Deformation And Stress Of Embankment Dam For Both Self-Weight And Water Load Effects Using Flac 3d.

Alex Alebachew  
University of Gondar

Abstract  
The deformation and stress analysis is carried out using step construction approach for both self-weight and water load effects by using a software called FLAC 3D.  
The software to be used for the modelling purpose is known as FLAC3D. The software is very powerful and can carry a wide variety of problems ranging from stress-deformation to heat transfer analysis. Mohr-Columb constitutive model is used for the analysis purpose since large deformations are expected.  
Lateral and vertical deformation and stress values are determined at center clay of core, upstream, downstream, and 53m section of the dam. The Governing deformation and stresses are assessed entirely on the dam body. For the entire dam the maximum displacement due to water load occurs around the same location as the self-weight only (i.e. 73.87m) and has a magnitude of 1.5621m which is a little bit larger than 1.5171m (due to self-weight only).  
The major aspect in the dam body in general is the reduction of the negative displacement at upstream side and the increase in the positive displacement in the downstream side due to the push imposed by the water upstream of the dam.  
The vertical displacement of the dam in all cases of this study is increased in the case of impounding reservoir load additionally considered over self-weight. Maximum vertical stress is occurred at the bottom of the dam on both sides of the filter and transition zones. Both minimum vertical and horizontal stress occurred at the crest, toe and heel of the dam but the magnitude is different.  

Key word: Embankment dam, deformation, stress, self-weight, water load

1. Introduction  
Embankment Dam is an artificial impoundment collectively constructed by available local materials to retained required amount water behind the structure. Embankment dam comprises of Soil and rocks as fill and anchored materials. The economical achievement of an embankment dam can be insured by controlling or monitoring of any type and cause of failures probably triggered in design, analysis and construction of the dam.  
Geological spatiality at the abutment of the dam, the reservoir condition, and the characteristics of the embankment materials as well as the involvement of dynamic actions are the main influencing characteristics of the dam safety. A numerical evaluation of a dam analysis is going to be proved by checking the distribution of stress and deformation contours on the dam body.  
Finite element and finite difference method methods are the basic tools to analyze the deformation and stress effect of the embankment dam. Many researchers conduct their study of deformation and stress analysis using these methods. Geo-studio, plaxis, FLAC-3D and others are some of the software tools used to analyze the deformation and stress in embankment dams.  
At the commencement and due process of dam construction there is an expected movement of dam body with an applied load induced by staged fill materials. At the end of the construction, the reservoir gradually filled to the dam consequently deformation and stress change will be considerable regardless of the magnitude.  
(Salmasi, 2014) Selected Alavian earth dam for stress-strain analysis using Geo-Studio software. The settlements from the single layer embankment simulations were compared with the settlements calculated for 3, 7, 10 and 15 soil layers considered in construction processes. Results showed that maximum displacement in single layer dam has happened in the crest of the dam. Increase in embankment layers, resulted in the maximum displacement creation in the middle of the
downstream shell. The simulation layers for the construction have little effect on the stresses in the dam; however, that may cause a significant effect on the deformations of the dam.

(Robert J. Huzjak) presents the analyses, field and laboratory data, and design features that were incorporated to accommodate bedrock settlement below a 200-foot-high embankment dam that is being constructed in two phases on a clay stone bedrock foundation. Construction and post construction settlement were computed using one-dimensional hand and two-dimensional finite-element methods. Data obtained from settlement sensors, piezometers, and outlet conduit surveys during and after the first phase of construction were used to calibrate laboratory-obtained settlement properties to observed behaviour. These calibrated properties, and the time since completion of the first construction phase, were used to predict the ultimate settlement for the foundation of the completed embankment. Features incorporated into the embankment and the ancillary facilities to accommodate the predicted construction and post construction bedrock settlement will be described.

(Kharagpur, 2010) Estimate permanent displacements of Tehri dam due to an earthquake of magnitude Mw = 8.5, the occurrence of which has a high probability in the region, and for an earthquake of magnitude Mw = 7-0, for which the dam has been currently designed. A two-dimensional finite element analysis and five different semi-empirical and empirical methods, like, Seed and Makdisi’s method, Newmark’s double integration method, Jansen’s method, Swaisgood’s method and Bureau’s method have been utilized to study the probable dynamic behaviour of the dam and their results are compared to get a range of values within which, the permanent displacement of the dam, is estimated to lie. The present study shows that the predicted displacements due to an earthquake of magnitude Mw = 7-0 are significant but not enough to compromise the safety of the dam. However, the displacements predicted for an earthquake of magnitude Mw = 8.5 are quite high and might cause rupture of filter zones.

(Robertshaw, 1994) Studied stress changes associated with fluctuations in reservoir water level under normal operating conditions. The mechanisms causing these deformations are examined with emphasis on the effect of the position of the watertight element. Simple analyses are presented to illustrate the difference in behaviour of the upstream fill of a dam with a central watertight element and one with an upstream watertight element. Field observations of embankment deformations during drawdown are described and some results from a programme of laboratory testing to determine the one dimensional compression behaviour of the upstream fill are presented. When values of constrained modulus from the laboratory tests were used in the simple analysis for a dam with a central watertight element, the calculated surface settlements of the upstream slope compared well with field measurements.

(Seyed Morteza Marandi, 2012) In the present study a Genetic Programming model (GP) proposed for the prediction of relative crest settlement of concrete faced rock fill dams. To this end information of 30 large dams constructed in seven countries across the world is gathered with their reported settlements. The results showed that the GP model is able to estimate the dam settlement properly based on four properties, void ratio of dam’s body (e), height (H), vertical deformation modulus (E) and shape factor (Sc) of the dam. For verification of the model applicability, obtained results compared with other research methods such as Clements’ formula and the finite element model. The comparison showed that in all cases the GP modelled to be more accurate than those of performed in literature. Also a proper compatibility between the GP model and the finite element model was perceived. (S.R.TAFTI, 2008)In this paper, the results of dynamic analyses performed on Karkheh embankment dam in Iran, incorporating different core materials, are presented. Appropriate models for the core materials are utilized, based on the laboratory test results. The Newmark method is used to evaluate the permanent displacement. It is shown that seismic pore pressure build-up in composite clays can significantly increase the Newmark permanent displacement.

(Germanov, 2000) Some results of the computations for determining the pore water pressure distribution and settlement during construction and operation of a high dam and ultimate stress-strain behaviour of the dam sub-grade are analysed. The computations are performed applying computer codes, developed by the author, using the solutions of the two-dimensional consolidation of multi-
phase clayey soils taking into account the non-linear strain characteristics of the clay, deformability of the pour liquid (due to the incomplete saturation) and the creeping of the soil skeleton.

2. Materials and methods
2.1. Description of the Dam

The proposed dam has a height 134.87m, bottom width 470.79m, top width 10.41m average upstream shoulder slope 1:2 and average downstream shoulder slope 1:1.8. The dam has different zonings or is comprised of seven material types. The dam is to be classified as zoned for the sole reason it is composed of different zonings.

The upstream reservoir has a capacity to retain 134.03m3 deep water up to the maximum peak anticipated level. The bottom of the dam is assumed to be impervious and strong against shear failure. The internal zoning has been provided with a 0.44:1 slope to the upstream direction to control seepage and minimize deformation characteristics of the dam.

The dam is not provided with cut off since the foundation is found to be impervious. Thus, there is no susceptibility of the dam for uplift. The downstream slope of the dam has high staggering shape as compared to the upstream slope. The dam is also provided with a small coffer dam which is useful during the construction period of the dam. The downstream side of the coffer dam can be maintained dry as far as the construction proceeds up to the top level of the coffer dam. Figure 1.1 showed the dam section given for further analysis purpose.

![Fig 2.1 Dam Geometry and Zoning Materials](image)

2.2. Modelling of the dam

The unprecedented computing power now available has resulted in advanced software products for engineering and scientific analysis. The ready availability and ease-of-use of these products makes it possible to use powerful techniques such as a finite element and difference analysis in engineering practice.

Software tools such as FLAC3D do not inherently lead to good results. While the software is an extremely powerful calculator, obtaining useful and meaningful results, from this useful tool depends on the guidance provided by the user. It is the users’ understanding of the input and their ability to interpret the results that make it such a powerful tool. Hence, proper modelling of the problem at hand is necessary.

The modelling is done by the user while the analysis is carried by the software. Here, in this case FLAC3D is used mainly for the entire analysis but occasionally; through these numerical modelling the main target is to predict quantitatively the results of a certain engineering problems.
Modelling start with the simplification of the actual physical condition to a geometry that is easy to handle and of course obtain good results. The simplification should be carefully studied so as not to get misleading results which ultimately lead to failure of a given structure. The geometry of the dam is simplified as outlines in the following bullets:

- The upstream slope is maintained constant with a slope of 2H:1V
- The downstream slope is maintained constant with the indicted average slope 1.8H:1V
- The cofferdam to be constructed at the beginning of the construction of the dam is included to the upstream portion of the dam since it will serve as component of the main dam during service period of the dam and majority of the construction material is of type Zone3B (same material as to that of the upstream shoulder).
- The majority of bottom of the dam is considered to be flat since no cut-off is provided. Figure 2.2 shows the simplified model geometry of the dam body

![Figure 2.2 Simplified Geometry of the dam body](image)

### 2.2.1. Modelling Software and discretization of the Dam Domain

The software to be used for the modelling purpose is known as FLAC3D. Short hand representation for Fast Lagrangian Analysis for Continua in 3 Dimensions. The software is based on the advanced computation method called finite difference method and has great importance in the area of geotechnical engineering problems because much of geotechnical problems are related to continues medium like soil.

The software is very powerful and can carry a wide variety of problems ranging from stress-deformation to heat transfer analysis. In some circumstances GeoStudio 2007 is used, and it is powerful 2D analysis software. This one is based on finite element method of analysis.

Discritization of the domain into finite elements is the first step in the finite difference method. This is equivalent to replacing the domain having an infinite number of degrees of freedom by a system having finite number of degrees of freedom. The shape, size, number and configuration of elements have to be chosen carefully so that the original body or domain is simulated as closely as possible without increasing the computational effort needed for the solution. Among the various considerations taken in the Discritization process are the followings:

- Types of elements:
- Size of elements
- Location of nodes

Considering the above points the is domain Discretiszed out in such a way that calculation time will be minimized and accuracy is maximized. There are different types and sizes of zones used in this model. Unit width is considered in the discritization of the dam in y-direction. Hence, 2D analysis will be carried out in all the upcoming analysis. Figure 2.3 illustrates the discredited domain of the dam.
2.2.2. Constitutive Model and Material Properties

Mohr model is used for the analysis and material behaviours required for this model and FLAC3D are extracted. Table 2.1 showed the summary of material behaviours considered.

<table>
<thead>
<tr>
<th>Material Description</th>
<th>G (Mpa)</th>
<th>K (Mpa)</th>
<th>Unit weights (γ)</th>
<th>c’</th>
<th>Φ’</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>dry</td>
<td>Wet</td>
<td>Sat</td>
</tr>
<tr>
<td>Impervious core - Zone1</td>
<td>14.81</td>
<td>44.44</td>
<td>21.50</td>
<td>22.00</td>
<td>23.40</td>
</tr>
<tr>
<td>Fine Filter - Zone2A</td>
<td>26.69</td>
<td>69.61</td>
<td>18.00</td>
<td>19.50</td>
<td>21.90</td>
</tr>
<tr>
<td>Coarse Filter - Zone2B</td>
<td>27.31</td>
<td>59.17</td>
<td>19.50</td>
<td>20.50</td>
<td>22.20</td>
</tr>
<tr>
<td>Transition - Zone3A</td>
<td>28.40</td>
<td>47.33</td>
<td>21.00</td>
<td>22.00</td>
<td>23.20</td>
</tr>
<tr>
<td>Upstream rock fill - zone3B</td>
<td>28.40</td>
<td>47.33</td>
<td>21.50</td>
<td>22.50</td>
<td>23.50</td>
</tr>
<tr>
<td>Downstream rock fill - zone3C</td>
<td>28.40</td>
<td>47.33</td>
<td>21.00</td>
<td>22.00</td>
<td>23.20</td>
</tr>
</tbody>
</table>

Table 2.1 Summary of the Material Behaviours used in FLAC3D

G is the shear modulus of the materials and K is the bulk modulus of the materials obtained by using E (young’s modulus) and v (poison’s ratio). The formulas used are shown below:

\[ G = E(1+\mu) \]

\[ K = \frac{E(1-2\mu)}{2(1+\mu)} \]

2.2.3. Boundary Conditions

The bottom of the foundation is assumed to be fixed both in the vertical and horizontal direction. Hence both xdisp and zdisp are set zero for all grid points located on the bottom of the dam. The deformation sideways in the direction of the valley or y-direction is assumed to be zero.

3. Results and discussion
3.1. Deformation of the dam

The analysis of deformations of embankment dams, either as an aid to the assessment of stability or to assess the likelihood of cracking or hydraulic fracture, has become possible within most dam engineering organizations, with the advent of powerful personal computers and relatively inexpensive finite element and finite difference analysis programs. The deformation obtained through using numerical methods has generally the tendency to be higher than those obtained through field measurements.
The deformation analysis is carried out using step construction approach for both self-weight and water load effects. About sixteen (16) steps are considered for the stepped construction while four (4) steps are considered for mechanical effect of water load.

Mohr-Columb constitutive model is used for the analysis purpose since large deformations are expected. Some other necessary material behaviours (G and K) for this model are calculated from the elastic parameters and other like C' and $\Phi$ are taken directly from the table.

3.1.1. Vertical and Lateral Displacements due to Self-Weight

The following figure shows the vertical displacement contour due to self weight effect during construction stages only. The maximum vertical deformation at the end of 16th step is found to be 1.5171m. The location where this maximum deformation occurs is about 73.85 m from the bottom of the dam which is to mean that this occurs at 0.55H. (H=134.87m) shown from figure 3.1. The lateral displacement is shown in the figure 3.2. The maximum lateral displacement is found to be -0.2948 m at the upstream shoulder and in the direction of the upstream face.

![Fig 3.1 plot of vertical displacement contour](image1)

![Fig 3.2 plot of horizontal displacement contour](image2)

3.1.2. Deformation in lateral and vertical along the center line of the dam due to self-weight

An axis passing through the center of the dam core is selected to look at the displacement distribution from the bottom to the crest of the dam. The following figure 3.3 shown the history points selected to sample displacement data.
The lateral displacement is almost zero on specified axis that passes through the centre of the clay core. But, on a relative scale the maximum negative lateral displacement with magnitude -0.075m occurs at 64m from the bottom of the dam. The maximum positive lateral displacement with magnitude 0.026m occurs at 120.00m from the bottom of the dam. The vertical displacement distribution along the center of the clay core is shown in the figure 3.4. The maximum vertical displacement along the centreline of the core has a magnitude 1.45m located at around 74.00m from the bottom of the dam.

3.1.3. Deformation in lateral and vertical direction across a horizontal axis at 53m from the bottom of the dam.

The lateral displacement along a section parallel to the base of the dam will have a profile shown in figure 3.5. The lateral displacement can easily be shown that at the center of the dam is nearly zero. Positive lateral displacement occurs in the downstream side and negative lateral displacement occurs in the upstream side. The maximum positive lateral displacement for this particular section is found to be 0.260m (nearer to the positive maximum for the entire dam) at 210.00m from the upstream end and maximum negative displacement is found to be -0.250m at 76.00m from the upstream end of the cross section.
The vertical deformation along the same cross section as the above yields the following result shown on the plot 3.6.

![Vertical Displacement Profile](image)

The maximum vertical deformation has occurred at the middle of the dam at around 152.00m and has a magnitude -1.31m (less than the absolute max. Vertical displacement).

### 3.1.4. Deformation in lateral and vertical directions along the upstream slope

The lateral displacement of the upstream slope has been determined and the profile is presented as shown in fig 3.7

![Lateral Displacement Profile](image)

The maximum lateral displacement of the upstream slope has a magnitude -0.1141 m and is located at 40.00 m from the bottom of the dam. (0.30H). Positive lateral displacement occurs at a height above 115.00m. The largest positive lateral displacement has a magnitude 0.0443m. Zero lateral displacement is recorded around a depth 78.00m from the bottom of the dam, this nearer to plane where the maximum vertical displacement occurs.

The vertical displacement on the upstream face of the dam will have the following profile shown in fig 3.8. The maximum vertical displacement on the upstream slope has a magnitude of -0.3415m and is located at 94.00m (0.70H) from the bottom of the dam.
3.1.5 Deformation in lateral and vertical directions along the downstream slope

The lateral displacement of the downstream slope has the following form as shown in fig 3.9.

The maximum lateral displacement has been found as 0.1418m at 40.00m from the bottom of the dam which is the same height as the maximum lateral displacement occurred for the upstream slope. (0.3H). Almost the downstream side will deform laterally in the +ve x direction or the displacement are positive.

The figure 3.10 shows the vertical displacement profile of the downstream face of the dam. The maximum vertical displacement is obtained as -0.2821m located at 94.00m from the bottom of the dam. (0.7H)

3.1.6 Vertical and Lateral Displacements due to Both Self weight and Water load
This section presents the effect of water load on the displacement/deformation of the dam at the specified locations discussed in the previous sections after impoundment of the dam with water. We try to analyze the effect of water load by comparing the different deformation profiles with the previously discussed ones and justify scientific reasons for the resulting outputs. The figure 3.11 shows the vertical displacement contour after the application of water load on the previously analysed dam for self-weight.

Fig 3.11 Vertical displacement contour after reservoir impoundment

The maximum displacement occurs around the same location as the previous one (i.e. 73.87m) and has a magnitude of 1.5621m which is a little bit larger than 1.5171m (due to self-weight only). The dam has displaced 0.045m downward at the centre due to the reservoir impoundment. The lateral displacement contour is presented in the figure below. The major aspect in this regard is the reduction of the negative displacement at upstream side and the increase in the positive displacement in the downstream side due to the push imposed by the water upstream of the dam.

Fig 3.12 Lateral displacement contour after reservoir impoundment

The maximum upstream shoulder lateral displacement is -0.26m and downstream shoulder displacement is 0.301m. The previous upstream lateral displacement was -0.2948m while the downstream lateral displacement was 0.26225m. Comparing the results obtained after impoundment and before impoundment, we can easily understand that the upstream lateral displacement has decreased by 0.0348m and the downstream lateral displacement has increased by 0.03875m.

The water load has posed a lateral pushing stress so that there is a movement all the way in the downstream direction, which in turn tend to lower the initial lateral displacement of the upstream shoulder and enhance the lateral displacement of the downstream shoulder.
3.1.7 Deformation in lateral and vertical directions along the centre line of the dam due to both self weight and water load

Fig 3.13 comparison of the lateral displacements along the centreline of the clay core

From the plot 3.13, we can see that the initial (due to self-weight) has shifted to the right due to the water load imposed on the upstream face of the dam. The maximum lateral displacement after the reservoir impoundment occurs at around 115.00m from the bottom of the dam and has a magnitude of 0.126m.

Fig 3.14 comparison of the vertical displacements along the centreline of the clay core

Fig 3.15 lateral displacement across section 53m above the base of the dam after impoundment

After the dam has been filled with water the lateral displacement will increase for all points across the section. The plot above depicts that the water pushes the dam away from it so that the lateral displacements will increase. The change in the lateral displacement seems to be large in the upstream, small around the centre of the dam and intermediate in the downstream side. The maximum negative displacement increased from -0.257 to -0.1441m and maximum positive displacement increased from 0.2581 to 0.3288m. The vertical deformation variation after the impoundment is also shown as in the figure be low.
What special conclusion can be drawn from the above plot is that the vertical deformation is the same before and after impoundment for the whole point on the section except near to the upstream slope. This is due to the presence of water load, imposing a hydrostatic pressure on zone3B material just on the upstream face of the dam. The variation of the displacement can be calculated as: $-0.2807m + 0.1472m = -0.1335m$. Hence, the water load causes the nearby upstream material to compress by 0.1335 m downward.

### 3.1.9 Deformation in lateral and vertical direction on the upstream slope due to both self-weight and water load

The lateral deformation of the upstream slope due to the application of water load additional to the initial self-weight will alter the displacement profile as shown in the figure below.

The maximum lateral displacement initially due to self weight only occurs at a depth of 40m (0.3H) from the bottom of the dam but after impoundment it has shifted to 94m (0.7H) from the bottom of the dam. Initially the magnitude was -0.1141m and after impoundment the magnitude counterbalanced to 0.2304m; producing a net change of 0.3445m. The vertical displacement of the upstream slope is analyzed in the same manner as the horizontal displacement and is presented in the following figure.
The vertical profile remains unchanged before and after impoundment and the maximum vertical displacement occurs at 94.00m (0.7H) from the bottom and the magnitude is -0.3415m. This makes both the maximum lateral and vertical displacements to occur at the same location after the impoundment of water.

### 3.1.10 Deformation in lateral and vertical direction on the downstream slope due to both self-weight and water load

The lateral displacement after and before impoundment has been summarized as follows in fig 3.19:

![Lateral displacement of the downstream slope before and after impoundment](image)

The lateral displacements of all points on the downstream slope have increased from their initial displacement due to self weight. The maximum lateral displacement has occurred initially at 40.00m (0.30H) with a magnitude of 0.1418m but after impoundment the maximum lateral displacement has occurred at 60.00m (0.44H) with a magnitude of 0.1773m. The total maximum deviation is 0.1773-0.1418 =0.035m. The vertical displacement profile of the downstream slope obtained for self weight only coincides with the profile obtained after impoundment. (Max. at 0.70H).

Summary of the locations of Max. Deformations on different parts of the dam

<table>
<thead>
<tr>
<th>Description</th>
<th>Location of Maximum Lateral deformation</th>
<th>Location of Maximum Vertical deformation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Entire Dam</td>
<td>0.40H</td>
<td>0.55H</td>
</tr>
<tr>
<td>Upstream Slope</td>
<td>0.70H</td>
<td>0.70H</td>
</tr>
<tr>
<td>Downstream Slope</td>
<td>0.44H</td>
<td>0.70H</td>
</tr>
</tbody>
</table>

Table 3.3 summary of locations of maximum deformations

### 3.1. Stress Analysis of the dam

Stress analysis after the impoundment has been carried out to identify the maximum and minimum normal and shear stresses and the location of occurrence of these stresses. Total stresses will be studied since the pore water pressure development due to seepage has not been considered at all. The stress analysis encompass the determination of

- Vertical (Szz) total stress distribution
- Horizontal (Sxx) total stress distribution
- Shear (Sxz) stress distribution
- Minor and Major Principal Stress distribution

#### 3.1.1. Vertical Stress Distribution (in Z-direction)

The vertical stress distribution after the impoundment of water behind the dam is shown in the figure below
Maximum vertical stress is occurred at the bottom of the dam on both sides of the filter and transition zones and has a magnitude of 2.36Mpa (compression) and the minimum occurs at the crest, toe and heel of the dam.

### 3.1.2. Horizontal Stress Distribution (in X-direction)

Horizontal stress distribution is shown according to the contour plot depicted in figure below:

The maximum horizontal stress has a magnitude of 0.9135Mpa (compression) and occurs at the bottom of the upstream filter and transition zones. The minimum horizontal stress has a magnitude of 0.1Mpa (compression) and occur at the crest, toe and heel of the dam. The ratio of the maximum horizontal to vertical stress will become:

\[
\frac{S_{xx}}{S_{zz}} = \frac{0.9135\text{Mpa}}{2.36\text{Mpa}} = 0.387
\]
3.1.3. Shear Stress Distribution (XZ-Plane)

The shear stress contour is shown below for the condition of after impoundment has ceased.

![Shear Stress Contour](image)

The shear stress distribution in the upstream and downstream zones has opposite sign due to the opposite lateral movement and the ‘symmetrical shape’ of the dam about the center of zone 1. On the upstream zone the maximum shear stress has a magnitude of 0.39018 Mpa and the downstream zone has a maximum shear stress of 0.26154 Mpa as shown on the contour plot. In general no shear stress occur around the central zone of the dam. The maximum shear stresses occur at the bottom of the dam near the foundation.

3.1.4. Major and Minor Principal Stresses Distribution

The major and minor principal stresses are very important in knowing the state of stresses at a point and whether or not failure will occur. The major principal stress is the positive larger normal stress applied on a plane without shear stress and minor principal stress is less than the major principal stress and occurs on a plane without shear stress. The following plot shows the distribution of the major principal stress inside the dam body:

![Major Principal Stress](image)
The maximum major principal stress occurs at the bottom of the dam around the clay core. The magnitude of the maximum major principal stress is 0.84104Mpa (Compression). The minor principal stress distribution is shown in the figure below.

![Minor Principal Stress distribution](image)

**Fig 3.24 Minor Principal Stress distribution**

The maximum minor principal stress occurs at both the upstream and downstream side of the bottom of the dam. The maximum magnitude of the minor principal stress is 2.4633Mpa (compression). The minor principal stress is less than the major due to the reason that major principal stress must be algebraically greater than minor principal stress.

### 3.1.5. Check for Cracking and Yielding of the Dam

The principal stresses are investigated for the existence of crack in the dam body and the block state command on the FLAC3D software is used to know the states of the block with regard to failure.

### 3.1.6. Check for Cracking

The following plot of minimum principal stress is used to check for the existence of cracks. (Smax corresponds to minimum principal stress in FLAC).

![Plot of minimum principal stress](image)

**Fig 3.25 Plot of minimum principal stress**

The minimum principal stress is positive around the toe of the dam. Hence, crack might be formed around the toe of the dam. In order to check Yielding of the dam the block state command is used and the following plot with the state of each plot with regard to failure conditions is obtained.
According to the block states shown above, there is no any block (zone) that fails currently by shear or tension. Therefore, we can conclude that the dam is safe against shear failure or yield in.

4. Conclusion

For the entire dam the maximum lateral and vertical displacement found at 0.4H and 0.55H from the bottom of the dam respectively. The vertical displacement of the dam in all cases of this study is increased in the case of impounding reservoir load additionally considered over self-weight. Along the upstream slope of the dam both vertical and lateral deformation is occurred at 0.7H from the bottom of the dam. Lateral displacement in the upstream slope of the dam decreased when the water load is encountered to counterbalance the displacement due to self-weight, on the contrary the deformation increased in the positive x-direction on the downstream slope of the dam. Along the downstream slope the location of the maximum lateral and vertical deformation is found at 0.4H and 0.7H from the bottom of the dam respectively.

Maximum vertical stress is occurred at the bottom of the dam on both sides of the filter and transition zones. Both minimum vertical and horizontal stress occurred at the crest, toe and heel of the dam but the magnitude is different. The maximum horizontal stress occurs at the bottom of the upstream filter and transition zones. In general no shear stress occur around the central zone of the dam. The maximum shear stresses occur at the bottom of the dam near the foundation.

The minimum principal stress is positive around the toe of the dam. Hence, crack might be formed around the toe, crest and heel of the dam.
References


