures are applied independently and gravity is considered in each time step along with earthquake ground motion.

The earthquake excitation (Accelerogram) used in dynamic analysis is given below

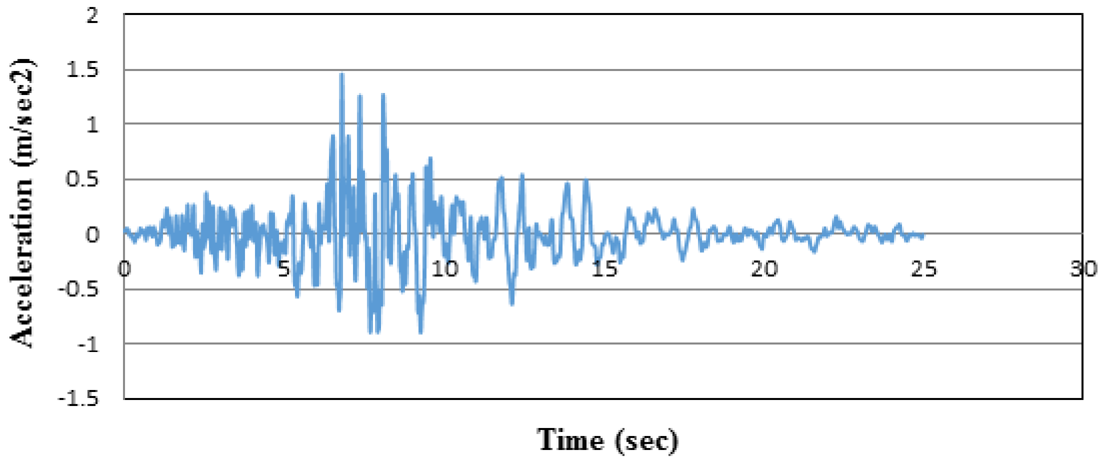


Figure 6.7 Earthquake excitation used in Dynamic Analysis

In the transient analysis the loading is considered as ramped loading. The algorithm used for analysis is Newmark's algorithm. The damping coefficients i.e. mass matrix multiplier (ALPHA) and stiffness matrix multiplier (BETA) used in analysis are given below.

Table 10 Damping coefficients for Dam profile 1

	Fixed Base	Foundation Glued	Contact interface
ALPHA	1.741	0.83611	0.8364
BETA	0.00099	0.00164	0.00164

Table 11 Damping coefficients for Dam profile 2

	Fixed Base	Foundation Glued	Contact interface
ALPHA	1.485	0.7135	0.714
BETA	0.000997	0.00153	0.00154

7 RESULTS AND DISCUSSIONS

7.1 STATIC ANALYSIS RESULTS

7.1.1 Results for Dam Profile 1

Gravity load

The results obtained from gravity analysis of three models are presented below in the form of stress contours in figures 7.3 to 7.4. From the results of the three modelling alternatives, for gravity loading the maximum stress in fixed base model is obtained near heel, which is because the resultant is falling near heel. Whereas in the other two models the maximum stress occurred at heel, because of the stress concentration. Except that stress concentration at heel, the maximum stress in remaining part of base was found to occur in dam with fixed base model. The displacements are observed to be higher at crest of dam. The crest displacement in both x and y directions is maximum for model which accounts the friction between dam and foundation.

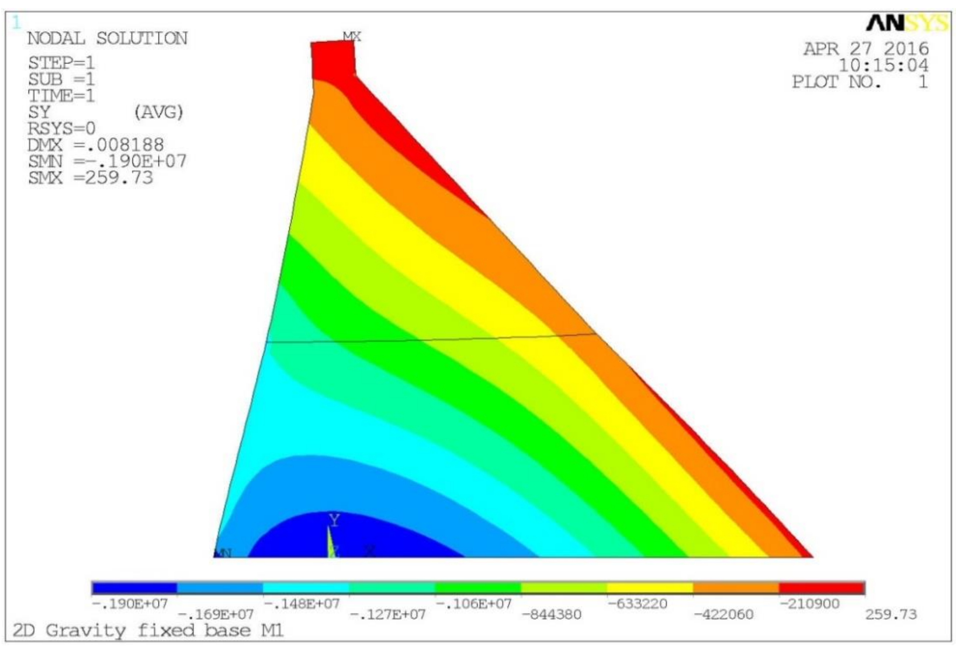


Figure 7.1 Plot of Vertical Stress (SY) in Dam profile1 with Fixed Base for Gravity Load

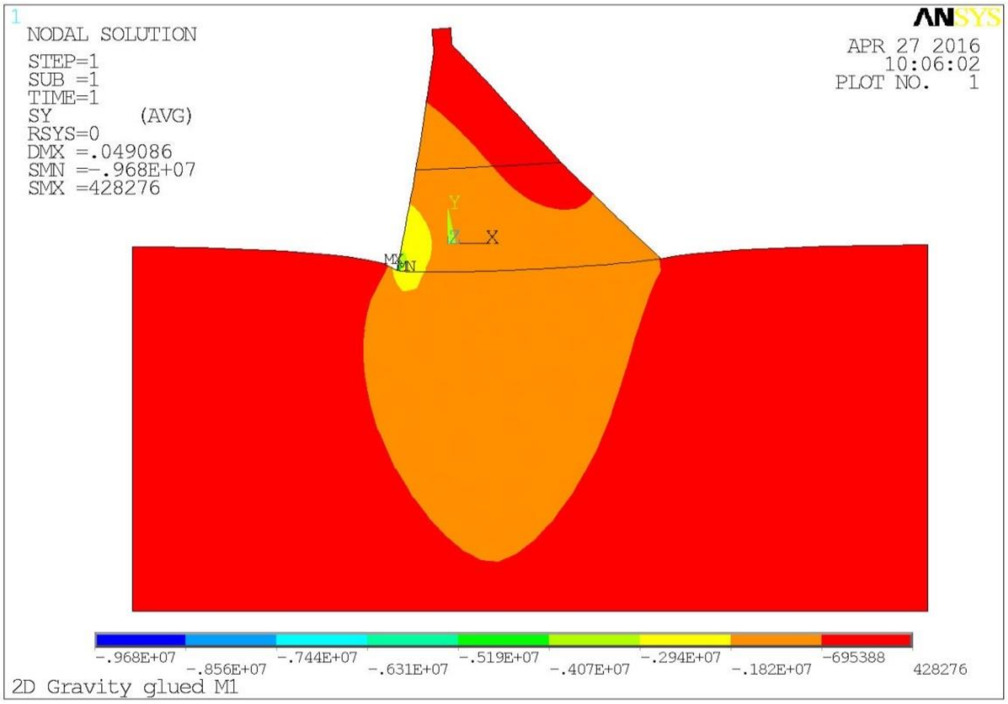


Figure 7.2 Plot of Vertical Stress (SY) in Dam profile1 with Foundation glued for Gravity Load

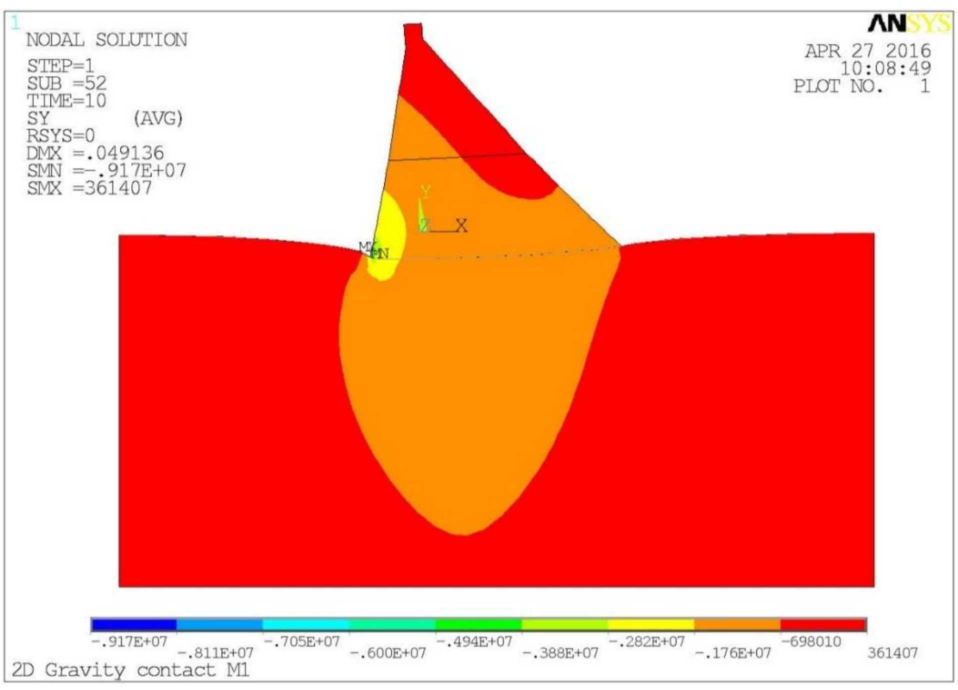


Figure 7.3 Plot of Vertical Stress (SY) in Dam profile1 with Interface element for Gravity Load

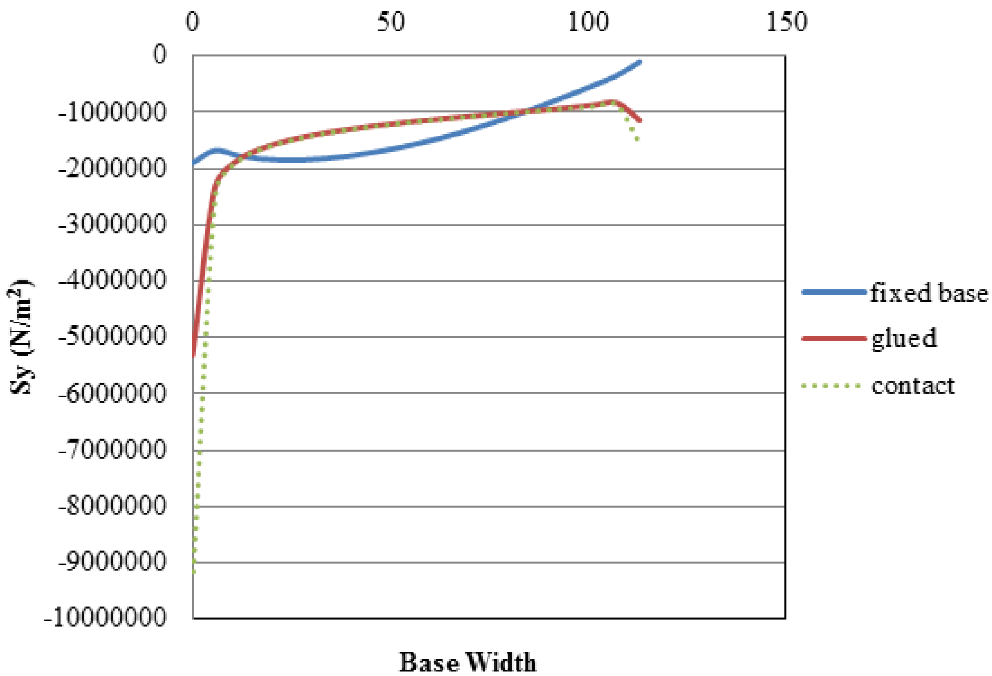


Figure 7.4 Comparison of SY at Base for three models of Dam section 1 for Gravity Load

Gravity and Hydrostatic load

The results obtained from stress analysis for gravity and hydrostatic load for three models are presented in following section in the form of stress contours in figures 7.5 to 7.8. It is observed that due to gravity and hydrostatic load for fixed base model tensile stress developed at heel. The maximum compressive stress was obtained near middle of base, which is because the shift of resultant due to addition of hydrostatic pressure from heel to middle of base. Whereas in the other two models the maximum stress occurred at heel, because of the stress concentration. Except that stress concentration at heel, the maximum stress in remaining part of base found to be developed in dam with fixed base model. Similar to gravity load case the displacements in x and y directions were higher in contact element model. The difference of displacements among all three models is observed to be higher in x direction than y direction because of small sliding induced in x direction due to hydrostatic pressure.

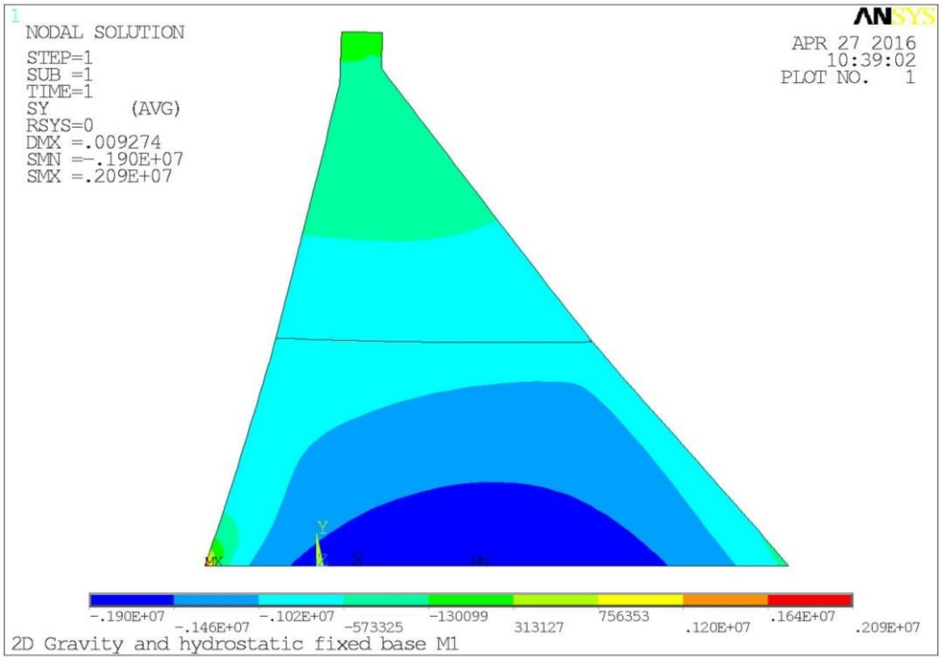


Figure 7.5 Plot of Vertical Stress (SY) in Dam profile1 with Fixed Base for Gravity and Hydrostatic Load

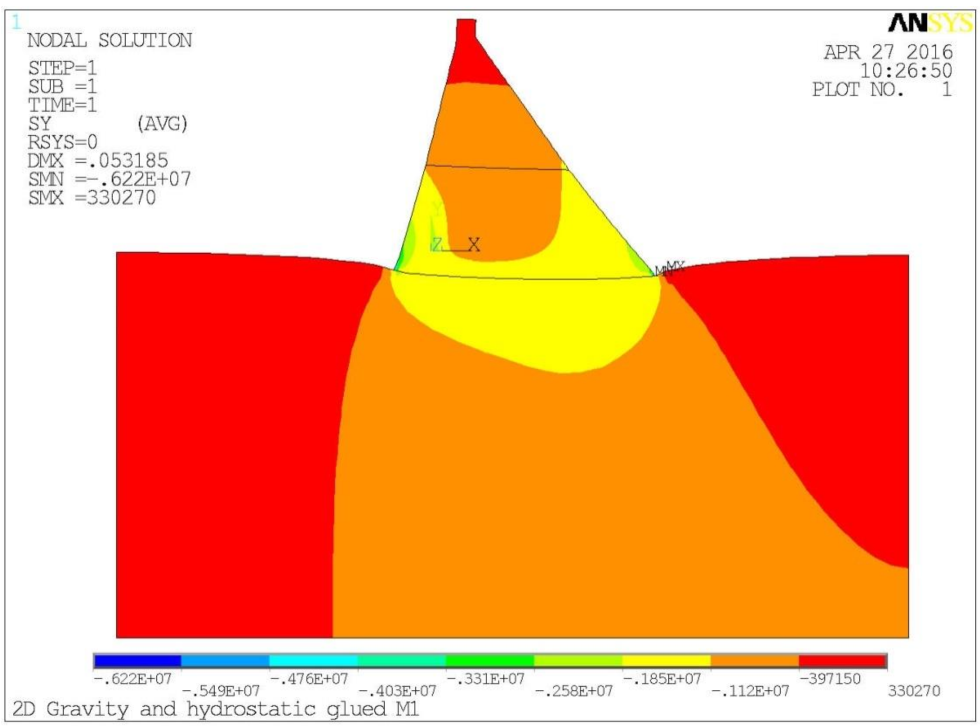


Figure 7.6 Plot of Vertical Stress (SY) in Dam profile1 with Foundation glued for Gravity and Hydrostatic Load

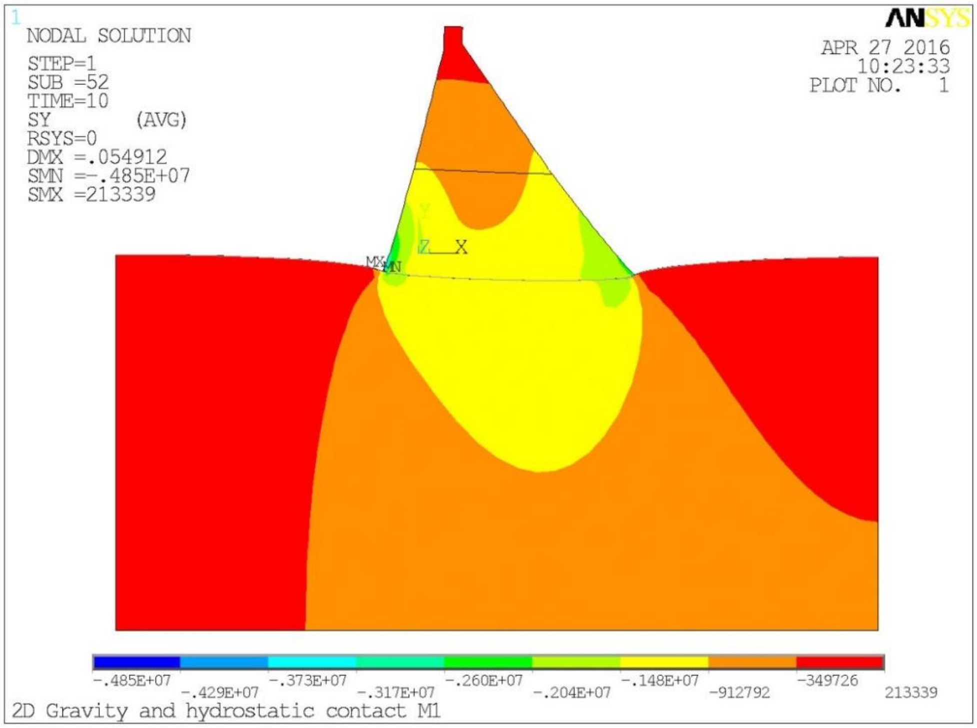


Figure 7.7 Plot of Vertical Stress (SY) in Dam profile1 with Interface element for Gravity and Hydrostatic Load

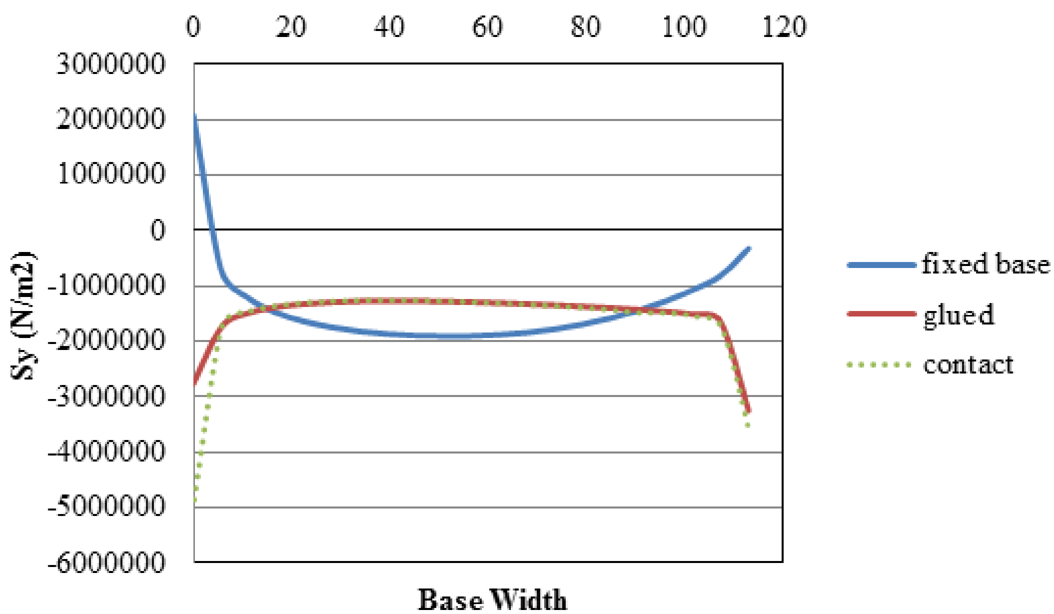


Figure 7.8 Comparison of SY at Base for three models of Dam section 1 for Gravity and Hydrostatic Load

Gravity Hydrostatic and Uplift load

The results obtained from stress analysis for gravity, hydrostatic and uplift load for three models are presented in following section in the form of stress contours in figures 7.9 to 7.12. From the results of the three modelling alternatives it is observed that due to gravity, hydrostatic and uplift load for fixed base model tensile stress is developed at heel. Due to application of uplift pressure, the tensile stress increased further compared to previous load case. The maximum compressive stress is obtained near middle of base, which is because of the shift of resultant load due to addition of hydrostatic pressure from heel to middle of base. The compressive stress decreased because of uplift action. In the other two models the maximum stress occurred at heel, because of the stress concentration. Except that stress concentration at heel, the maximum stress in remaining part of base found to be developed in dam with contact interface model. This is because the uplift pressure applied on both faces of contact elements.

Unlike the other two load cases, the displacement at crest is observed to be higher for glued model than contact model. This may be attributed to the uplift forces being applied in both the directions in case of contact model.

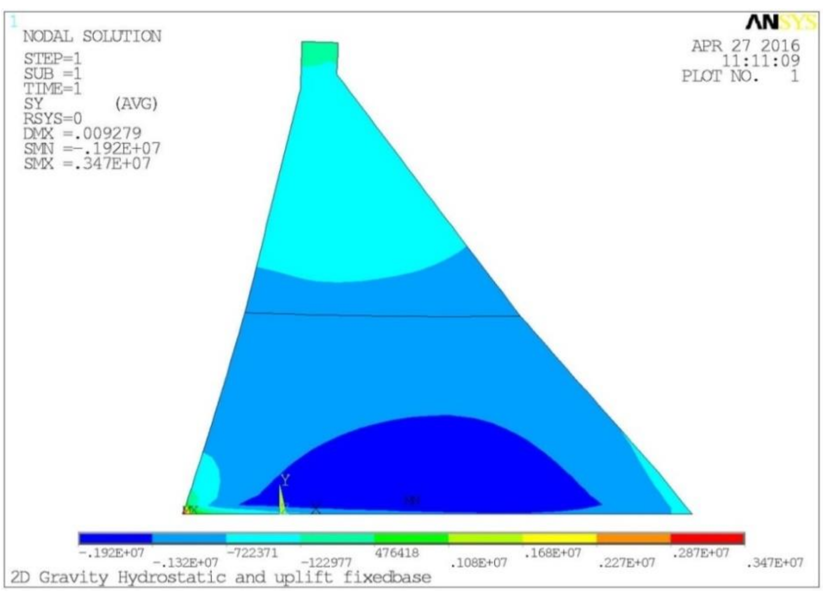


Figure 7.9 Plot of Vertical Stress (SY) in Dam profile1 with Fixed Base for Gravity, Hydrostatic and Uplift Load

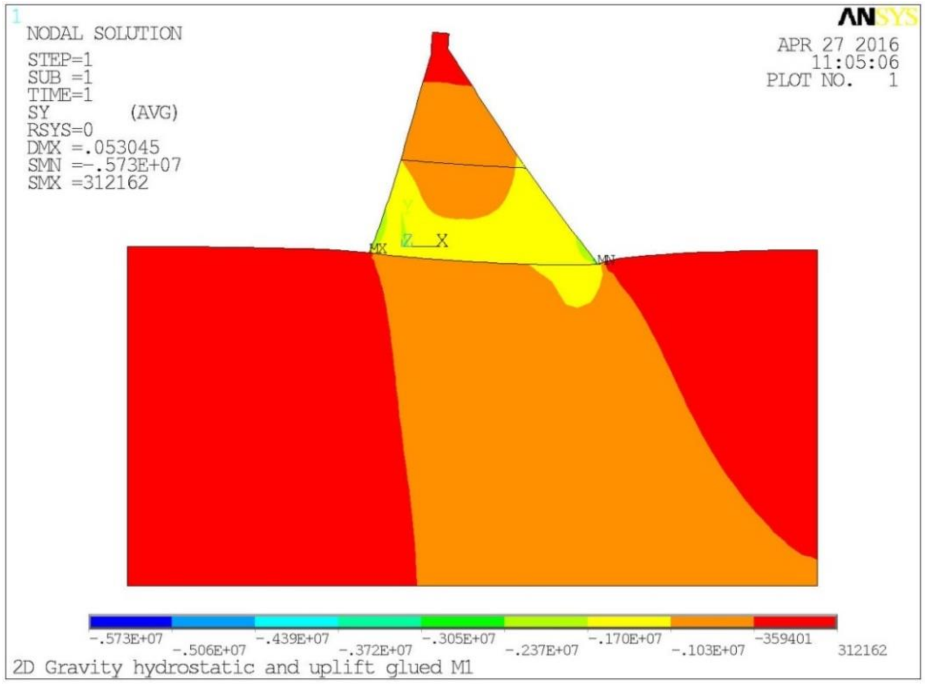


Figure 7.10 Plot of Vertical Stress (SY) in Dam profile1 with Foundation glued for Gravity, Hydrostatic and Uplift Load

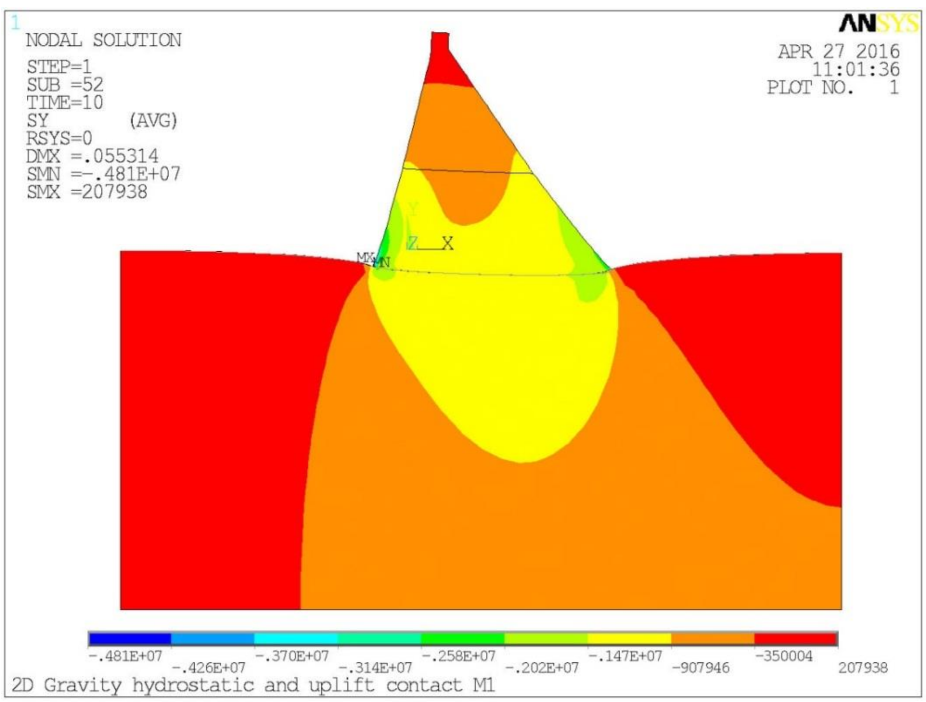


Figure 7.11 Plot of Vertical Stress (SY) in Dam profile1 with Interface element for Gravity Hydrostatic and Uplift Load

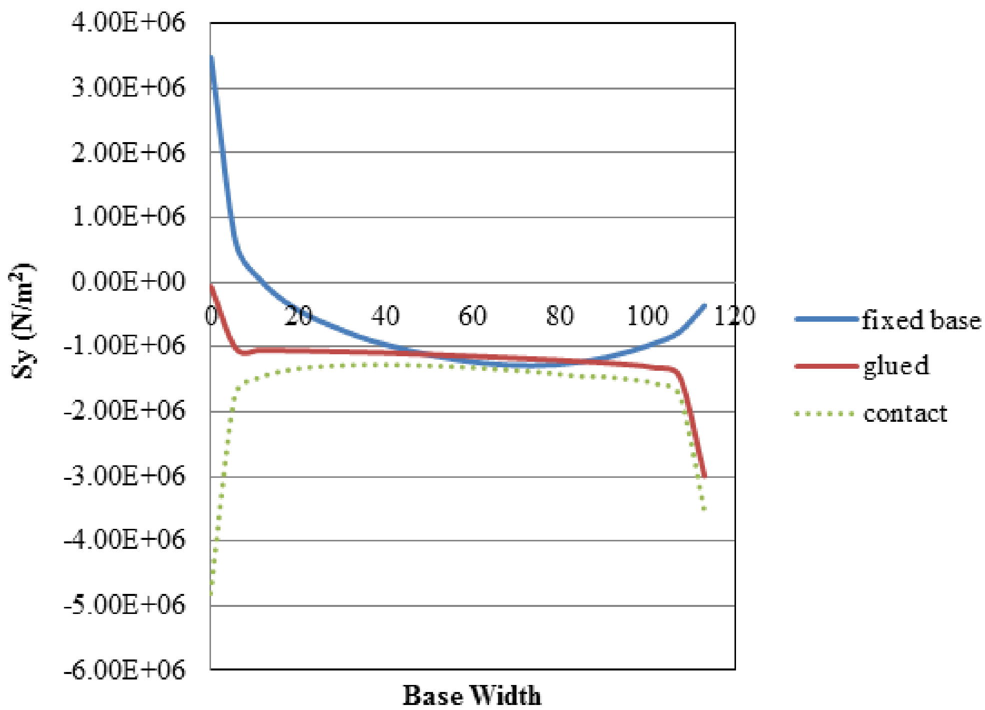


Figure 7.12 Comparison of SY at Base for three models of Dam section 1 for Gravity, Hydrostatic and Uplift Load

7.1.2 Results for dam profile 2

The following are the results obtained from stress analysis of dam profile 2

Gravity load

The results obtained from gravity analysis of three models were presented in following section in the form of stress contours as shown in figures 7.13 to 7.16. From the results of the three modelling alternatives for gravity loading the maximum stress in fixed base model was obtained near heel, which is because the resultant is falling near heel. Whereas in the other two models the maximum stress occurred at heel, because of the stress concentration. Except that stress concentration at heel, the maximum stress in remaining part of base found to be maximum in dam with fixed base model. The displacement observed to be higher at crest of dam. The crest displacement was found to be maximum in case of contact model in both x and y directions compared to other two models.

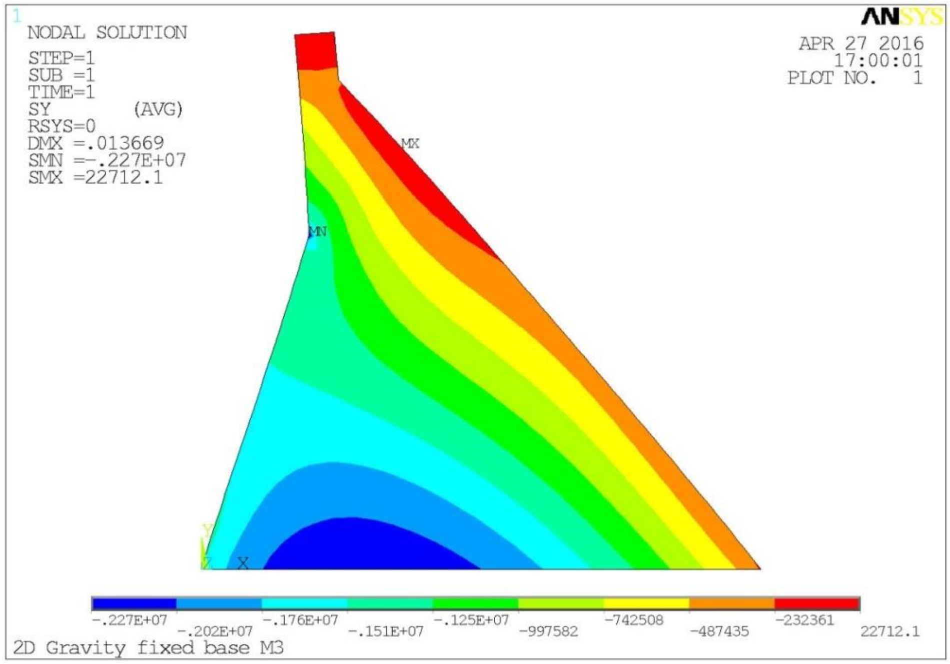


Figure 7.13 Plot of Vertical Stress (SY) in Dam profile2 with Fixed Base for Gravity Load

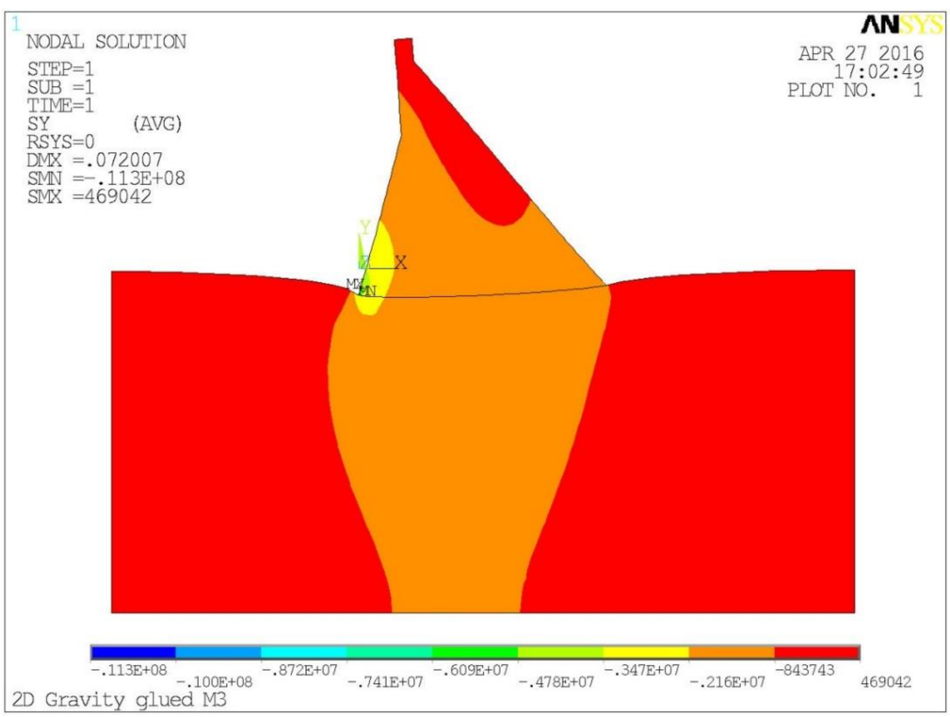


Figure 7.14 Plot of Vertical Stress (SY) in Dam profile2 with Foundation Glued for Gravity Load

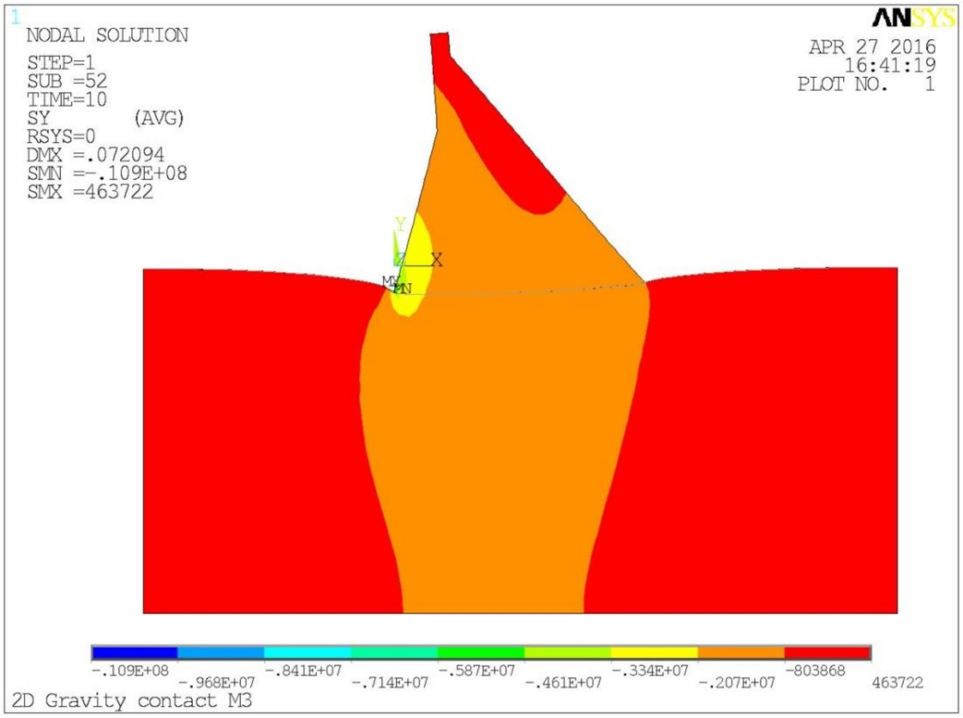


Figure 7.15 Plot of Vertical Stress (SY) in Dam profile2 with Interface Elements for Gravity Load

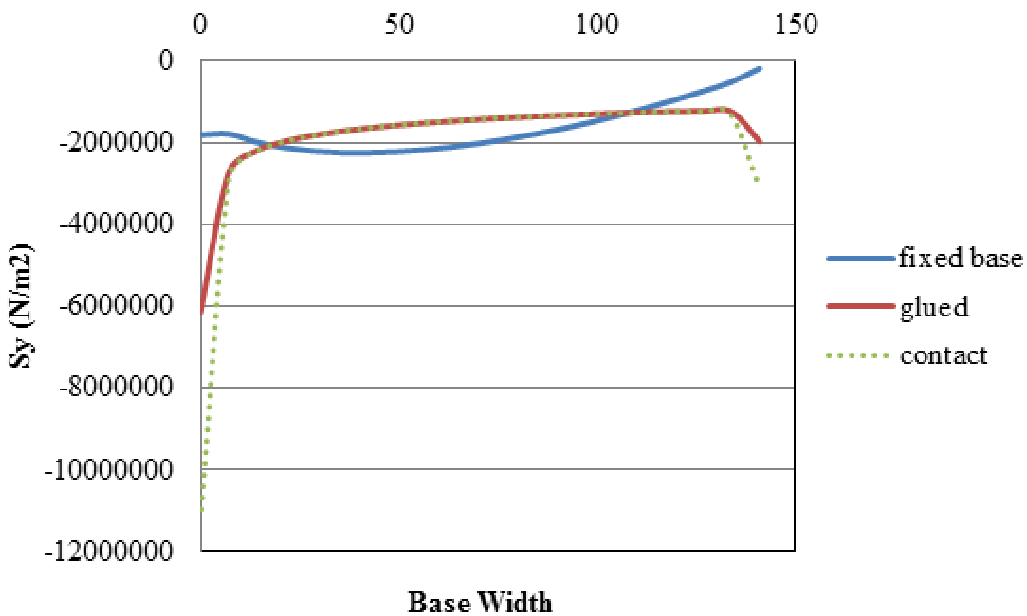


Figure 7.16 Comparison of SY at Base for three models of Dam section 2 for Gravity Load

Gravity and Hydrostatic load

The results obtained from stress analysis for gravity and hydrostatic load for three models are presented in following section in the form of stress contours as shown in figures 7.17 to 7.20. From the results of the three modelling alternatives it is observed that due to gravity and hydrostatic load for fixed base model tensile stress developed at heel. The maximum compressive stress was obtained near middle of base, which is because the shift of resultant due to addition of hydrostatic pressure from heel to middle of base. Whereas in the other two models the maximum stress occurred at heel, because of the stress concentration. Except that stress concentration at heel, the maximum stress in remaining part of base found to be developed in dam with fixed base model. Similar to gravity load case the displacements in x and y directions were higher in contact element model. The difference of displacements among all three models was observed to be higher in x direction than y direction because of small sliding induced in x direction due to hydrostatic pressure.

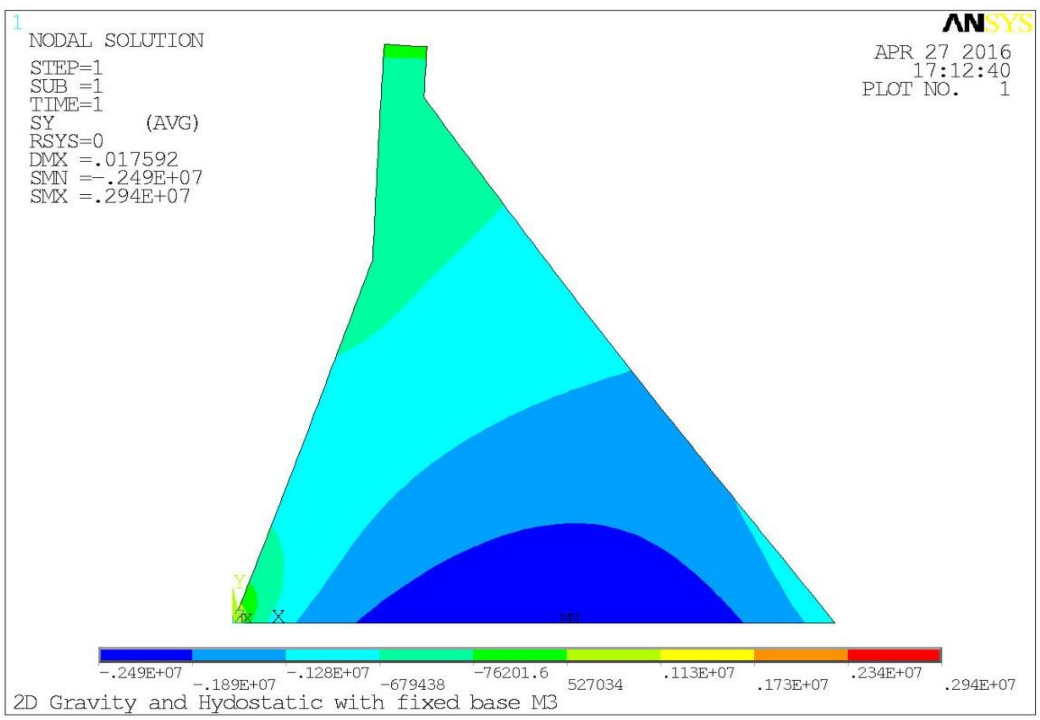


Figure 7.17 Plot of Vertical Stress (SY) in Dam profile2 with Fixed Base for Gravity and Hydrostatic Load

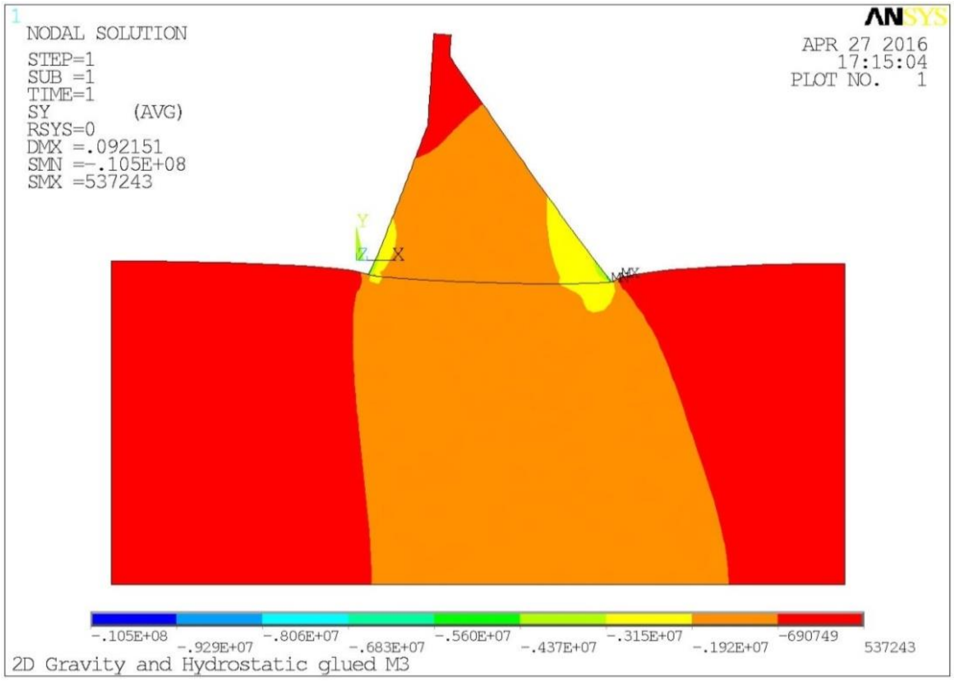


Figure 7.18 Plot of Vertical Stress (SY) in Dam profile2 with Foundation Glued for Gravity and Hydrostatic Load

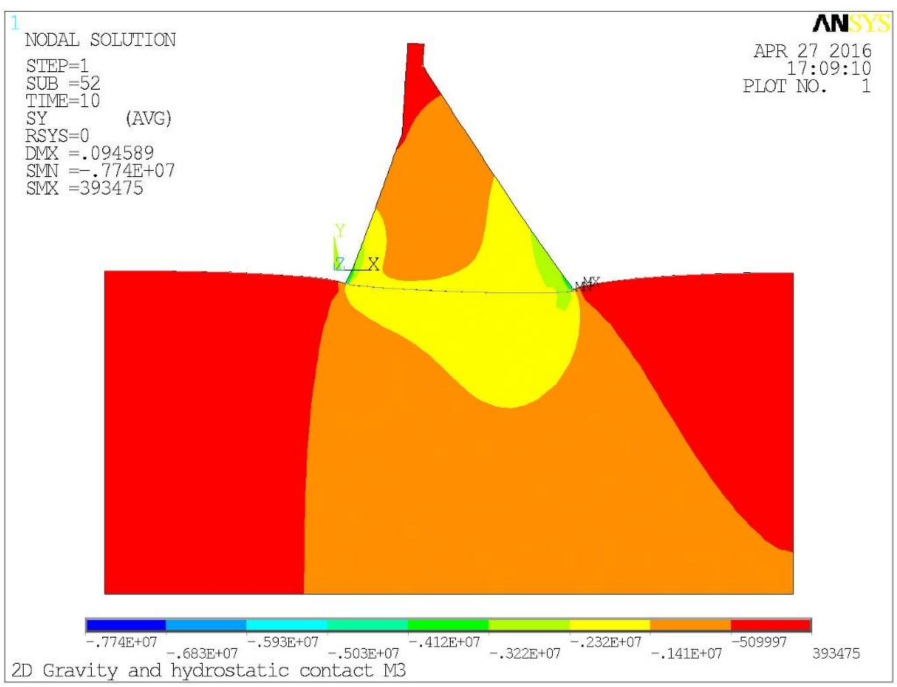


Figure 7.19 Plot of Vertical Stress (SY) in Dam profile2 with Interface Elements for Gravity and Hydrostatic Load

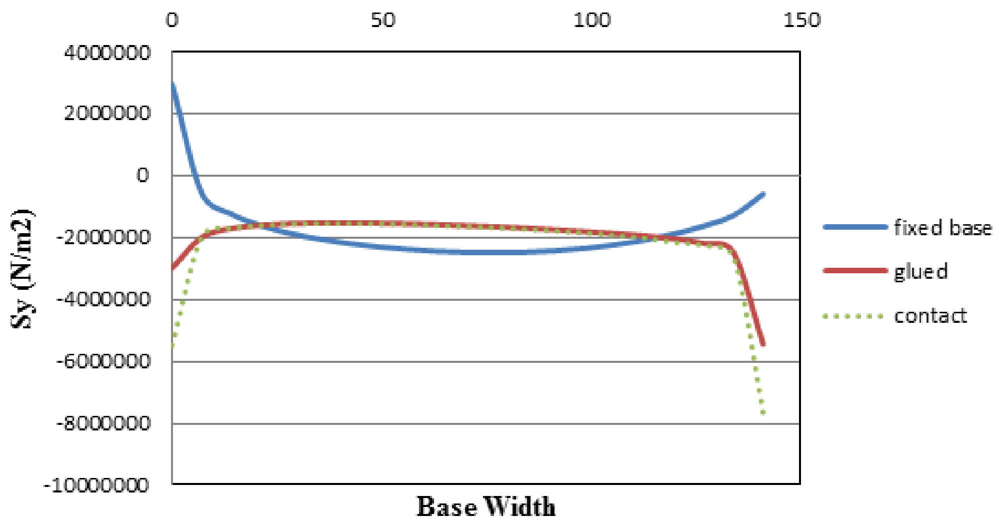


Figure 7.20 Comparison of SY at Base for three models of Dam section 2 for Gravity and Hydrostatic Load

Gravity Hydrostatic and Uplift load

The results obtained from stress analysis for gravity, hydrostatic and uplift load for three models are presented in following section in the form of stress contours as shown in figures 7.21 to 7.24. From the results of the three modelling alternatives it is observed that due to gravity, hydrostatic and uplift load for fixed base model tensile stress developed at heel. Due to application of uplift pressure the tensile stress increased further compared to previous case. The maximum compressive stress was obtained near middle of base, which is because the shift of resultant due to addition of hydrostatic pressure from heel to middle of base. The magnitude of compressive stress was decreased because of uplift action. Whereas in the other two models the maximum stress occurred at heel, because of the stress concentration. Except that stress concentration at heel, the maximum stress in remaining part of base found to be developed in dam with contact interface model. This is because the uplift pressure applied on both faces of contact elements.

Unlike the other two load cases, the displacement at crest was observed to be higher for glued model than contact model. The reason behind this was the counteracting force in case of contact model.

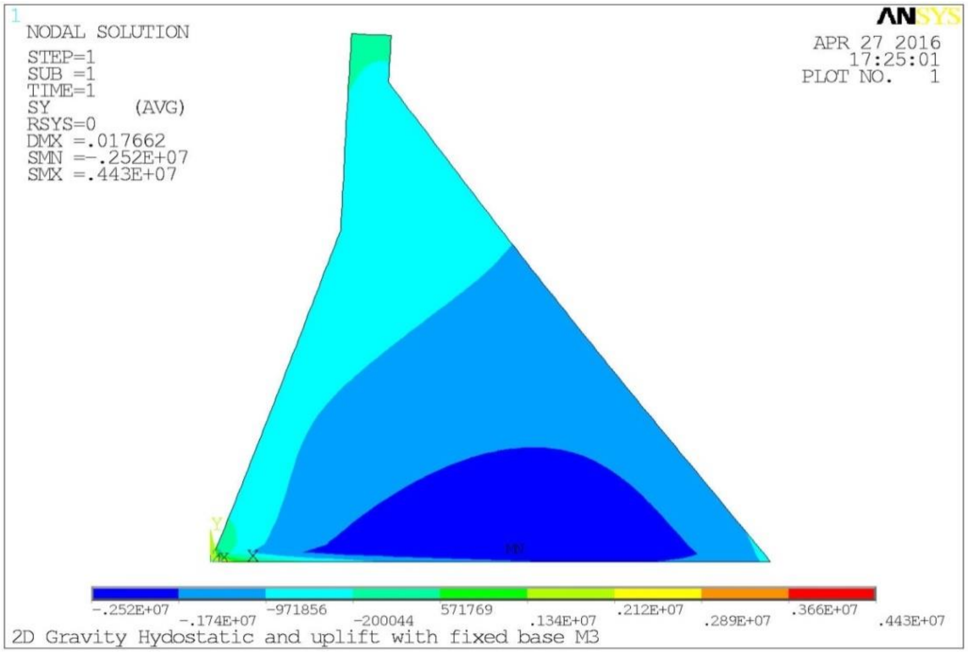


Figure 7.21 Plot of Vertical Stress (SY) in Dam profile2 with Fixed Base for Gravity, Hydrostatic and Uplift Load

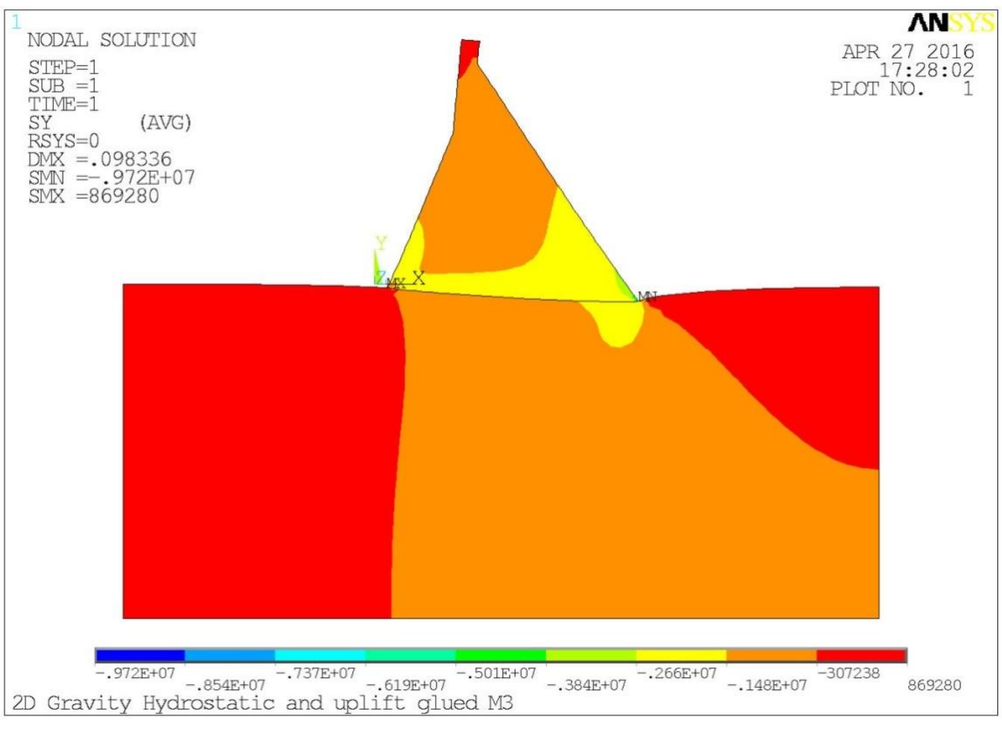


Figure 7.22 Plot of Vertical Stress (SY) in Dam profile2 with Foundation Glued for Gravity, Hydrostatic and Uplift Load

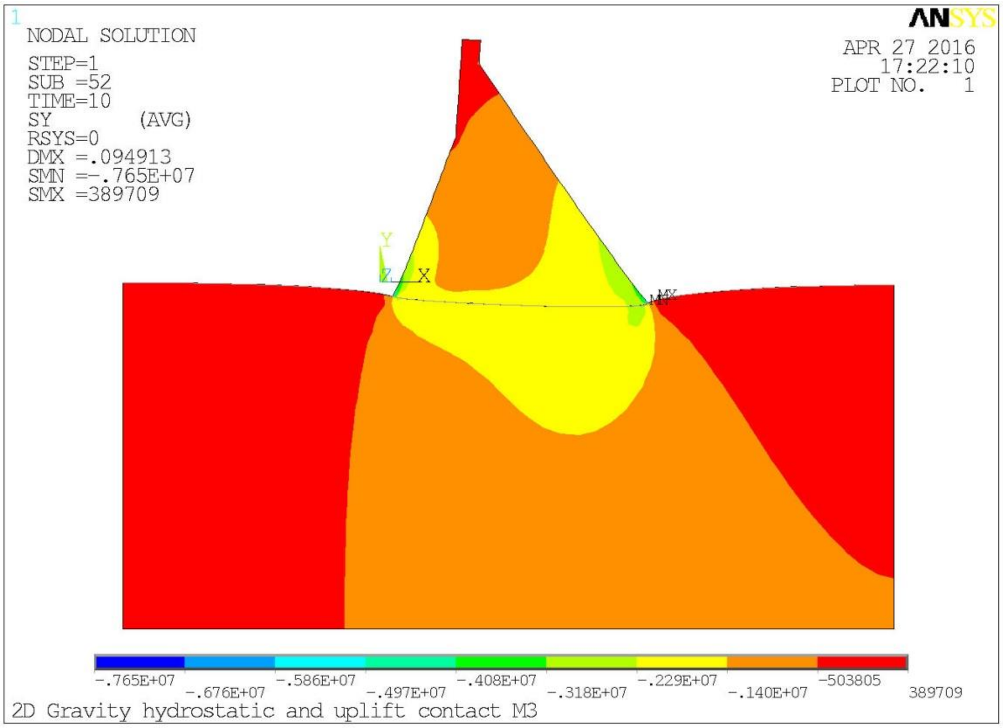


Figure 7.23 Plot of Vertical Stress (SY) in Dam profile2 with Interface Elements for Gravity, Hydrostatic and Uplift Load

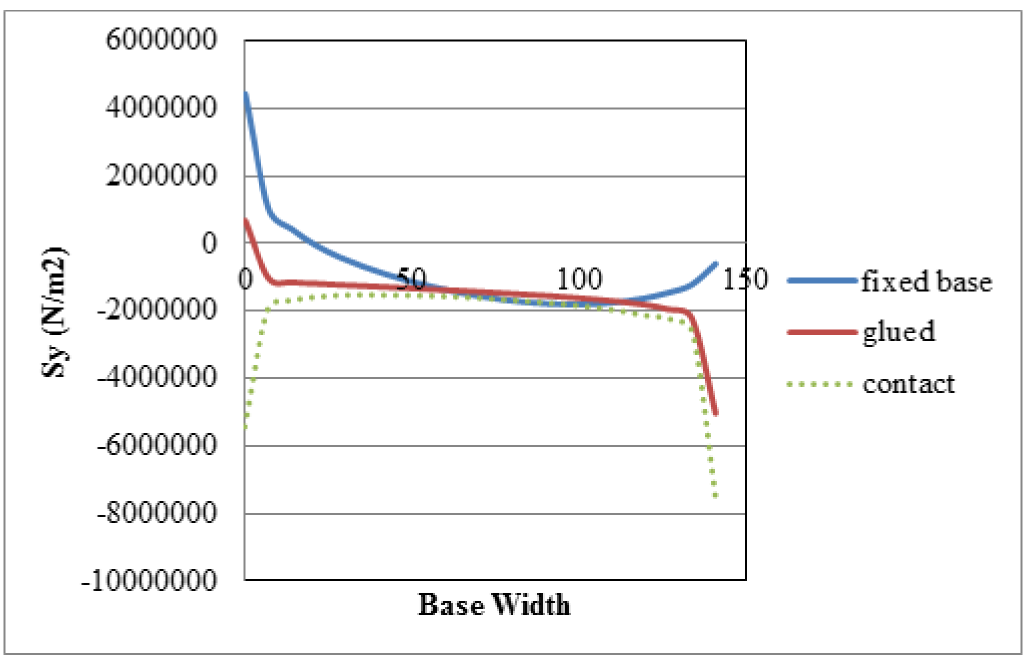


Figure 7.24 Comparison of SY at Base for three models of Dam section 2 for Gravity, Hydrostatic and Uplift Load

7.2 DYNAMIC ANALYSIS RESULTS

The dynamic analysis results are presented in terms of displacements and stresses at typical locations like crest, heel, toe and midway of base of dam profiles.

7.2.1 Results for dam profile 1

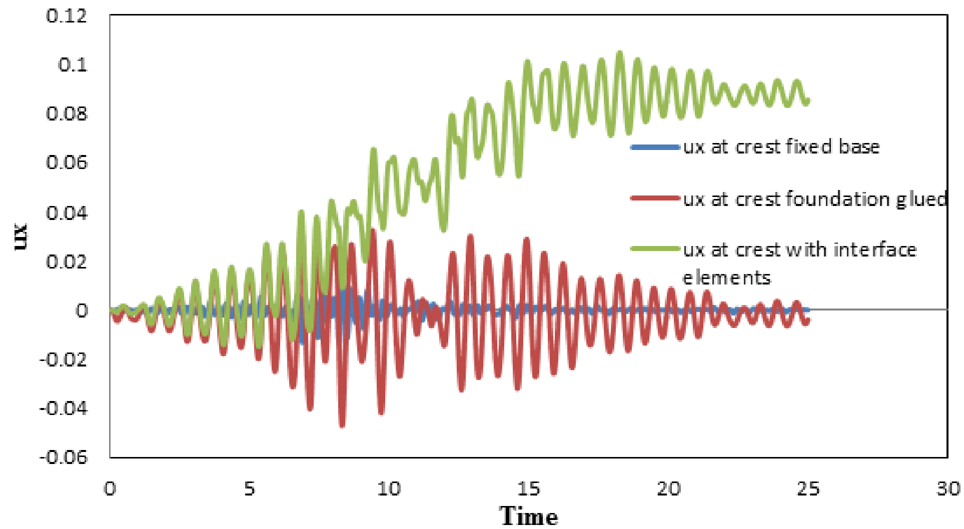


Figure 7.25 Comparison of Crest Displacement in X-direction (UX) for all foundation conditions for Dam profile 1

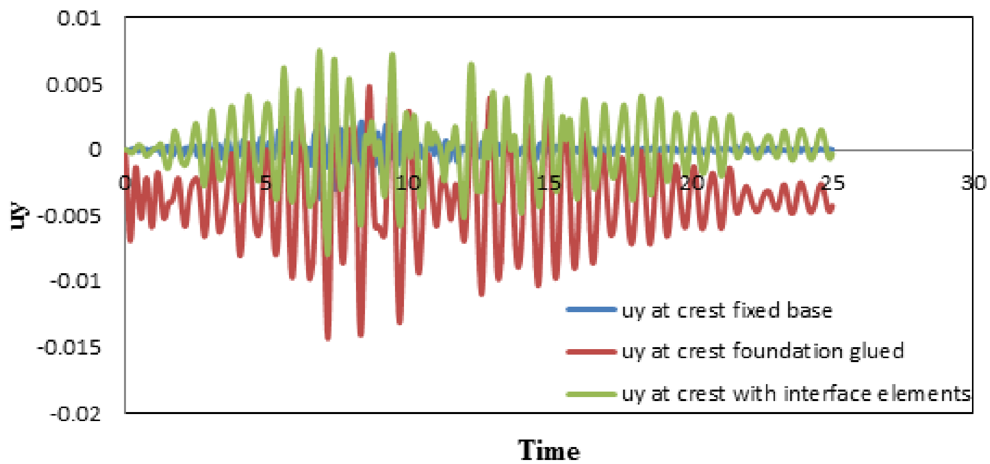


Figure 7.26 Comparison of Crest Displacement in Y-direction (UY) for all foundation conditions for Dam profile 1

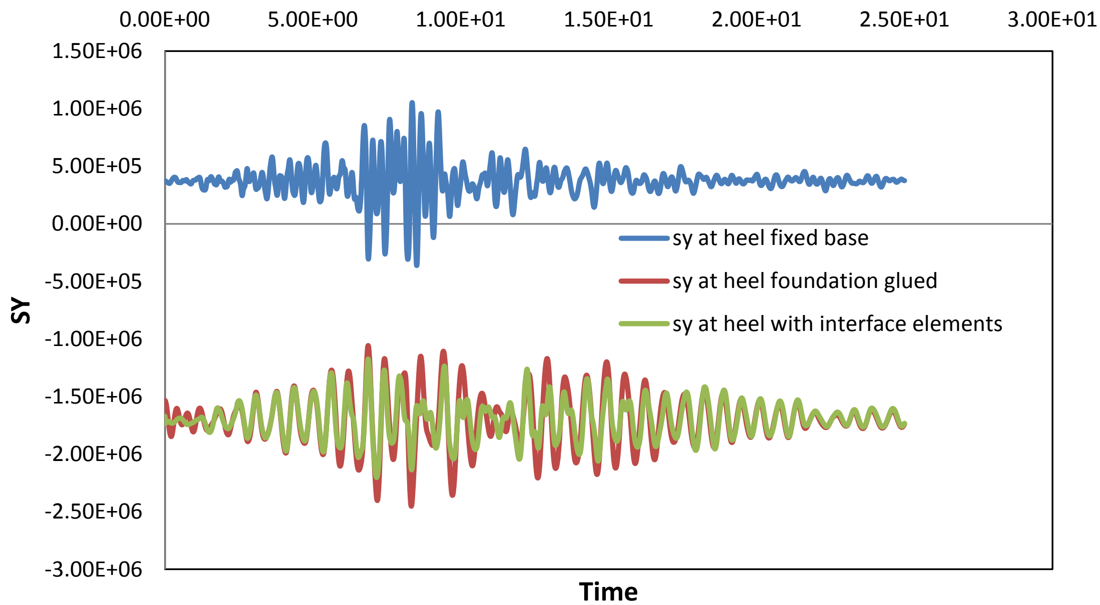


Figure 7.27 Comparison of Stress at Heel in Y-direction (SY)(N/m2) for all foundation conditions for Dam profile 1

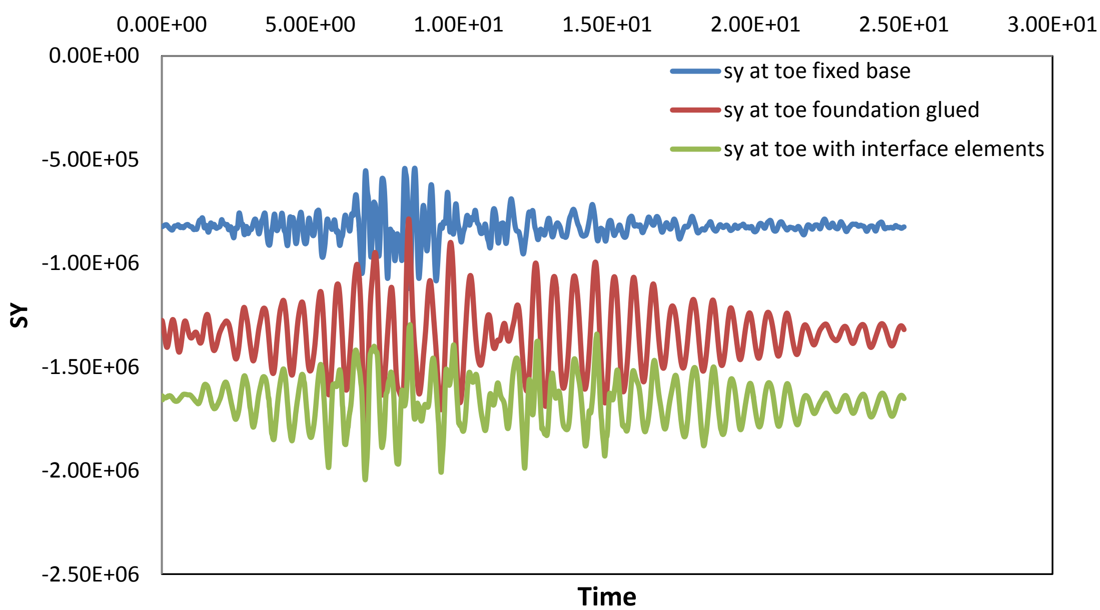


Figure 7.28 Comparison of Stress at Toe in Y-direction (SY)(N/m2) for all foundation conditions for Dam profile 1

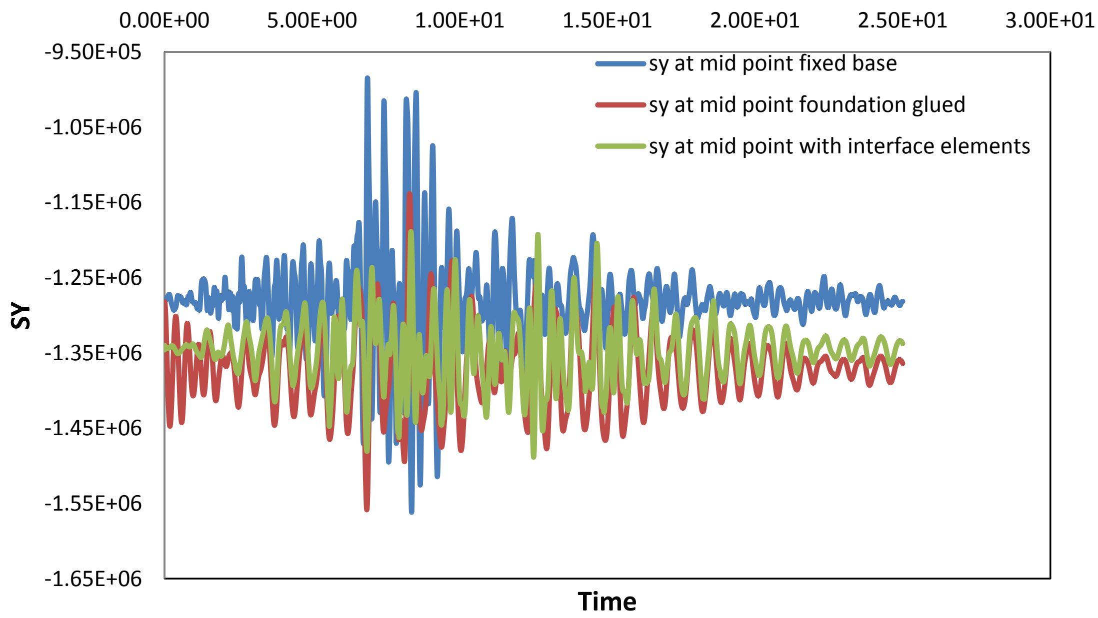


Figure 7.29 Comparison of Stress at Mid of base in Y-direction (SY)(N/m2) for all foundation conditions for Dam profile 1

The dynamic analysis results obtained for dam profile 1 were presented in the form of time versus stress and displacement plots at typical locations as shown above plots from figure7.25 to 7.29. From the plot 7.25 it is clear that for contact model permanent sliding in X direction occurred due to friction.

At heel contact interface model is subjected to more compression than other two models. For fixed base model as occurred for static loading tensile stresses are developed at heel. At the middle of the base where the resultant expected to fall the stresses are compressive for all three models and maximum for dam with contact interface model. At the toe the stresses were compressive for all three models and maximum for contact interface model. In contact interface model slip is observed at base, which resulted in higher compression at toe.

7.2.2 Results for dam profile 2

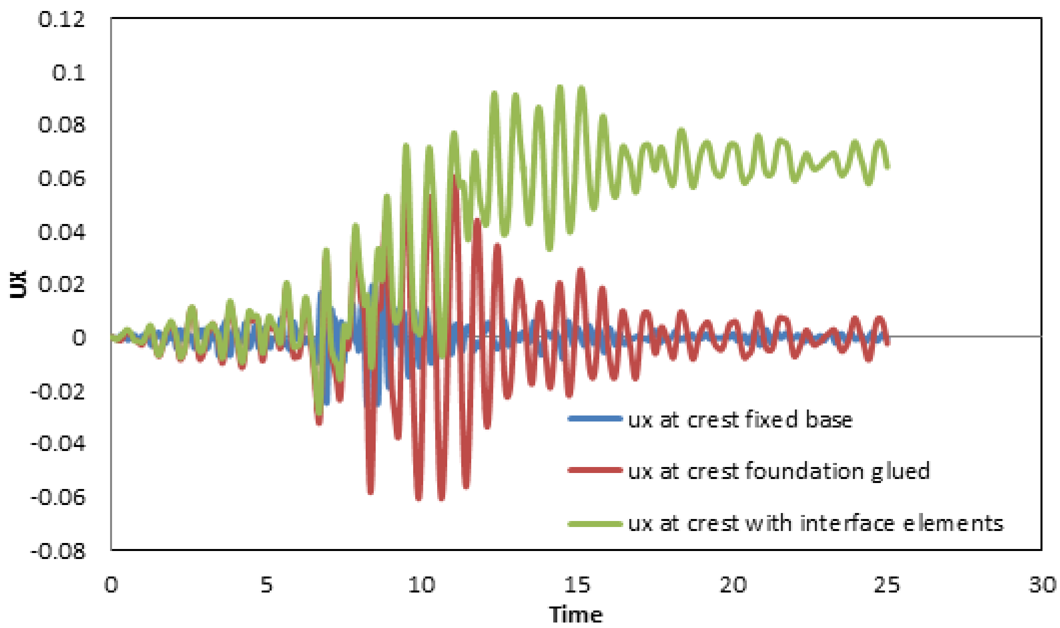


Figure 7.30 Comparison of Crest Displacement in X-direction (UX) for all foundation conditions for Dam profile 2

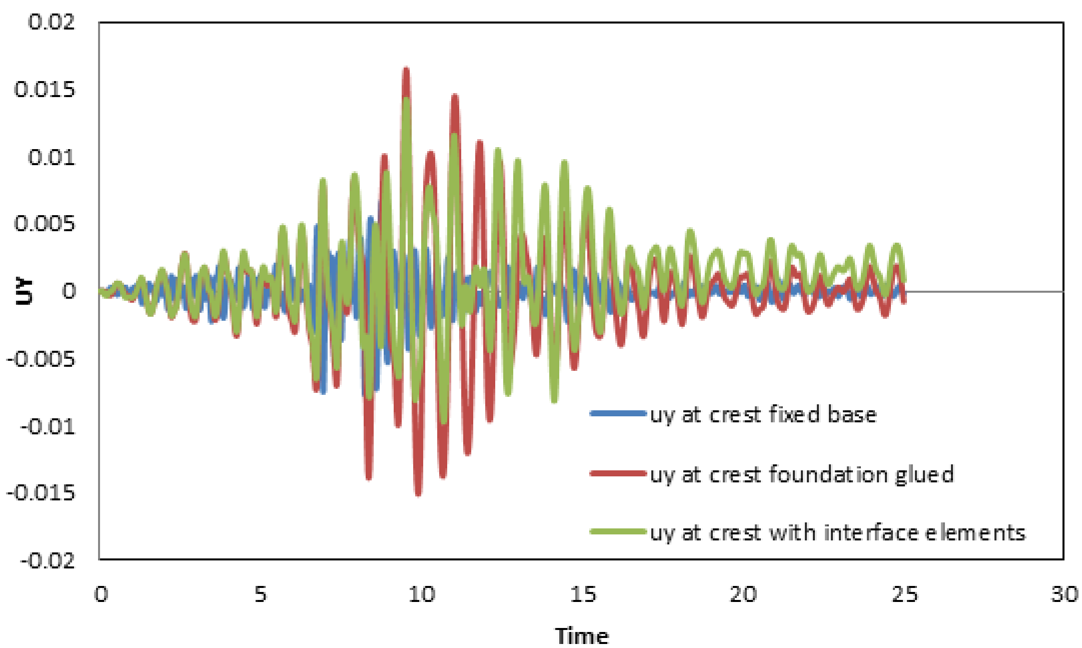


Figure 7.31 Comparison of Crest Displacement in Y-direction (UY) for all foundation conditions for Dam profile 2

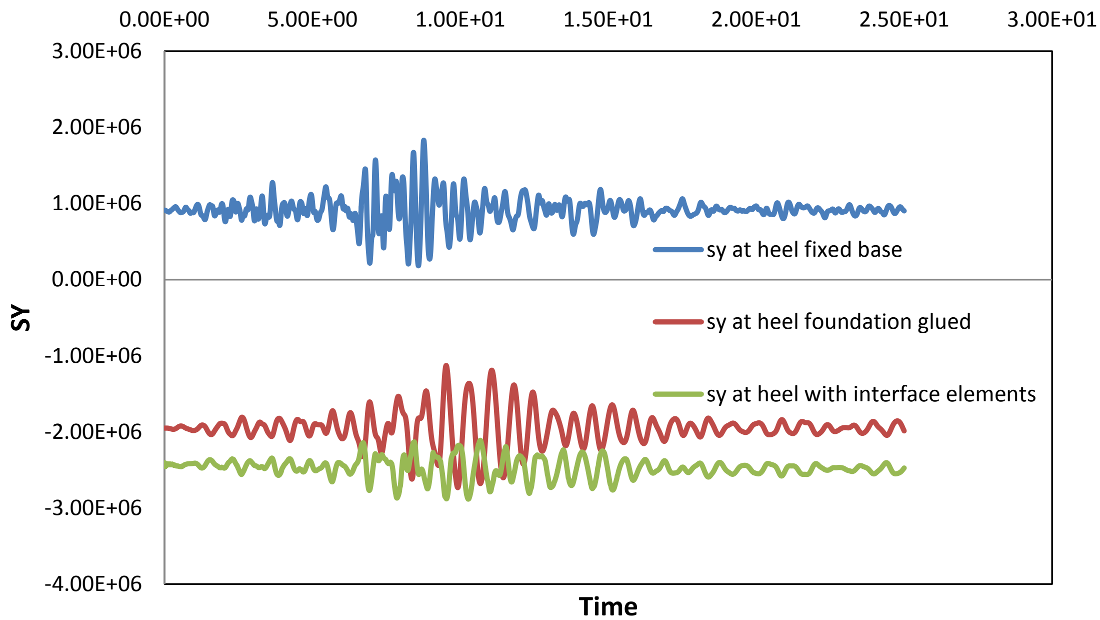


Figure 7.32 Comparison of Stress at Heel in Y-direction (SY)(N/m2) for all foundation conditions for Dam profile 2

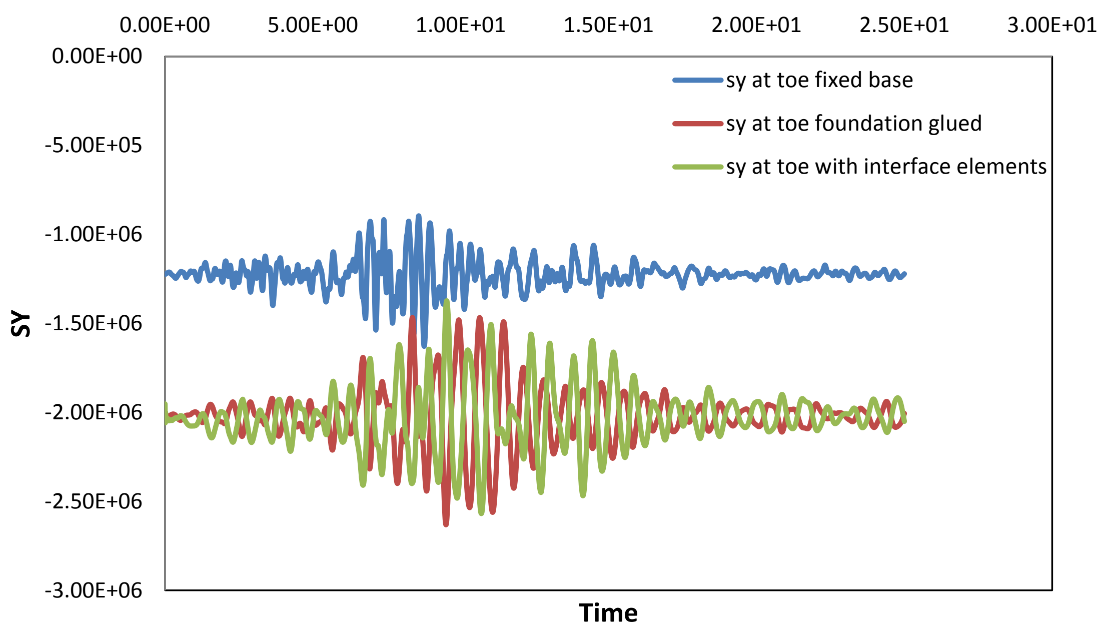


Figure 7.33 Comparison of Stress at Toe in Y-direction (SY)(N/m2) for all foundation conditions for Dam profile 2

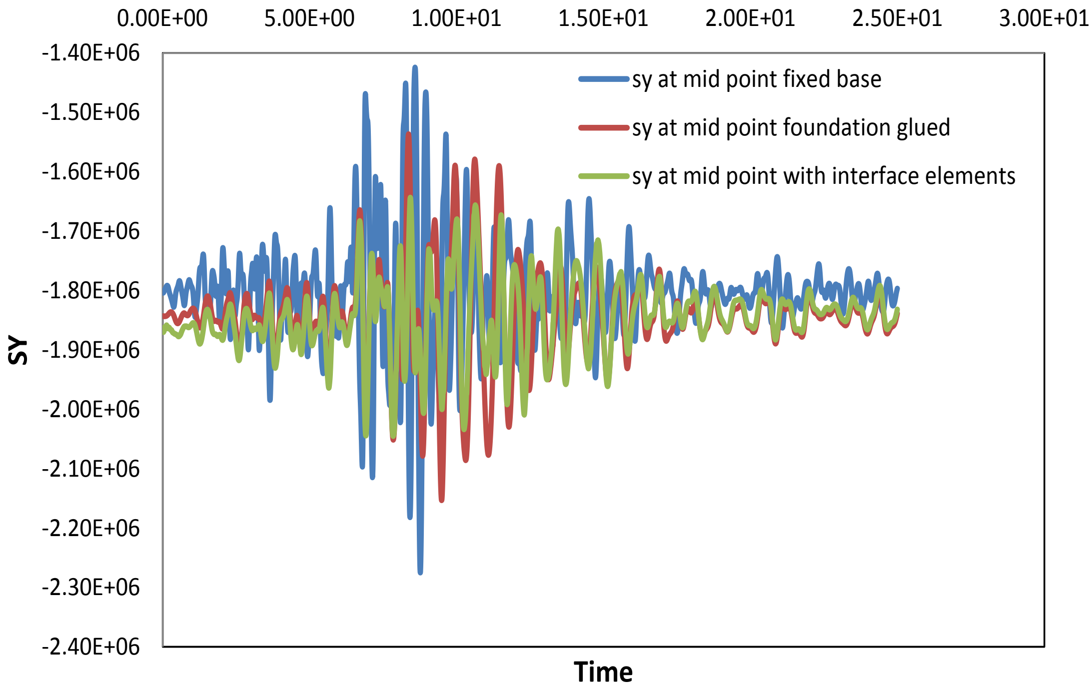


Figure 7.34 Comparison of Stress at Mid of base in Y-direction (SY)(N/m2) for all foundation conditions for Dam profile 2

The dynamic analysis results obtained for dam profile 2as profile 1 were presented in the form of time versus stress and displacement plots at typical locations as shown above plots from figure7.30 to 7.34. From the plot 7.30 it is clear that for contact model permanent sliding in X direction occurred due to friction.

At heel contact interface model is subjected to more compression than other two models. For fixed base model as occurred for static loading tensile stresses are developed at heel. At the middle of the base where the resultant expected to fall the stresses are compressive for all three models and maximum for dam with contact interface model. At the toe the stresses were compressive for all three models and maximum for contact interface model. In contact interface model slip is observed at base, which resulted in higher compression at toe.

7.3 STABILITY ANALYSIS OF THE TWO DAM PROFILES

The factor of safety calculated against sliding and overturning as per IS: 6512-1984 for all different load combinations are tabulated below in table 11:-

Table 12 Factor of safety of 2 Dam Profiles in sliding and overturning

Dam profile	Load Combination	Sliding	Overturning
Dam Profile 1	Load Combination A	---	---
	Load Combination B	1.687	1.992
	Load Combination C	1.378	1.883
	Load Combination D	3.02	3.801
	Load Combination E	1.283	1.846
	Load Combination F	1.378	1.883
	Load Combination G	1.283	1.846
Dam Profile 2	Load Combination A	---	---
	Load Combination B	1.39	1.68
	Load Combination C	1.294	1.62
	Load Combination D	4.46	4.93
	Load Combination E	1.15	1.3
	Load Combination F	1.294	1.62
	Load Combination G	1.15	1.3

8 CONCLUSIONS

8.1 CONCLUSIONS

The following conclusions are derived from the overall study

1. For dam with fixed base model the vertical stresses for gravity load are observed to be compressive and higher along the base of the dam. The stresses resulted are within the permissible compressive strength of concrete. For load combinations F and G, tensile stresses are observed to be developed at heel and those are within the permissible limits of tensile stresses suggested by A.K Chopra [10]. Although this model resulted in conservative stresses for gravity loading, for other load combinations it ended up with producing tensile stresses which are undesirable for concrete point of view.

2. For dam with foundation glued model, the vertical stresses along the base of dam resulted for gravity loading are found to be compressive and are on the lower side compared to fixed base model. Like fixed base model tensile stresses are developed at heel and are observed to be within the permissible limits of concrete [10] for load combination F. while for the dynamic loading except the stress concentrations at the edges the vertical stresses are observed to be compressive at typical locations like heel, toe and near resultant of loads. Even though tensile stresses are developed at heel for load combination F, for load combination G (which is critical) this model ended up with producing higher vertical compressive stresses and are within the permissible limits of compressive strength of concrete.

3. For dam with contact interface model, for all load combinations the vertical stresses resulted along the base of the dam are observed to be compressive. For load combination A, those stresses obtained along the base are found to be lesser than fixed base model and almost same as foundation glued model. For load combination F, the vertical stresses are observed to be higher in contact interface

model compared to other two models which because of the uplift consideration on top of foundation also. As for load combination F, for load combination G also the vertical stresses resulted are found to be higher at typical locations heel, toe and resultant of loads than the other two models. The vertical stresses resulted for all combinations for this model are found to be with in the permissible limits of compressive strength of concrete.

4. Hence the model with contact interface provides more conservative stress values for design with in permissible limits than the other two modelling alternatives. This contact interface model is also capable of showing permanent sliding if any.

5. After performing the seepage analysis for isotropic foundations it is observed that uplift head distribution at dam base is in agreement with the codal provisions. It is also observed that the uplift head pattern for such cases was independent of coefficient of permeability. However, a detailed seepage analysis should be performed for anisotropic foundations as the uplift head distribution is found to depend on ratio of permeability in both the directions.

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