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RESERVOIR SLOPE STABILITY OF BUNAKHA HYDRAULIC PROJECT BY FINITE ELEMENT METHOD

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Abstract — *The stability of the hill slopes along the reservoir rim is important for the feasibility of the hydroelectric project. In this paper stability analysis of hill slopes of Bunakha Hydroelectric Project (BHEP), Bhutan has been carried out in two different methods like three-dimensional (3-D) strength reduction method (SRM) and nonlinear static method. Factor of safety (FOS) is determined for different conditions of dam like dry, steady seepage, submergence, drawdown and earthquake loading at two critical slope chainages of 1700 and 4000m.The factor of safety is found to be 1.67 for dry condition which increases to 1.97 in submergence condition at water level 110m at chainage 1700m. The rapid drawdown condition of the slope is also studied when water level reduces from RL 2006 to 1956m and the FOS value of slope is found to be 1.52. Similarly for another critical cross section at chainage 4000 m the reservoir slopes are found to be safe.*

Keywords- *Finite Element Method, Strength Reduction Method, Drawdown, Slope Stability, Factor of Safety*

I. INTRODUCTION

Tehri Hydro Development Corporation Ltd., (THDCL), is entrusted with the work of updation of DPR for Bunakha Hydroelectric Project (BHEP), Bhutan, by Govt. of India and Royal Govt. of Bhutan. In pursuance of this, THDCL has referred the studies to CWPRS, Pune for assessing the stability of the hill slopes along the rim of the reservoir after submergence. The project is located at latitude 27^0 8' 00" N and 89^0 32' 33"E near Bunakha, on river Wang Chhu, 3.25 km upstream of the Chukha dam. The project consists of a concrete gravity dam of 197 m height and a shaft type power house of 180 MW capacity. The reservoir storage is 211.82 M.cum. and it covers an area of 3.66 km2 at FRL. Slope stability is affected by the strength and deformation properties of intact rocks, geometry and distribution of discontinuities throughout the rock mass. Thus, it is the nature of the discontinuities (joints, fractures, bedding planes and faults) and not the intact rock that governs the mechanical behavior of the rock mass. For ensuring the stability of rock slopes analysis of the structural fabric of the site is required to be carried out. Traditionally, analytical calculations of potential instability are carried out by means of limit equilibrium models; more recently, however, numerical models are dominantly in use.

The stability of rock slopes has long been regarded as a classic and difficult problem for geotechnical engineers because rock masses are heterogeneous, discontinuous media composed of rock materials and naturally occurring discontinuities, such as joints, fractures, and bedding planes.

Many investigators have evaluated rock slope stability, thereby developing many new methods for meeting engineering requirements, such as the limit equilibrium method [1-3], the finite element method [4-7] and the limit analysis methods [8-9]. In the assessment of rock slope stability, the use of rock mass parameters directly from a nonlinear failure envelope is known to yield a highly accurate safety factor.

In general, slope stability analysis consists of two steps: the first step is to calculate a factor of safety for a specified slip surface, and the second is to find a critical failure surface that is associated with the minimum safety factor. Conventional methods of slope analysis based on the concept of limit equilibrium have been widely adopted, owing mainly to their simplicity and applicability. The safety factor, resulting from the limit equilibrium method, however, is not uniquely determined, because it varies with the assumption made for the slip surface. The result may not be reliable if non homogeneous and anisotropic stratifications are considered. The fundamental feature of limit equilibrium method is only considering the static equilibrium condition and Mohr-Coulomb criterion; that is, the solution to the problem is the analysis of the equilibrium of forces at the moment of the failure of soil mass. However, there is a restriction of Mohr-Coulomb criterion to the description of rock mass; for example, it fails to explain the impact of high stress areas but only reflects the feature of linear failure in rock mass [10-11]. The various 2D slice methods of Limit equilibrium analysis have been well surveyed by Fredlund and Krahn [12].

Numerical methods permit the treatment of slope stability problems involving complexities relating to geometry, material anisotropy and nonlinear behaviour. In particular, for the stability analysis of rock slopes, the numerical methods are more suitable because the behaviour of rock slope is much more dependent on the characteristics and integrity of the rock mass. The finite element method (FEM) in particular is becoming increasingly popular for slope stability analysis in situations where the failure mechanism is not controlled completely by discrete geological structures.

II. LITERATURE REVIEW

Research work and investigations carried out in the field of slope stabilization by finite element method based upon SRM and other techniques are discussed below. Duncan, 1996; Griffiths and Lane, 1999 showed their study on the assumption that FEM is better approach while dealing with slope stability problems in terms of accuracy with respect to failure mechanism [13-14]. The study observed that FEM required dealing with a lesser number of prior assumptions that provided for precise and output results. The paper illustrated various examples of slope stability problems and compared them with other solution models for soil slopes. Graphical presentation revealed that deformations and failure mechanisms were way better represented by FEM in comparison with traditional Limit Equilibrium Method. As a conclusion this study further emphasized on using FEM to obtain precise solutions for geotechnical problems.

Dawson et al., 1999 investigated major differences between strength reduction method and traditional method of slices [15]. The study established that the critical failure surface and critical factor of safety are found automatically rather than assuming the failure plane. SRM analyses are performed on a wide range of slope angles and slope heights and it is concluded that SRM provided fractionally marginal better values of Factor of Safety. The study mentioned that long run times required for performing SRM analysis is one of the major disadvantages of this method. Hsiung et al., 2010 studied ground deformation during construction of a cross passage during the construction stage of Delhi Metro via MIDAS GTS [16]. They have prepared 3D Finite Element Model and modified Mohr Coulomb method is used to perform the post construction deformation and displacement analysis caused by induced stresses. The study concluded by simulating the cross passage on the software that the soil properties had to be reconsidered to be able to minimize the deformations and displacements. Soil strength parameters used in the design stage are re-examined and it was recommended to change these properties based on the 3D analysis. Saikia et al. [17] conducted SRM analysis on soil slopes of Sonapur in Meghalaya and concluded that the factor of safety decreased with increase in the angle of inclination. Slopes going above 20m in height are concluded to be severely vulnerable to slope failures both in the dry as well as rainy conditions.

Chu et al. [18] primarily aimed at identifying landslide failure and its mechanism along with sliding depth of the slope failure at the Su-Ao-Nan-Ao section of the Su-Hua Highway. The specific region is undertaken for this study because of the very high rainfall intensity in the region. The damage was aggravated during the 2010 Maggie Typhoon that caused havoc all across the country. The factor of safety of the slope is calculated as 0.9 whereas the shear strain distribution is also found to be near critical stage. The sliding depth is determined to be upwards of 15.6m all along the highway section. The study concluded that gully erosion is the primary cause of landslide induced slope failure on this section of the highway. Tao modeled mountain roads in the Yunnan province of western China using MIDAS and stabilized failing slopes using anti-slide piles. The insertion of piles improved the factor of safety from 0.95 to 1.10 and improved the overall safety of the slope [19]. Yijie et al. [20] not only designed geo-grid for slopes but also observed the performance of these geo-grids fixed on sensitive slopes by installing Fiber Bragg Grating sensors on them to observe the strain distribution on the slope during critical conditions. Andreea observed stability of dam under two different water conditions by means of SRM by MIDAS GTS Nx. Maneciu Dam, a 75 meter high earth dam situated on the Teleajen River in Romania is chosen as the study site [21]. 3D analysis using SRM is performed for steady state and transient state and factor of safety is found to be safe. The finite element method represents a powerful alternative approach for slope stability analysis which is accurate, versatile and requires fewer a priori assumptions, especially, regarding the failure mechanism. Slope failure in the finite element model occurs 'naturally' through the zones in which the shear strength of the soil is insufficient to resist the shear stresses. The paper describes several examples of finite element slope stability analysis with comparison against other solution methods, including the influence of a free surface on slope. Graphical outputs are given to illustrate deformations and mechanisms of failure. It is argued that the finite element method of slope stability analysis is a more powerful alternative to traditional limit equilibrium methods and its wide spread use should now be standard in geotechnical practice. In particular, strength reduction method (SRM) can be used to simulate the failure process without any previous assumption. In SRM, shear strength and friction angle gradually decreases until the solution does not converge. The maximum strength reduction ratio at a point is used to calculate the minimum safety factor of the slope.

III. GEOLOGY AND ROCK MASS PROPERTIES

The geology around dam comprises of Thimpu formations of crystalline complex belonging to upper amphibiotite faces of metamorphism. The litho unit at the dam site is characterized by heterogeneous lithology consisting of banded gneisses, biotite gneiss with large boundaries and bands of quartzite and their interlayer of biotite schist. The slope profiles of hill sections at every 1000 m, from 1700 to 18000 m upstream of the dam axis indicated that the slopes vary from steep 1(V) : 0.6(H) i.e. 59^0 to flatter than 1(V) : 4(H) i.e. 14^0 with horizontal. The details of cross sections on the upstream side of the hill profile are given in Table 1. The site exploration work comprised drilling of 9 boreholes, three at each cross section, 50 m deep each, on left bank hill slopes at chainages of 1700, 5000 and 10000 m upstream of dam axis. The locations of exploration holes are shown in Fig.1 and details are given Table 2. The overburden strata consist of weathered and fractured rock fill/gravelly material with thickness varying from 3.5 m to 32.0 m. Rock strata of banded/jointed gneiss underlie the overburden stratum are visible in Fig. 2. On the basis of the bore logs received from the BHEP authorities the core recovery percentage is found to be very poor. The rock samples are of metamorphic rock, which has closely spaced foliation planes. It is seen that the permeability values range from 5 Lugeon to 25 Lugeon, indicating that the rock strata has moderate permeability with presence of some open joints. These samples are splitting easily along foliation planes with application of horizontal loads. The foliations are dipping obliquely into the hills, which is not favorable for slipping towards river. According to the Geomechanics Classifications (Rock Mass Rating system) [22] the rock mass is classified and final rating numbers are determined. The rock mass existing in the area is highly jointed even at deeper depths, which was ascertained from the very low RQD values from the drilled boreholes.

The RMR system is based on the six parameters i.e. uniaxial compressive strength of rock material, RQD, spacing of discontinuities, condition of discontinuities, orientation of discontinuities and ground water conditions. The strength parameters of overburden rock mass from RMR System is found to be c=0.2MPa and $\phi=35^0$. The stability of these hill slopes are studied by adopting c = 0.2MPaand $\phi = 35^0$.

Table1. Details of cross sections on the upstream side of the hill profile

Distance from u/s of dam	Right bank slope 1(V):H	Left bank slope 1(V):H	Distance from u/s of dam	Right bank slope 1(V):H	Left bank slope 1(V):H
1700	0.6 2.7 0.9	0.6 1.1	10000	1.5	1.6
1900	1.2 3.7	0.6	11000	4.0 0.9	1.3
2000	1.1 4.4	0.6	12000	1.7	1.2
3000	1.3 0.7	1.5	13000	1.6 8.6	0.7
4000	2.5 0.6	1.2	14000	1.3	2.2 10.6 8.5 12.3 3.3
5000	1.0	1.7	15000	3.7 0.1	1.0
6000	0.8	1.6 3.0	16000	0.8 2.6	2.0 5.5
7000	1.7	3.1	17000	2.9	2.5
8000	15.2 34.5 7.0 36.6 23.7 4.7	7.8 1.6	18000	5.4	4.5
9000	0.9	5.7			

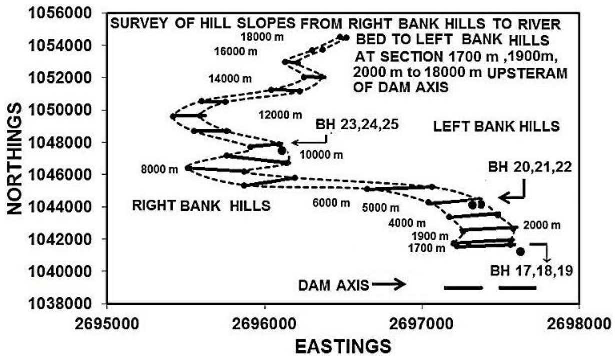


Fig.1. Location of Bore Holes.

Fig. 2 Rock Mass from 2000 to7000m u/s of dam.

Table 2. Location of boreholes

Bore Hole numbers and Locations			Bore hole chainage
Top	Middle	Bottom	
17	18	19	Near 1700 m u/s
20	21	22	Near 5000 m u/s
23	24	25	Near 10000 m u/s

The average values of density, percentage of water absorption, crushing strength, tensile strength are found as 2.71 gm/cc, 0.11%, 54.41 MPa and 5.20 MPa respectively. For determining the strength parameters of rock material of the

confined rock mass, rock core samples are tested for triaxial compression test [23]. The strength parameters ϕ and c for the 23 samples extracted from reservoir rim, after grouping them as per the different ranges of values of elastic modulus of 1.17 to 1.96, 2.85 to 3.94, 4.76 to 7.35, GPa is varying from 51.17 to 68.93^o and 1.21 to 11.08 MPa respectively. Whereas, the strength parameters ϕ and c for the 23 samples, after grouping them depth wise as 0 to 19, 19 to 29, 29 to 39, 39 to 45 m from reservoir rim area is varying from 56.92 to 69.09^o and 0.48 to 7.16 MPa respectively. The average values of ϕ and c which are adopted for slope stability analysis when the rock mass is not exposed and is under confined condition are 59.40^o and 6.12MPa respectively for reservoir rim area.
The Standard Penetration Test values in overburden ranged from 62 to100 blows per 300 mm penetration indicated the average friction angle (ϕ) of the sandy strata to be 40^0 with negligible cohesion. Laboratory Direct shear test results on remolded soil samples, collected from top 2.5 m of overburden, indicated the average shear strength parameters of cohesion (c) and angle of internal friction (ϕ) as 0.02 MPa and 32^0 respectively. THDCL made arrangements to collect the gravelly soil samples of the overburden strata from the pits (9 Nos) of size 1.5 x 1.5 x 2.5m deep. These pits were located near the bore holes. Density of the strata was determined by core cutter. Gradation and Direct shear tests on these samples are done in CWPRS laboratory [24].

IV. METHODOLOGY

4.1. Strength reduction theory

To simulate slope failure using the strength reduction method, the safety factor is computed at an arbitrary point where the Mohr circle is in contact with the failure envelope as shown in Fig. 3. The stress state at this point can be determined as the failure state and when this failure stress increases than the strength, overall slope collapse occurs. The finite element analysis at this limit state diverges and the safety factor at this point is defined as the minimum safety factor.
Stability by strength reduction technique (SRM) is achieved by weakening the soil in an elastic plastic finite element analysis until the slope fails. The Factor of Safety (FOS) is deemed to be the factor by which the soil strength is to be reduced to reach failure. This method automatically satisfies the translational and rotational equilibrium conditions.

Mathematically the factored strength parameter formulation is given below;

$$\frac{\tau}{F} = \frac{c'}{F} + \sigma \frac{tan\phi'}{F} \quad (1)$$

$$\frac{\tau}{F} = C^* + tan\,\phi^* \quad (2)$$

Where $C^* = \frac{c'}{F}$ and $\phi^* = arc\ \tan\sigma\frac{\tan\phi'}{F}$ are the factored strength parameters, C is cohesion, ϕ is angle of friction and σ is normal stress.

4.2. Modeling procedure

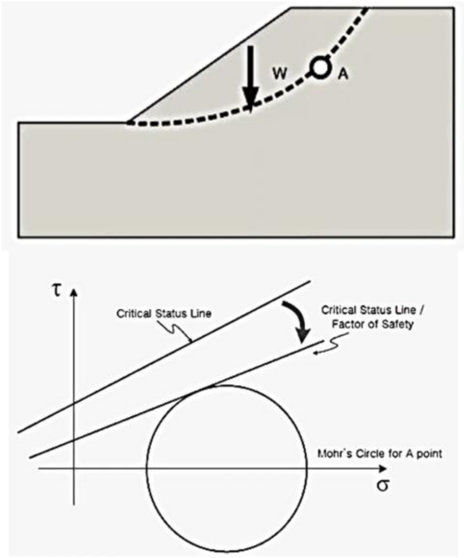
The SRM involves exhaustive computation and accurate results are obtained. Moreover, the transition from initial slope condition to slope failure can be examined without having to assume the failure slope in advance. SRM primarily involves the following three computational processes:

- Depending upon the type of slope, a certain reduction factor (SRF) known as Strength Reduction Factor is triggered while analysis is carried out.
- Multiple iterations are carried by varying the SRF until the soil slope model becomes unstable.
- Thus a critical SRF is determined based on which the factor of safety of the slope is determined.

The SRM adopts Mohr-Coulomb strength for slope stability analysis. This method is most commonly used criteria applicable to failure analysis of slopes in geotechnical engineering. A distinctive feature of this slope failure analysis model is the fact that it can explicitly predict the failure in principal stress space (maximum displacement) and shear normal stress space (maximum shear) to which the given slope is subjected.

4.3. Stability of hill slopes at different sections

Two left banks critical sections of hill slopes i.e. at 1700 m *u/s* and 4000 m *u/s* are studied for the hill slope stability. The thickness of the overburden strata of the slopes is considered as per the details of the bore logs. The analysis is carried out for conditions, before and after impoundment of reservoir, with drawdown condition from FRL (RL 2006 m) to MDDL (RL 1950 m) at drawdown rate of 1m/month **[25]** and also for the dynamic conditions. The earthquake time curve is given in Fig. 4. Fig. 5 gives these hill slope sections considered for the analysis. The properties adopted in the model for the different type of strata are given in Table 3.



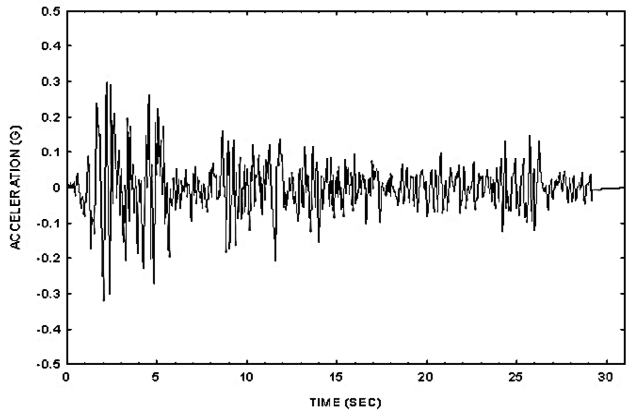
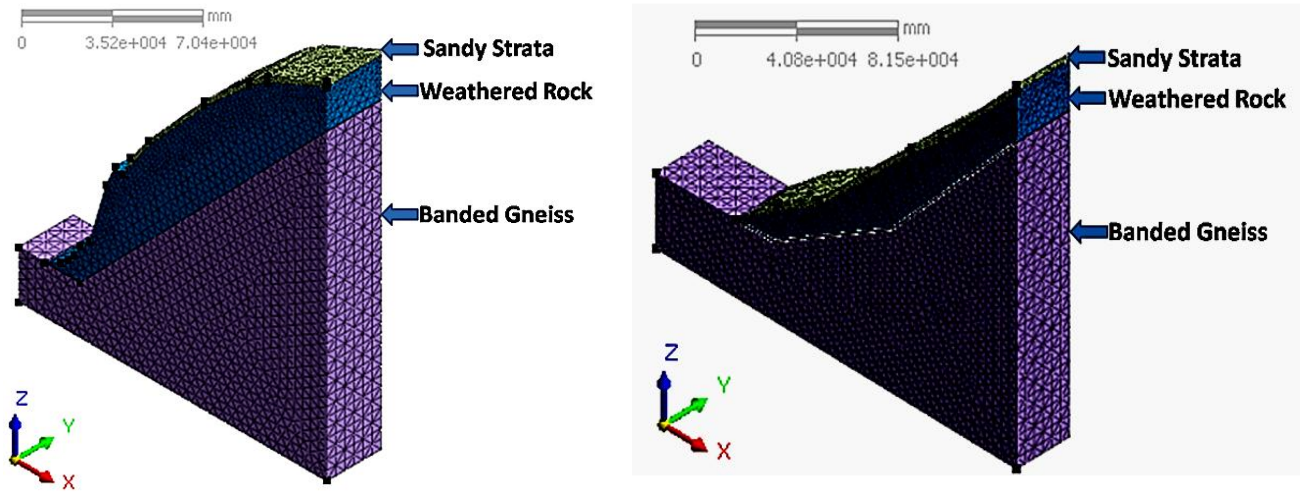


Fig. 3 Slope failure using the strength reduction method.

Fig. 4 Earthquake time curve



(a)

(b)

Fig. 5 Figure shows 3D FEM model of hill slopes of sections at (a)1700 m and (b) 4000 m of u/s, for left bank of dam axis.

Table 3. Properties adopted in the model

Properties	Banded Gneiss	Weathered Rock	Sandy Strata
Mesh Size (m)	5	3	2
Deformation Modulus (GPa)	35	2	0.9
Poisson's ratio	0.2	0.3	0.3
Cohesion (C) MPa	6.12	0.25	0.02
Angle of Friction in (ϕ)	59.4	35	32
Unit weight, (Kg/m^3)	2000	1700	1500
Unit Weight (Sat.) , (Kg/m^3)	2500	2400	2300
Permeability, e^{-007} m/sec	1	5	10

The slope stability analysis was carried out for hill slopes before and after submergence, with and without earthquake loads. The results are given in Table 4.

V. RESULTS

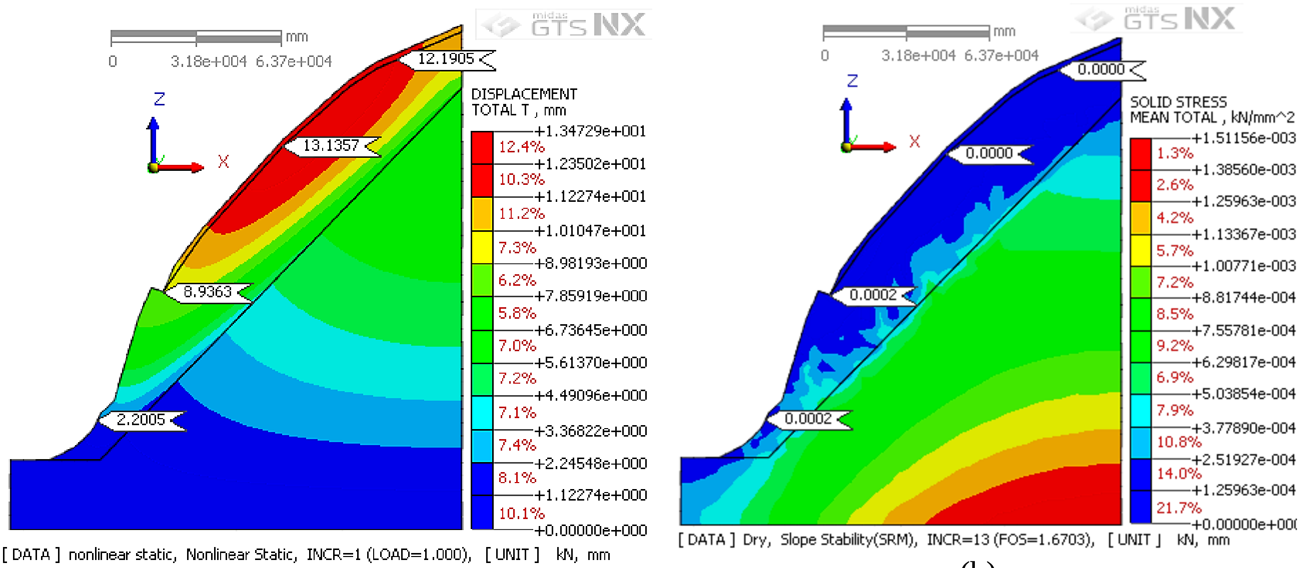
The slope stability analysis was carried out for hill slopes before and after submergence, with and without earthquake loads. The results are given in Table 4.

5.1 Analysis of hill slope at chainage 1700m

Fig. 6 shows (a) total displacement of 13.47 mm, (b) solid stress mean total is 15.12x10^{-4} KN/$mm^2$and FOS is 1.67 in dry and without earthquake condition.

Table 4. Results of slope stability analysis

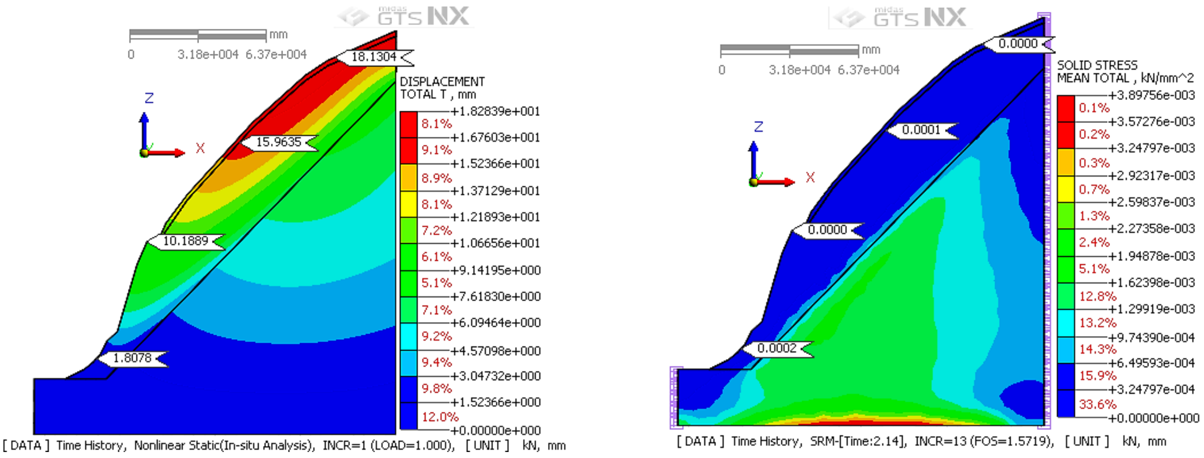
Type of analysis	FOS at 1700m	FOS at 4000m
I. Before Submergence		
w/o Earthquake	1.67	2.5
with Earthquake	1.57	1.5
II.After Submergence		
w/o Earthquake	1.97	2.35
with Earthquake	1.75	1.41
III. Drawdown from RL 2006 to 1956 m		
w/o Earthquake	1.52	1.81



(a) (b)

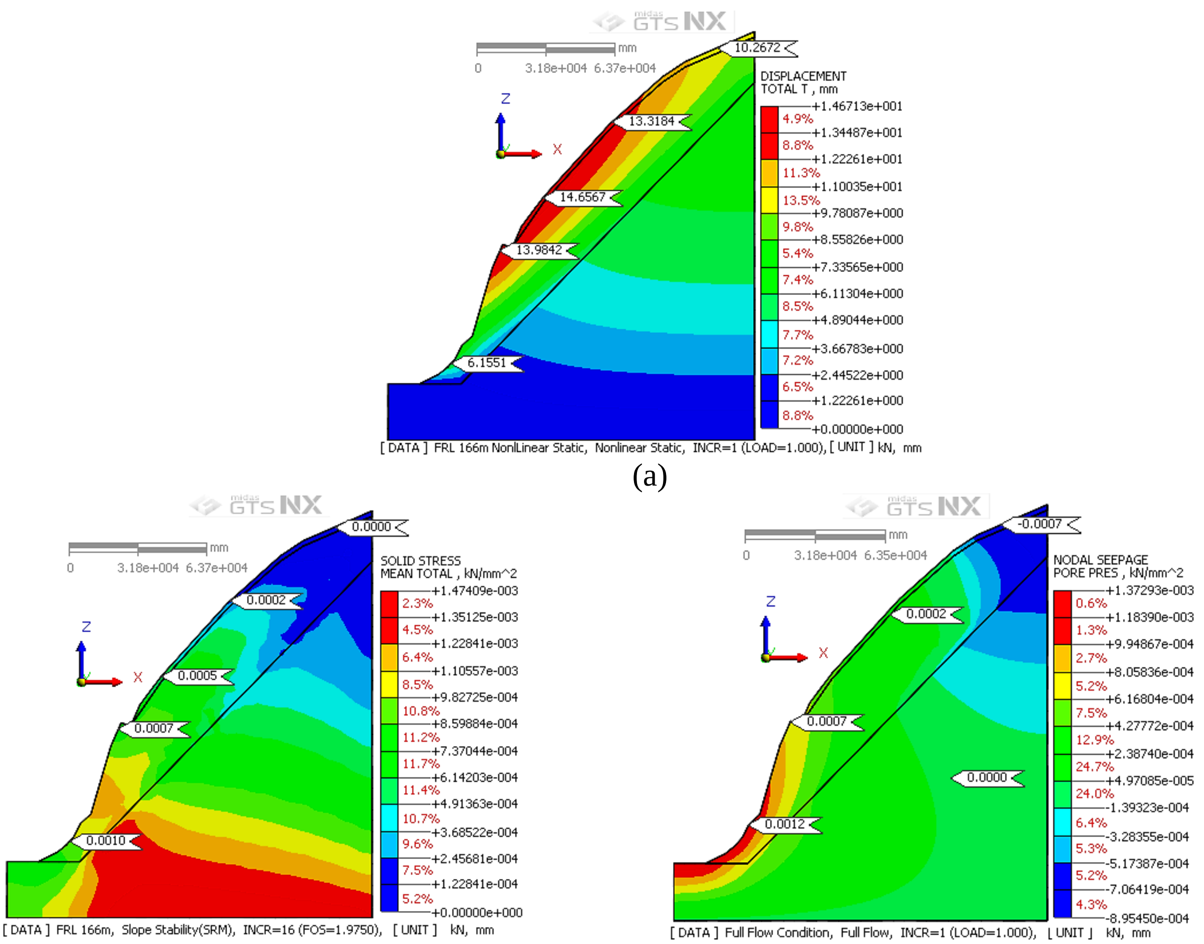
Fig. 6 Figure shows (a) total displacement (b) solid stress mean total of hill slope at chainage 1700 m in dry, without earthquake condition

Fig. 7 shows (a) total displacement, 18.28 mm and solid stress mean total 3.90x10^{-3} KN/mm^2 with FOS 1.57 in dry and earthquake condition. Fig. 8 shows (a) total displacement of 14.67 mm, (b) solid stress mean total 1.47x10^{-3} KN/mm^2, (c) Nodal seepage pore pressure 1.37x10^{-3} KN/mm^2 respectively with FOS 1.97, of hill slope at chainage 1700 m in submerged condition without earthquake. Fig. 9 shows (a) total displacement 17.77 mm and (b) solid stress mean total 2.44x10^{-3} KN/mm^2 of hill slope of at chainage 1700 m with FOS 1.75 in submerged and earthquake condition. Fig.10 (a) solid stress mean total 5.09x10^{-2} KN/mm^2(c) solid stress pore stress is 0.0 KN/mm^2 with FOS 1.519 of hill slope at chainage 1700 m in rapid drawdown @ 1m/day from RL 2006 m to 1950 m without earthquake.



(a) (b)

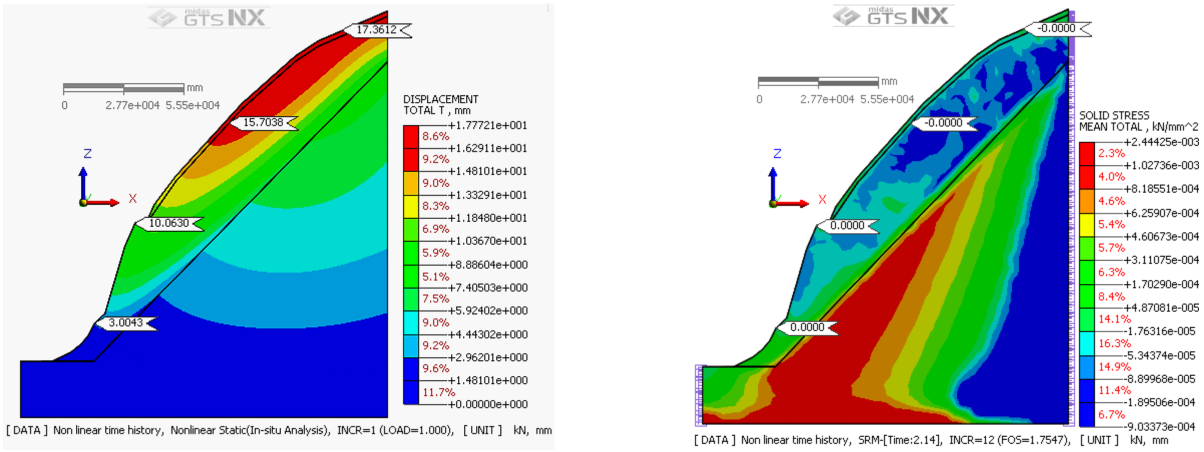
Fig. 7. Figure shows (a) total displacement (b) solid stress mean total of hill slope of at chainage 1700 m in dry, with earthquake condition



(a)

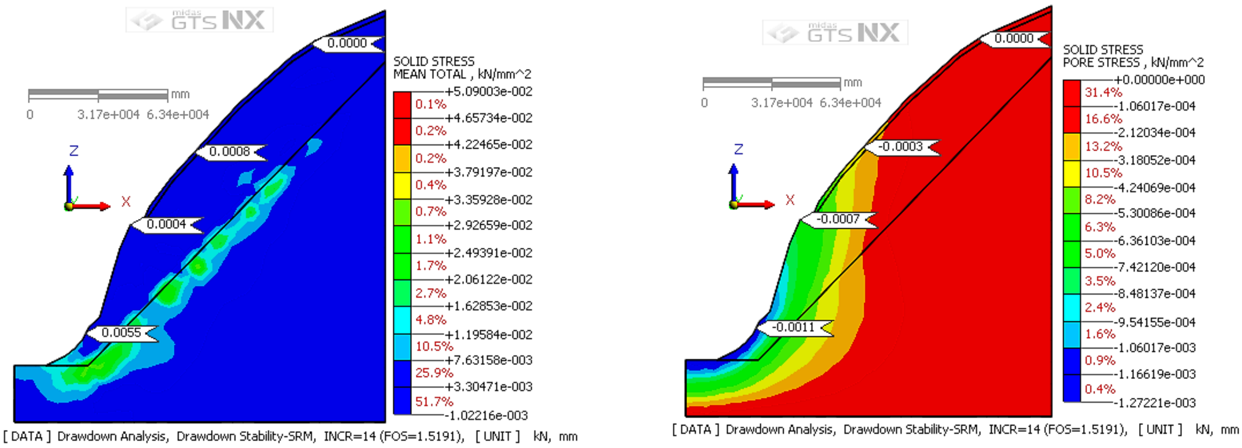
(b) (c)

Fig. 8 Figure shows (a) total displacement, (b) solid stress mean total (c) nodal seepage pore pressure of hill slope at chainage 1700 m in submerged condition without earthquake



(a) (b)

Fig. 9 Figure shows (a) total displacement (b) solid stress mean total of hill slope of at chainage 1700 m in submerged with earthquake condition



(a) (b)

Fig. 10 Figure shows (a) solid stress mean total (b) solid stress pore stress of hill slope at chainage 1700 m in rapid drawdown @ 1m/day from RL 2006 to 1950 m without earthquake

5.2 Analysis of hill slope at chainage 4000 m

Slope section model at chainage 4000 m is shown in Figure 4(b).The slope stability analysis is carried out for the similar conditions as explained for the chainage 1700 m. The results are given in the Table 4.

VI. DISCUSSIONS

The bore holes drilled along three sections, at 1700, 5000 and 10000 m upstream of dam axis, on the left bank hills, showed that the slopes predominantly consist of overburden strata underlain by rock strata of banded gneiss. The Overburden strata consist of weathered, fractured rock/gravelly materials. It is observed that the thickness of the overburden strata varies from 3.5 to 32.0 m at location 1700 m upstream of dam axis and having average thickness of about 23.0 m at 5000 and 10000 m location u/s of dam axis.

The hill slope profile showed that on left bank, from 1700 to 2000 m u/s of dam axis, the slope is 1 (V) : 0.6 (H) indicating steep slopes. At 6000m the slope is 1:0.8 and at 13000 m the slope is 1:0.7, and at other locations the slopes are flatter than 1:1. On the right bank, the steeper slopes are seen at 1700, 4000, 6000, 9000, 11000 and 16000 m u/s of dam axis.

The slope stability analysis indicated that left bank sections at 1700 and 4000 m, u/s of dam axis have safe factor of safety against failure for different loading conditions.

The analysis assumes that the pore pressure is not dissipated during the drawdown condition. However, it is mentioned that overburden material consists of 50% of gravel and remaining sand with little silt. The permeability of the material of such gradation is estimated to be greater than 10^{-2} cm/sec, indicating overburden to be of pervious strata. Moreover, it is seen from bore-hole logs that there is a complete loss of drill water in overburden strata, indicating that the overburden is highly pervious. As such, there will be no pore pressure in the overburden strata during drawdown and the factor of safety will be higher.

VII. CONCLUSIONS

i. The slope stability analysis of left bank hill section at 1700 m upstream of dam axis, showed a minimum factor of safety of 1.52 for the most critical condition of drawdown and earthquake.

ii. The slope stability analysis of left bank hill sections at 4000 m, showed values of Factor of Safety more than required, under different loading conditions.

iii. The slope sections at other chainages i. e. from 4000 to 10000 m are flatter than the slope sections at 1700 and 4000m chainages. As the strata are same as of ch.1700 and 4000m, so it is concluded that the slopes on left and right bank of the river are safe.

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REFERENCES

[1] Bishop AW, “The use of the slip circle in the stability analysis of slopes”, Geotechnique, vol. 5, pp. 7-17, 1955.
[2] Deng Dong-ping, Li Liang, Wang Jian-feng, Zhao Lian-heng, “Limit equilibrium method for rock slope stability analysis by using the Generalized Hoek–Brown criterion” International Journal of Rock Mechanics and Mining Sciences, vol. 89, pp. 176–184, 2016.
[3] Saade A, Abou-Jaoude G, Wartman J, “Regional-scale co-seismic landslide assessment using limit equilibrium analysis”, Engineering Geology, vol. 204, pp. 53–64, 2016.
[4] Yao Y-P, Zhou A-N, Lu D-C, “Extended transformed stress space for geomaterials and its application”, Journal of Engineering Mechanics, vol.133, no.10, pp. 1115–23.
[5] Mahtab MA, Goodman R, “Three dimensional finite element analysis of jointed rock”, International Society of Rock Mechanics Proceedings, vol. 3, no. 7-12, pp. 353-360, 1970.
[6] Hammah RE, Curran JH, Yacoub T, Corkum B, “Stability analysis of rock slopes using the finite element method” Proceedings of the ISRM regional symposium EUROCK. 2004, Citeseer.
[7] Shen J, Karakus M, Xu C, “Chart-based slope stability assessment using the Generalized Hoek-Brown criterion”, International Journal of Rock Mechanics and Mining Sciences, vol. 64, pp. 210–9, 2013
[8] Lyamin A, Sloan S, “Lower bound limit analysis using non-linear programming”, International Journal of Numerical Methods in Engineering, vol. 55, no. 5, pp. 573–611, 2002.
[9] Lyamin AV, “Sloan S. Upper bound limit analysis using linear finite elements and non- linear programming” Int J Numer Anal Meth Geomech,vol. 26, no. 2, pp. 181–216, 2002.
[10] N. Halakatevakis, A. I. Sofianos, “Correlation of the Hoek-Brown failure criterion for a sparsely jointed rock mass with an extended plane of weakness theory”, International Journal of Rock Mechanics and Mining Sciences, vol. 47, no. 7, pp. 1166–1179, 2010.
[11] E. Hoek, “Estimating Mohr-Coulomb friction and cohesion values from the Hoek-Brown failure criterion”, International Journal of Rock Mechanics and Mining Sciences and Geomechanics Abstracts, vol. 27, no. 3, 227–229, 1990.
[12] Fredlund D.G., Krahn J, “Analytical methods for slope analysis”, International Symposium on Landslides, pp. 229-250, 1984.
[13] Duncan JM, “State of the art, limit equilibrium and finite-element analysis of slopes”, Journal of Geotechnical Engineering, vol. 122, no. 7, pp. 577–96, 1996.
[14] Griffiths DV, Lane PA, “Slope stability analysis by finite elements”, Geotechnique, vol. 49, no. 3, pp. 387–403, 1999.
[15] E.M. Dawson, W.H. Roth, A. Drescher, “Slope stability analysis by strength reduction”, Geotechnique, vol. 49, no. 6, 835-840, 1999.
[16] Hsiung, B.C.B, Tsai, Y.Y, Tsai, C.C, “Analysis and construction of cross passage of Delhi Metro”, Proceedings of Indian Geotechnical Conference 2010, IGS Mumbai Chapter & IIT Bombay, GEOtrendz.
[17] R. Saikia, A. K. Dey, “Slope stability analysis of slides at Sonapur using strength reduction method” Proceedings of Indian Geotechnical Conference IGC 2016, 15-17 December 2016 IIT Madras, Chennai, India.
[18] Huang-Kuei Chu, Chia-Ming Lo, Yu-Lin Chang, “Numerical analysis of slope stability at the 115.9k Point of the Su-Hua highway”, Journal of Chinese Soil and Water Conservation, vol. 44, no. 2, 97-104, 2013.
[19] Xiang Tao, “The application and analysis of Midas/GTS in landslide control”, International Journal of Engineering Research, vol. 4, no. 8, pp.409-411, 2015.
[20] Sun Yijie, Hongzhong Xu, Peng Gu, Wenjie Hu, “Application of FBG sensing technology in stability analysis of Geogrid-Reinforced slope”, Sensors, vol. 17, no. 3, pp. 597, 2017.
[21] Carastoian Andreea, “Unsaturated slope stability and seepage analysis of a dam, Sustainable solutions for energy and environment”, EENVIRO - YRC 2015, Bucharest, Romania.
[22] Bieniawski Z.T. “Engineering classification of jointed rock masses”, Transactions of the South African Institution of Civil Engineers, vol. 15, no. 12, pp. 335 – 344, 1973.
[23] IS 13047 (1991): Method of determination of strength of rock materials in triaxial compression.

[24] IS 2720 (1986, Part 13): Method of tests of soil – Direct shear tests.
[25] IS 7894(1975): Code of practice for stability analysis of earth dams.