SLOPE STABILITY OF HILLOCK NEAR BANDA BRANCH CANAL USING GTS NX SOFTWARE - A CASE STUDY

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ABSTRACT

Tillari Interstate Irrigation Project is a Joint Venture of Maharashtra State & Goa State, situated in Sindhudurg District of Maharashtra state. Construction of main dam, LBC (length 56.179 Km) and RBC (length 48.395 Km) was completed in the year 2009. Banda Branch Canal having length of 57.00 Km is off taking from RBC. Construction of concrete lining to canal between chainage 9/800m to 9/870 was completed in the year 2009. On 5.10.2009 heavy rainfall took place, resulting in damage of concrete lining, due to adjacent hillock slide towards canal. Then, it was decided to construct R.C.C. conduit and its construction was taken up. Again due to heavy rainfall on 16 & 17.07.2011, adjacent hillock on IP side of canal was cut and was slided towards canal, which damaged to executed R.C.C Box Conduit. No construction work was taken in hand from year 2011 to 2016. In the year 2016 excavation work for foundation of RCC Box Conduit and for providing slope of 1.5: 1 to hillock and berm of 5.00 m wide at every 5.00 m depth was taken up. Again new cracks in hillock and uplift movement at foundation level of box were noticed during excavation. Project was included in the scheme of Hon. Pradhanmantri Krushi Sinchai Yojana from Central Government. So, the target was fixed for completion of the project before March 2018. To overcome the above situation installation of piles with tie beams were recommended by using GTS NX software at berm level and below RCC Box Conduit and were executed by Project Authorities. From the last two rainy seasons, no deflection was observed in the box conduit portion.

1. HISTORY OF THE PROJECT

Work of Concrete protection wall in between Ch. 9/800 m to Ch.9/870 m and Concrete lining in between Ch. 9/870 m to Ch. 9/920 m was completed to Banda branch canal km No.10 during 2009. Due to heavy rainfall in the vicinity of Banda area on 5.10.2009, adjacent hillock on Inspection Path side of canal cuts slide toward canal resulting into fully damage to the constructed protection wall and lining work. Crack of parabolic shape has developed in hillock about 1.50 to 2.00 m wide about 200 m length and earth from hillock gets slipped into canal area. After this incident, it was decided to construct R.C.C. Box Conduit in between Ch. 9/735 m to Ch. 9/920 m for which design was given by Central Design Organization, Nashik. Slipped material from canal was removed and construction work of RCC Box Conduit in between Ck. 9/735 m to Ch. 9/920 m was taken up. Work of construction was of RCC Box Conduit was started on Dt. 09.02.2011 and stopped on 30.05.2011 for rainy season. During the working period, length of about 40 m (Two Boxes of 20.00 m length) was fully completed and in balance length work was at construction stages. However due to heavy rainfall on 16 & 17.07.2011, again adjacent hillock on Inspection Path side of canal cuts has slide toward canal and come into canal resulting into fully damage of the executed R.C.C Box Conduit. Cracks in the hillock was developed and earth from hillock was slipped in canal area. RCC Box conduit was totally uplifted horizontally & vertically from its original position. No construction work was taken in hand from year 2011 to 2016 i.e about 4 to 5 years. This portion was kept under observation. In the month of March 2017, again construction of RCC box culvert was taken with Cover Box Conduit with loading of earth material about 6.00 m height on top of box as counter load and providing slope of 1.5:1 to hillock and berm of 5.00 m wide at every 5.00 m depth was taken up. However, again new cracks in hillock and uplift movement at foundation level of box was noticed during excavation. Soil from hillock slides towards canal and number & sizes of cracks has increased day by day in Feb 2018. Uplift movement is observed in between Ch. 9/775 m to Ch. 9/860 m at foundation level, Soil at foundation level is uplifted about 1.50 to 2.00 m vertically and 4 to 5 m horizontally. The constructed box culvert has been uplifted (Photo 3.) and slope slide from the top tension crack is shown in Photo-1 & 2.

Photo 1 & 2 : Downward slide of overburden soil mass from tension crack.

Photo 3 : The tilt and uplifting of the constructed culvert. **Photo 4** : Benching of soil mass as a treatment to stabilizing the slope.

A view of the slope after making of the benches is shown in Photo 4. Even after making of the benches the upward movement of the hill toe base and slide of top soil mass has not stopped. So some measures to strengthen the overburden soil mass has to be adopted.

To provide the measures to strengthen the overburden soil mass software Midas GTS NX is used to model the available strata and cross-sections at the site. Midas GTS NX is a comprehensive finite element analysis software package that is equipped to handle the entire range of geotechnical design applications including deep foundations, excavations, complex tunnel systems, seepage analysis, consolidation analysis, and embankment design, dynamic and slope stability analysis. GTS NX also has an advanced user friendly modeling platform that enables unmatched levels of precision and efficiency. Before giving this project to CWPRS, project authority has taken the remedial measure from IIT, Powai. Remedial measure given by IIT Powai to project authority this office have taken the run of that measure given in the FEM software model. It was observed that the measure suggested by IIT Powai costing was not sufficient to hold the laterite soil strata present along the hillock.

2. LITERATURE SURVEY

Reginald E. Hammah, John H. Curran, Thamer Yacoub & Brent Corkum (2004) presents a simple approach for performing the FE SSR analysis of rock slopes for which strength is modelled with the Generalized Hoek-Brown failure criterion. The paper shows that determining factored Hoek-Brown parameters, as is done for the Mohr-Coulomb, would be at best cumbersome and would slow down computations considerably. It suggests an approximate method that involves first determining a Mohr-Coulomb envelope equivalent to a Hoek-Brown model, and then applying the resulting equivalent cohesion and friction angle values in the standard SSR technique.

Saha, S. (2010) main objective of this study was to analyse the slope in a detailed way from the results obtained from FE analysis of slopes. For this purpose, he has taken few examples and derived a methodology to depict the slope failure pattern, the global F.S. of the whole slope, the deformation pattern, the local F.S. and joining those points through an assigned path, F.S. was obtained along that path. Again by minimizing the F.S. expression w.r.t. the angle of inclination ofthe failure path with the horizontal, the straight line failure path and the corresponding F.S. were obtained for that. Finally these numerical values of F.S. were compared with the results obtained using already established methods, and it was observed that these are in the close neighborhood; hence it can be used for the analysis of slopes for determining its stability.

Meftah Ali (2016) aims to analyze the static stability and pseudo-static slope by using the finite element method. The analyses were carried out using SAS-FEM (stability analysis of slopes using finite element method), modeling in two dimensions to calculate the safety factor values, under various loads such as earthquake effect, the water level, and road mobile charges, to evaluate the state of the slope. The results of the static study show that the slope is stable against by the results of the study indicate that the pseudo-static makes the slope unstable.

Above all researcher deals with the various researches associated with the various numerical FEM model for slope stability, has been performed earlier, in this paper also remedial measure to prevent sliding of hillock slope. CWPRS has recommended pile with beams as a tool measure to prevent sliding of hillock slope.

3. GEOLOGY SURVEY

3.1 Senior Geologist from CDO, Nashik surveyed the area and stated that, the area in the vicinity of the Banda branch canal belongs to Archean to Dharwar super group of paleoproterozoic to Archean era. The canal alignment is covered by Lateritic soil with some traces of Tachylite like material. The lateritic soil is derived from the rich iron and aluminum rocks and was commonly considered to have formed in hot and wet tropical areas. Nearly all laterites are of rusty red colour because of high iron oxide. Along the Eastern side of canal boulders of compact basalt is noticed.

3.2 Bore hole details

the top layer strata is laterite soil, which when expose to atmosphere reduces the cohesion in the strata, and the bottom of that strata is laterite murrum in some bore and rock observed in maximum bore. The average depth of hard stratum i.e. rock subsurface was taken as 14 m from the top surface of soil as shown in Fig.1 for slope stability analysis. The same stratum was assumed at the top surface of the hill, as borehole data is not available at the top of hill and even at the centre of the slope.

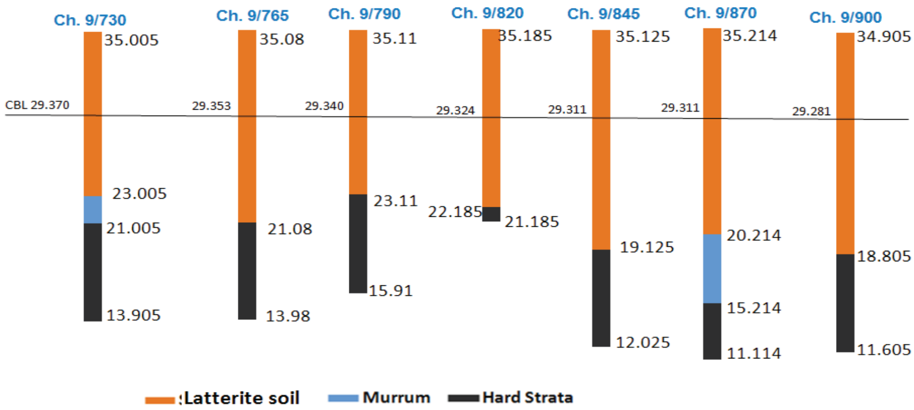

Figure 1 : Borehole details for strata investigations of Banda branch canal. Black colour indicates rock stratum.

4 ABOUT FINITE ELEMENT MODEL STUDIES

4.1 Seepage Analysis

The seepage analysis [12] is normally divided into steady state analysis and transient analysis. Steady state analysis calculates a time-independent solution, whereas transient analysis calculates a time dependent solution as a result of changing inflow/and or out-flow conditions.

4.2 Seepage Flow Rule

Midas GTS provides solution for seepage analysis based on laminar flow using Darcy's law [4]; Darcy's law is expressed by the following equation:

$$q = ki \tag{1}$$

where,

q : Seepage quantity of water per unit area, k : Permeability coefficient, and i : Hydraulic gradient

Darcy's law was originally derived for saturated soil, and is verified to be applicable to the flow of unsaturated soil according to published studies *(Darcy, 1856 [4], Whitaker, 1986 [18], Cunningham, and Williams [3], 1980, Pant, Mitra, and Secanell, 2012 [16], Kerkhof, 1996 [10]. Klinkenberg, 1941 [12], Brinkman [2], 1949 Jin, Uth, Kuznetsov, and Herwig, 2015 [9]*. The only difference is that the permeability coefficient of unsaturated soil is not a constant but is a function, and indirectly varies with the change of pore water pressure. Darcy's law is expressed in equation 2, as,

$$v = ki \tag{2}$$

where,

v is known as the Darcian velocity. When the water flows through permeable or porous media, the actual average velocity can be calculated by dividing the Darcian velocity by the porosity of soil.

4.3 Governing Equation

Midas GTS uses the differential equation, (3) for the seepage analysis:

$$\frac{\partial}{\partial_x}\left(k_x\frac{\partial H}{\partial x}\right)+\frac{\partial}{\partial_y}\left(k_y\frac{\partial H}{\partial y}\right)+\frac{\partial}{\partial_z}\left(k_z\frac{\partial H}{\partial z}\right)+Q=\frac{\partial \Theta}{\partial t} \quad \text{....(3)}$$

where,
H: defines total water head causing the flow; k_x, the Permeability coefficient in the direction of x; k_y, the permeability coefficient in the direction of y; k_z, the permeability coefficient in the direction of z; Q, the flow quantity; Θ, the volumetric water content and t is the time.

The above equation shows that the change of volumetric water content equals the difference between the inflow and outflow quantities over a large volume in an arbitrary location during an arbitrary time. In brief, the flow rates in the directions of x, y, and z-axes plus external flow quantity are same as the rate of change of volumetric water content.

The governing equation is the seepage equation for the transient flow. Since the flow quantities of inflow and outflow over the large volume are constant regardless of the time in the steady state flow, the governing equation (Eq. 4) is obtained by assigning the Eq. (3) equal to zero.

$$\frac{\partial}{\partial_x}\left(k_x\frac{\partial H}{\partial x}\right)+\frac{\partial}{\partial_y}\left(k_y\frac{\partial H}{\partial y}\right)+\frac{\partial}{\partial_z}\left(k_z\frac{\partial H}{\partial z}\right)+Q=0 \quad \text{....(4)}$$

The change of volumetric water content depends on the change of stress state and the characteristics of soil. The saturated and unsaturated stress state conditions are expressed as two state variables. These are $(\sigma - \rho_a)$ where σ is the total stress, ρ_a pore air pressure and ρ_w pore water pressure.

The seepage analysis in Midas GTS consists of the conditions of constant total stress. The loading and unloading on soil does not exist. The pore air pressure is constant under the atmospheric pressure during unsteady flow state since unloading on the soil by itself does not exist. $(\sigma - \rho_a)$ is a constant, and does not affect the change of volumetric water content. Thus, the change of volumetric water content is only a function of the change of pore pressure since it depends on the change of stress state $(\sigma - \rho_w)$ and ρa is constant.

The change of volumetric water content is related to the change of pore pressure according to the stress state and the characteristics of soil, and the relation can be expressed in Eq. 5 as:

$$\partial\Theta = m_w\,\partial\,u_w \quad \text{....(5)}$$

where,

m_w : Storage coefficient

Also, the total head is expressed as the summation of pressure head and potential head as shown in Eq. (6):

$$H=\frac{p_w}{\gamma_w}+y \quad \text{....(6)}$$

where,
H is the total head; p_w, the pore pressure; γ_w, the unit weight of water, and y defines the height.
Eq. (6) is written as:

$$p_w=\gamma_w\,(H-y) \quad \text{....(7)}$$

Substituting Eq. (7) into Eq. (5) gives the following, Eq.8:

$$\partial\Theta = m_w\gamma_w\,\partial\,(H-y) \quad \text{....(8)}$$

Substituting Eq. (8) into Eq. (3) gives the following, Eq. 9:

$$\frac{\partial}{\partial_x}\left(k_x\frac{\partial H}{\partial x}\right)+\frac{\partial}{\partial_y}\left(k_y\frac{\partial H}{\partial y}\right)+\frac{\partial}{\partial_z}\left(k_z\frac{\partial H}{\partial z}\right)+Q= m_w\gamma_w\frac{\partial (H-y)}{\partial t} \quad \text{....(9)}$$

Since height is constant, the derivative of y is eliminated but the following differential equation still holds;

$$\frac{\partial}{\partial_x}\left(k_x\frac{\partial H}{\partial x}\right)+\frac{\partial}{\partial_y}\left(k_y\frac{\partial H}{\partial y}\right)+\frac{\partial}{\partial_z}\left(k_z\frac{\partial H}{\partial z}\right)+Q= m_w\gamma_w\frac{\partial H}{\partial t} \quad \text{...(10)}$$

The in situ permeability values at various depths of the borehole and the strata properties observed in the test borehole are used in the FEM model.

5. INPUT DATA

The rock cores received from site had high strength , but for the analysis purpose, the same values cannot be adopted due to low core recovery values of about 45%. Due to weak zones/joints in the subsurface rock, it was considered to be a fair rock as per Geomechanics classification. The cohesion was taken as 250 Kpa and angle of friction as 35^0. The average density dry, saturated and Poisson's ratio of rock mass was 2.40, 2.60 g/cm^3 and 0.21 respectively. The modulus of deformation is determined as 6.00 Gpa. The permeability coefficient as determined during extracting of rock cores on I.P. side @10 m from centre is taken as 2.50 e^{-6} m/sec.

The soil samples received from the site was tested in CWPRS laboratory. The properties of overburden soil are taken as: Cohesion 0.15 Mpa and angle of friction as 31^0.The average density dry, saturated and Poisson's ratio of soil mass is 1.318, 1.5318 g/cm^3 is 0.30 respectively. The modulus of deformation was determined as 100 Mpa. The permeability coefficient as determined during extracting of rock cores on I.P. side @10 m from centre was taken as 5.30e^{-6} m/sec.

The average depth of hard stratum i.e. rock subsurface is taken as 14 m from the top surface of soil for slope stability analysis. The same stratum has been assumed at the top surface of the hill, as borehole data was not available at the top of hill and even at the centre of the slope.

From the supplied cross-sections by the project authorities (Figure 2) a critical section at chainage 9830 m was chosen for the analysis.

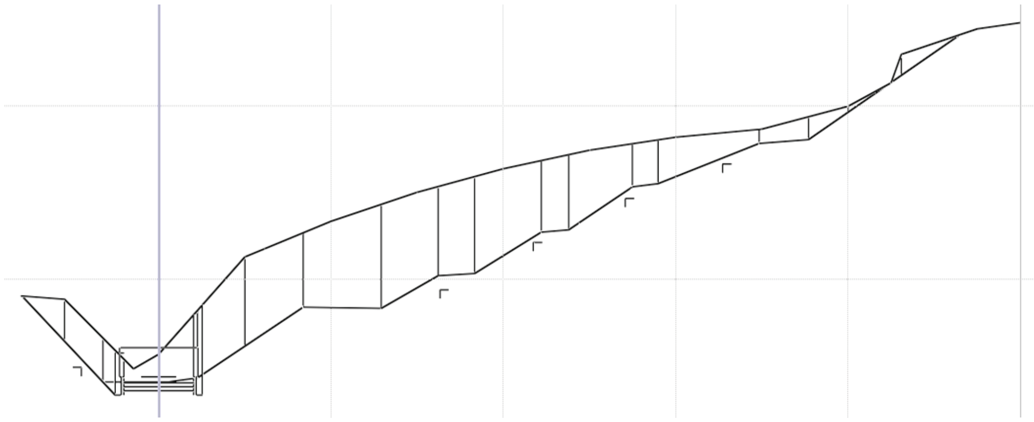

Figure 2 : Critical Cross section at chainage 9830m is used for analysis of slope stability.

6. GTS NX RUN WITHOUT MEASURE

A 2D model was analyzed without providing any treatment by adopting the given cross section (Figure 1). The results are shown in Figure 3.

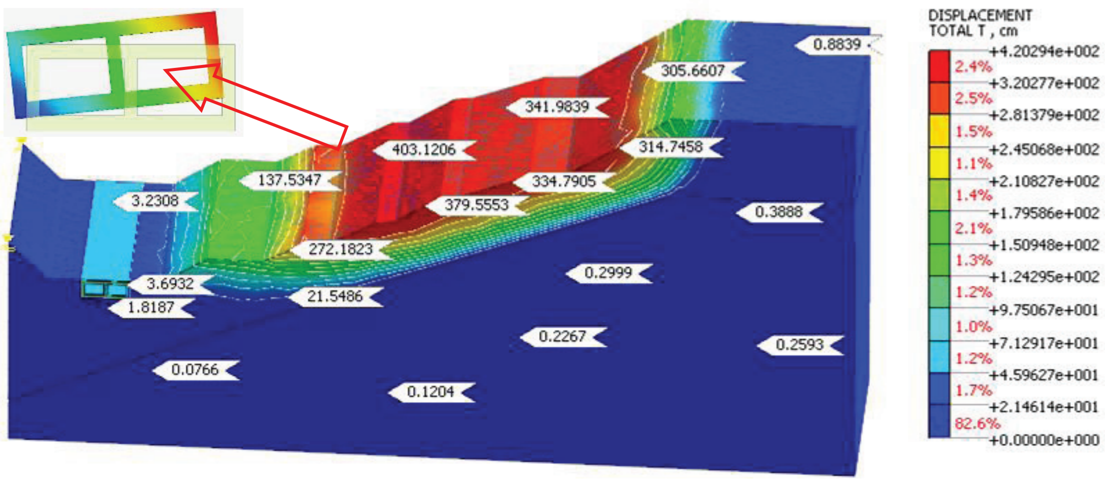

Figure 3 : FEM Model with original profile showing displacement values and displacement of culvert.

It was observed that the displacements at the top of slope was 379.79 cm. With so high displacement in the model, which made the slope unstable and further hit the lower portion of the slope. So, to reduce the displacement at higher reaches some remedial measures are required. As suggested by project authority attempt has been made by providing a retaining wall in the model as per the details shown in Figure 4.

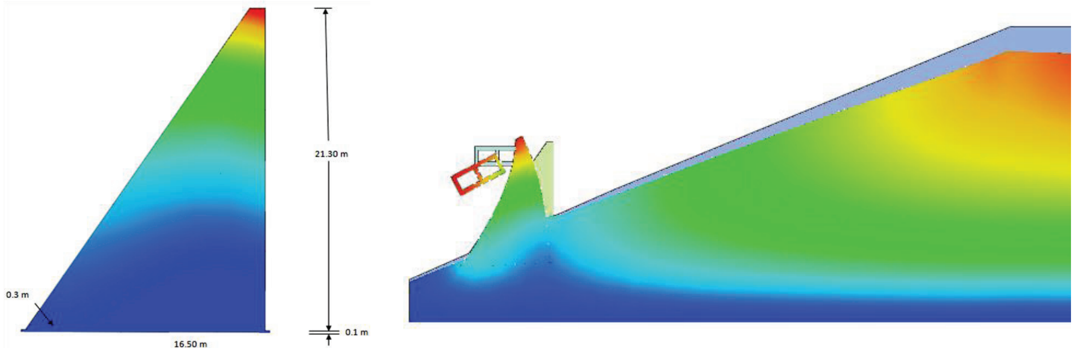

Figure 4 : Cross-Section of retaining wall provided in the model.

After the analysis, it was seen that it is not able to bear the sliding force of the overburden material as shown in Figure 5. It is observed that there is no change in displacement values compared with no measure model studies Figure 3. Similarly, deflection in the retaining wall is also observed.

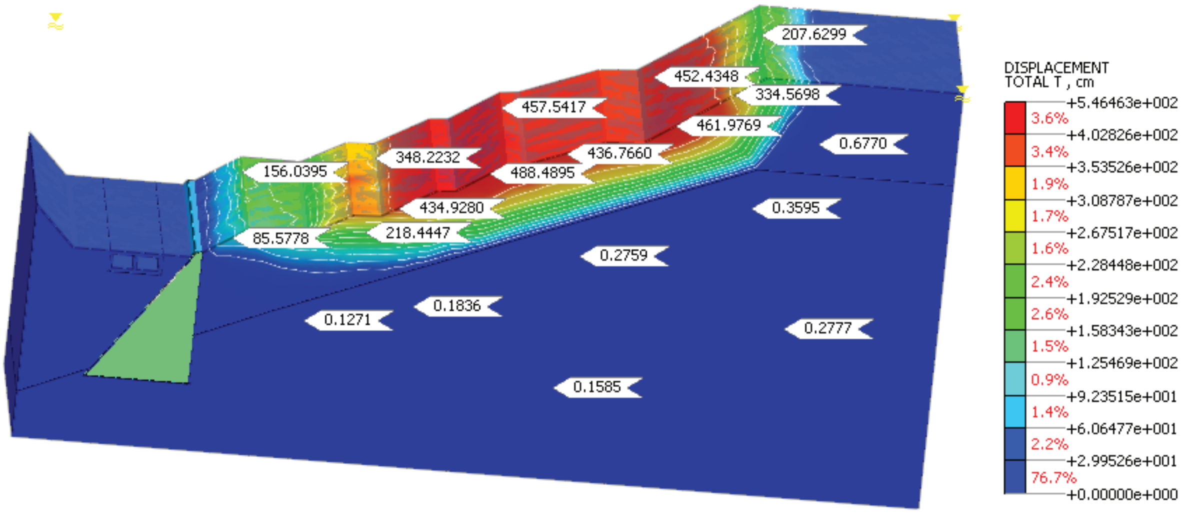

Figure 5 : 3D FEM Model with Retaining wall.

7. REMEDIAL MEASURES FOR STABILIZING THE SLOPE

7.1 Remedial measures suggested by IIT Powai

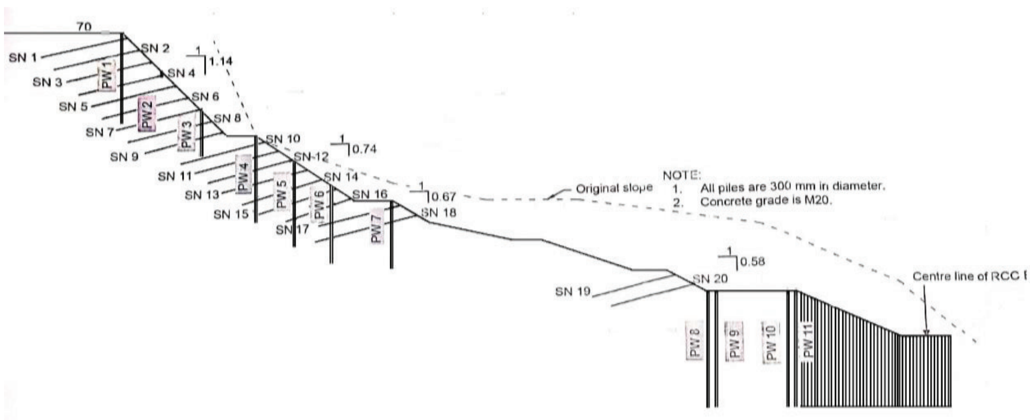

Figure 6 : Critical Cross section shown the measures suggested by IIT Powai.

Suggestions given by IIT, Powai like providing Piles, Anchors, Shortcrete was consider for stabilizing the slope. Various combination of centre to centre distance of piles, anchors & their position, sizes were adopted in the model to stabilize the slope profile and also the box culvert. The properties of material used for analysing the model is given in Table 1

Table 1 : Material properties used for the analysis of the Tillari slope stability model

Material	Soil	Rock	Piles	Anchors	Shortcrete
Properties					
Elastic Modulus (KN/m^2)	1e^5	5e^6	2e^7	2e^8	2e^7
Poisson's Ratio	0.3	0.21	0.25	0.3	0.2
Unit Weight (KN/m^3)	13.18	24	24	41	24
Unit Weight Sat. (KN/m^3)	15.318	25	26	42	25
Permeability Coefficient (m/sec)	5.3e^{-6}	2.5e^{-6}	1e^{-5}	1e^{-5}	1e^{-5}
Cohesion C (KN/m^2)	14.878	250	-	-	-
Friction Angle (ϕ)	31	35	-	-	-

In this model, 380 mm diameter piles 3 numbers in a row and 5 m center to centre below the box culvert with a length of 18 m are provided (Figure 5). Similar type of piles was provided at five different location viz EL.36.692 m, 40.701 m, 56.110 m, 68.193 m and EL68.193 m (Figure 6). Anchors of diameter 32 mm, with a sleeve of 100mm diameter having a length of 18 m are used from the base of slope to top, at spacing of 5 m center to centre horizontally and vertically. Due to low cohesive values of the soil material, anchors are grouted in the steel sleeve, to behave as composite material. Shortcrete of thickness 50 mm is provided all along at the top of the slope profile. By using the above measures analysis is carried out and the values of the displacement have reduced drastically and found under permissible limit in 3D models (Figure 7). The displacement values when compared with no measure and retaining wall models, are found under safe limits, which ensures the stability of the slope.

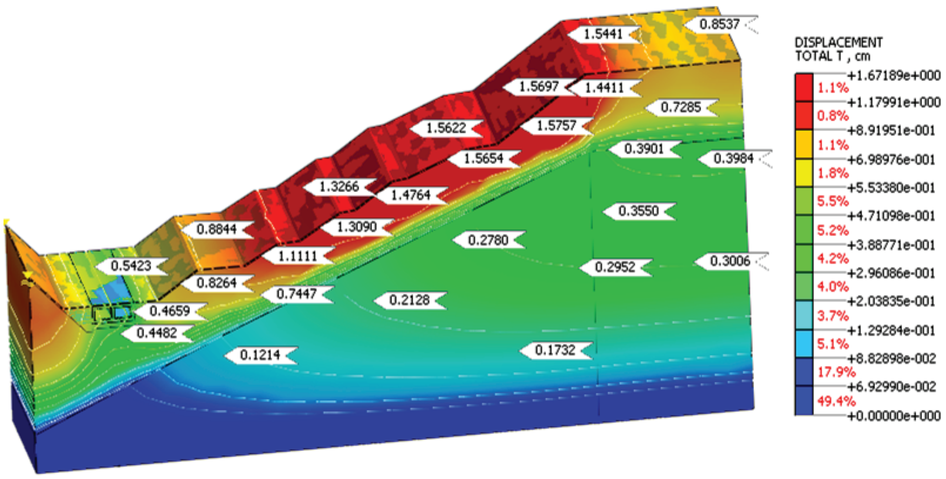

Figure 7 : 3D Model showing displacement values with Anchors at 5 m c/c, Piles at 5 m c/c and Shotcrete of 50 mm thickness. (The max displacement is about 0.47 cm near box culvert and 1.67cm along the slope and found safe for stability purposes.)

Further, the studies were also carried out with a change in centre to centre distance of piles and anchors, from 5 m c/c to 6 m c/c. The results are compared with 5 m c/c model. It is found that there is increase in minimun and maximum values of displacement from 0.47 to 0.53 and 1.67 to 1.97 cm respectively and are found safe (Figure 8).

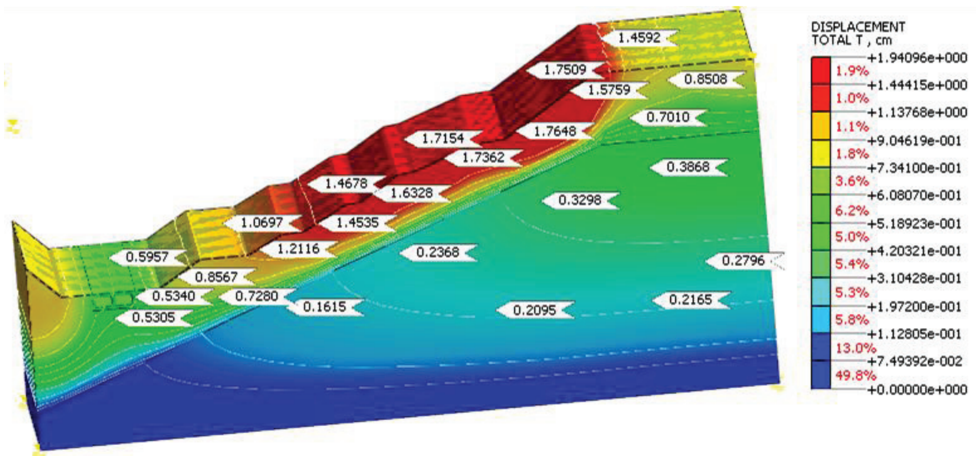

Figure 8 : 3D Model with alignment of Anchors 6 m c/c, Piles 6 m c/c and Shotcrete of 50 mm thickness showing displacement values. The max displacement is about 0.53 cm near box culvert and along the slope is 1.94 cm.

The measures provided as above for stability measures seems to be uneconomical and tedious to comply. These measures will be very useful when another structure is to made in the higher ends of the slope. Therefore, further a new model has been considered for the analysis i.e. by providing solid piles with beams

7.2 Remedial measures suggested by CWPRS Pune

The problem with us to stop the movement of box culvert and the slope base. Therefore, a new model was considered for the analysis i.e. by providing solid piles with beams only in the lower reaches i.e. near the base of the slope. In this model solid piles of 380 mm diameter, 3 numbers in a row with 4 m center to centre with a depth of 18 m below the box culvert were provided. Similarly, at EL.36.692 m, plies of 380 mm diameter in 2 rows were provided by connecting the top of the piles with beams of size 380 mm X 380 mm. The connecting of beam improves rigidity to the piles which further avoid deflection. All the piles were to be embedded in the rock for minimum depth of 2 m and pile below the box culvert should be constructed monolithic. The anchors and piles at the top levels are not to be considered in this model. The arrangement of piles, box culvert and beams are shown in Figure 9. The displacement values after the analysis are shown in Figure 10. It was observed from the results after the analysis that along the periphery of the box culvert the dislpacement values are ranging from 0.63 cm to 0.79 cm and similarly up side of hill the maximum displacement value is 20.74 cm.

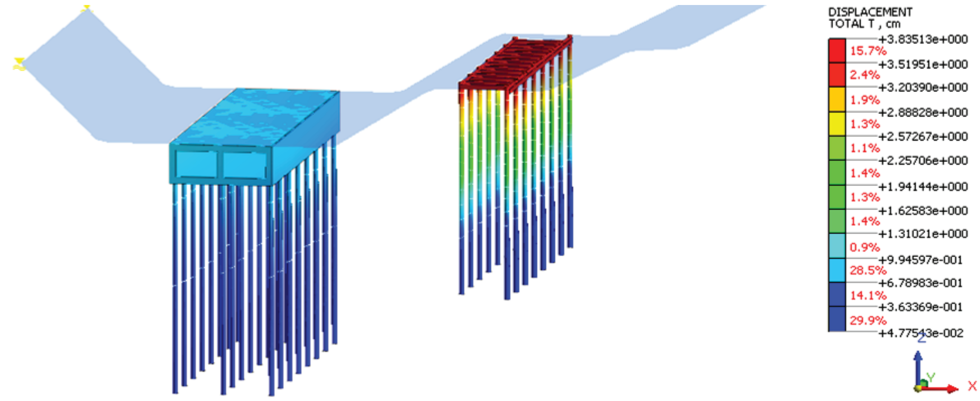

Figure 9 : Model showing box culvert with five rows of 380 mm dia. solid piles at 4 m c/c, 18 m depth and connecting beams of size 380x380 mm.

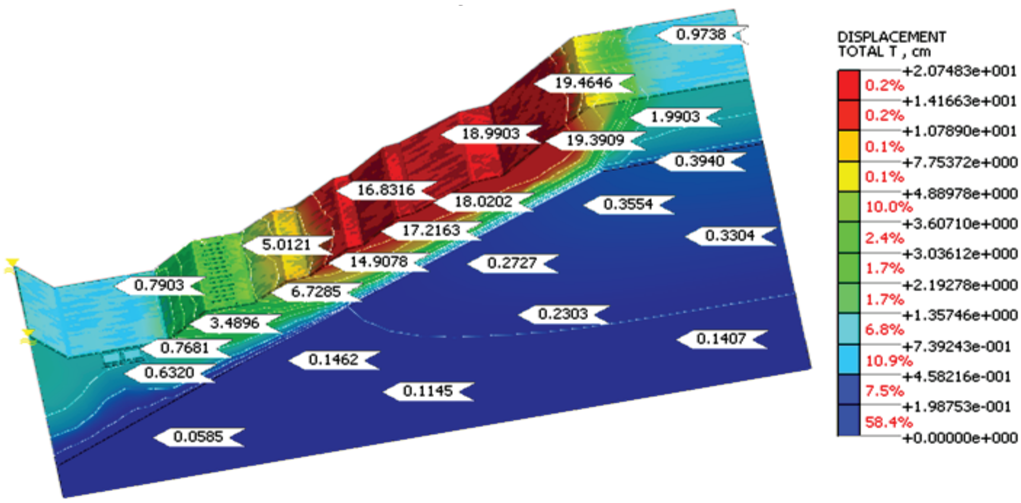

Figure 10 : 3D Model showing displacement values after the analysis with piles and connecting beams.

8. RESULT AND DISCUSSION

By providing retaining wall, slope stability problem was not tackled in this case. The displacement values on up side of the slope are high, which may create thrust for displacing the box culvert. The soil being a latterite having less cohesion, retaining wall foundation level should go beneath of rock strata for anchoring. Similarly, construction of retaining wall on less cohesive soil and depth of 16 m is very difficult.

When studies were conducted by using piles, anchors and shotcrete for 5 m or 6m centre to centre. The result are satisfactory and it is observed that for 5 m centre to centre displacement values are less than 5 mm, the result is appreciable. The length of the box culvert is 80 m, definitely there will be expansion joints at some interval. Displacement values less than 5 mm will not break the filling material between the joints. Providing above measures was not found economical, so the option without anchors and top slope piles were suggested.

Studies were conducted by providing 380 mm dia. piles at 4m c/c below box culvert and at EL.36.692 m. with connecting beams at top of pile. The displacements near box culvert found about 0.79 cm. For economical and construction point of view, these measures were provided and serves the purpose of stability of box culvert and its surrounding areas. It was observed along the periphery of the box culvert that the displacement values were within permissible limit. Similarly up side of hill the maximum displacement value is 20.74 cm. We can conclude that the measures provided in this option were fulfilling the requirement i.e. reduction in displacements values on up side of the hill. From the last two rainy seasons, no deflection was observed in the box conduit portion.

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