Indian Geotechnical Conference **IGC2016**
15-17 December 2016, IIT Madras, Chennai, India

ROCK MASS SLOPE STABILITY ANALYSIS UNDER STATIC AND DYNAMIC CONDITIONS IN MUMBAI, INDIA

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ABSTRACT: The rock slope stability has gained much importance in the recent years owing to the development of infrastructure in/on rock slopes areas in the form of high rise building, roads, railways, dams and other rock engineering projects. Rock slope stability is generally affected by the property of rock material along with the discontinuities in the rock mass. It is also dependent on slope angle, slope height, surcharge, ground water conditions, rain fall, and dynamic forces like earth quake. If this factors coupled with deep weathering of the rock mass, complicates the entire behavior of the rock slope and makes the analysis of rock slopes challenging task.

For the present work; field work was carried out for joint mapping on differentially weathered volcanic rock of selected slope sections at Mumbai in Deccan trap regions. Block and core samples were collected from each category of the rocks and their respective geotechnical properties measured experimentally and presented her in. The rock slope stability analysis was done using numerical modelling techniques; Finite Element Method (FEM) based two dimensional analyses (RS2) or Phase2 9.0 software from Rocscience. The rock slope having four different quality of rock mass was analyzed under static and pseudo-static conditions. Finally, the optimum support measures were provided to the slope for stabilizing it and the corresponding analysis was also done.

KEY WORD: Rock slope stability, rock mass properties, numerical modeling, RS2, stabilization

1 INTRODUCTION

Rock slope stability among many factors generally depends on the strength of the rock materials and characteristics of discontinuities (e.g. roughness, wall strength, aperture, fill material and persistence). In tropical climate, weathering of rock material and discontinuity walls also affect the rock slope stability and significantly influences the response of rock masses to loadings and unloading of excavations.

In recent years, numerical methods such as Finite Element Method (FEM), Finite Difference Method (FDM), and Discrete Element Method (DEM) are being more commonly applied to slope stability problems. Hudson et al., (2007) provided a flow chart for numerical modeling of rock engineering. It is a common practice to model the behaviour of discontinuities by a linear Coulomb relation using the parameters c and ϕ. However, the researchers in the field of rock mechanics and rock engineering well recognized that the shear strength parameters are not truly constant, it depends upon the normal stress and scale (Bhasin and Kaynia, 2004).

Hoek et al., 2002 has given c and ϕ in terms of Hoek parameters particularly to solve rock slope stability problems. For the present analyses and design of rock slopes different degree of weathered blocks and core samples were collected from the rock slope of Deccan trap region, India. Specimens were prepared as per ISRM standards and tested for tensile, compression and triaxial strengths. The average UCS and material constant of each weathered rock material was determined from experimental results. The GSI value of each of weathered rock mass was used for the estimation of rock mass properties (i.e. equivalent rock mass properties) for the analysis and design of slopes.

2 GEOLOGICAL DESCRIPTION AND SPECIMEN PREPARED

The rock slope is located at the eastern scarp of Cumballa Hill in Tardeo, Mumbai. Regionally the Rocks in Maharashtra and around Mumbai are of volcanic origin known as Deccan trap. The Deccan trap is composed of very hard, tough and compact rocks. The rocks are susceptible to weathering which begins on the exposed surface and along joints, cracks and fissures. With time it penetrates deep into the rock masses and resulting in complete loss of original skeleton structure of the rock mass. Endalu et al., (2016) studied the strength properties of different degree of weathered basaltic rocks of some locations in Deccan trap and observed that weathering has drastically reduced the strength of rocks. They found the strength of slightly weathered basalt to be 50% less than the fresh basalt strength.

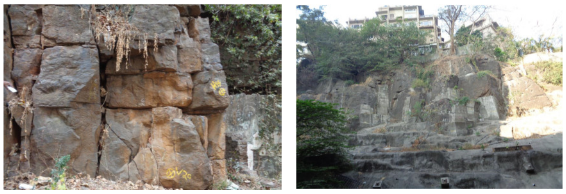

Fig. 1 The present Rock slope appearance on out crops from eastern to western corner of the slope (a) dominant joint sets, discoloration and staining of rock material due to weathering clearly visible (b)already stabilized portion of the rock slope.

Fig. 2 Specimen prepared for testing under tensile, uniaxial and triaxial loading.

For the present rock slope stability analysis efforts were made to incorporate the effect of chemical weathering and discontinuity characteristics on rock mass through detail field work such as joint mapping and laboratory investigations. Summary of the geotechnical properties of rocks are presented in Table 1 and 2.

3 MAKING SECTIONS FROM FENCE DIAGRAM

Five sections of the hill slope were plotted for chainage 15 m, 35 m, 65 m, 90 m and 110 m. After studying the core samples were obtained from already drilled borehole (22.5m to 50m deep) below Existing Ground Level (EGL). From the borehole data, the rock present at various depths of the slope in a borehole of a particular section is marked and joined with the same rock in another borehole of the same section, to get the inclination of various rock strata beneath the EGL of the hill slope.

From the borehole study, it was found that Basalt forms the top strata of the hill slope, as was observed on the site. Below the Basalt layer is the layer of fine volcanic breccia (FVB) which forms the lower portion of the hill slope, it is followed by a layer of coarse volcanic breccia (CVB) and then welded tuff (Fig. 3). The angle of the slope varies throughout the slope section and the average angle of the slope was found to be 30°.

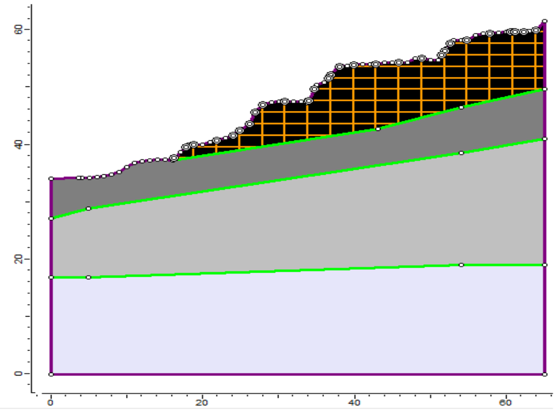

Fig. 3 Geometry and formation of the four rock mass at 65 m chainage.

4 DETERMINING ROCK MASS PROPERTIES

Rock material properties were obtained from laboratory testing including index properties tests, uniaxial, triaxial, Brazilian and point load test. The joint properties were calculated using direct shear test.

For the top Basalt layer, rock material property (σ_c =68.85 MPa and E =34.937 GPa) and joint property had been calculated in the laboratory and used in the analysis given (Tables 1 & 2). Normal stiffness and shear stiffness were taken from (Kainthola et al., 2014).

Since none of the under laying strata was exposed for the joint mapping, the equivalent rock mass properties were calculated based upon GSI Index chart (Hoek & Marinos, 2000).

The structure of the rock mass (intact, blocky, very blocky, disturbed, disintegrated and laminated) was studied along with the surface condition of the rock mass. Finally GSI was quantified accordingly for Fine Volcanic Breccia (FVB), Coarse Volcanic Breccia (CVB) and Welded Tuff (Tuff) as 20, 40 and 40 respectively.

Rock data 5.0 software from Rocscience was used to determine the equivalent rock mass properties. The triaxial test data was given as an input to the software along with GSI value and Uniaxial Compressive test results (σ_c = 26.1, 33.3, 32.4 MPa and E =11.32, 3.22 and 5.58 GPa) for FVB, CVB and Tuff, respectively.

Also, the residual values of cohesion and friction angle were taken as 10% and 50% of the peak values, respectively for the slope stability analysis. The intact and equivalent rock-mass properties are tabulated in Tables 1 and 2.

Table 1 Basalt intact rock properties and equivalent rock mass Properties of Fine Breccia, Coarse Breccia and Tuff

Properties	Fine Breccia	Coarse Breccia	TUFF	Basalt
GSI	20	40	40	Intact
γ_{dry} (KN/m^3)	21.79	21.50	20.56	27.91
γ_{sat} (KN/m^3)	22.39	22.740	22.40	28.21
σ_t (MPa)	0.005	0.007	0.01	19.670
E (MPa)	517.14	514.0	892.1	34937
c Peak (MPa)	0.812	1.152	1.62	5.365
c Residual (MPa)	0.0812	0.1152	0.016	0.5365
$\varphi^{(\circ)}$peak	23.92°	41.51°	38.3°	65.7°
$\varphi^{(\circ)}$Residual	12°	20°	19°	32.53°

Table 2 Joint Property of Basalt

Joint Property		Value
c (Saw cut)	(MPa)	0.0397
$\varphi^{(\circ)}$ (saw cut)		36.479°
Normal Stiffness	(MPa/m)	29700
Shear Stiffness	(MPa/m)	12270

5 SLOPE STABILITY ANALYSIS

The geometry was made for different slope sections at respective chainage and each rock strata in every slope section was assigned with corresponding rock mass and joint properties accordingly in the Phase^2 model. Then the boundary conditions were assigned to the model.

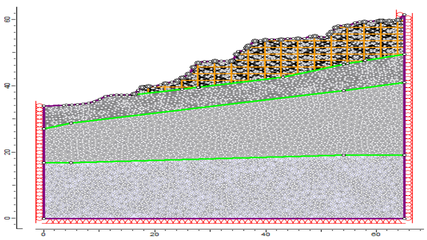

Fig. 3 Discretized phase2 model of slope Section at 65m chainage.

The ground surface of the slope was made free in both X and Y direction. The vertical boundaries of the section was free to move in Y direction only and restrained in X direction. The base of the section of slope was fixed and was restricted to move in both X and Y direction. A uniform mesh was used in the FEM model for the analysis of all the five slope sections.

5.1 Static Analysis

Model of the slope sections were loaded with field stresses by taking gravitational load from actual ground surface. All the analyses were done using Mohr Coulomb Failure Criterion.

Table 3 Results of the static Analysis

Chainage of the slope section, m	Factor of safety	Max. total displacement, m
15	2.65	0.0073
35	1.89	0.0027
65	1.75	0.0019
90	2.85	0.0079
110	2.34	0.0024

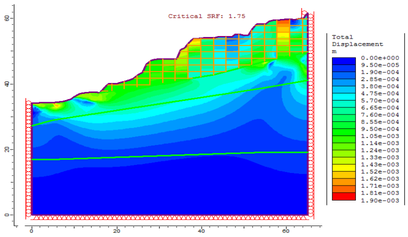

Fig. 4 Total displacement contour plot at critical SRF of section at 65 m chainage

The critical SRF for the slope section at 65 m chainage was found to be 1.75 from the normal static gravitational analysis without any seismic conditions.

5.2 Dynamic Analysis

The present rock slope is located at Mumbai which falls under seismic zone 3 for which the horizontal seismic coefficient value (K_h) is prescribed as 0.16 for moderate seismic intensity (I.S.1893.1.2002). Numerical Analysis was done under pseudo static loading using the horizontal seismic coefficient to model their stability under any seismic event. From static analysis, it has been concluded that the slope section at chainage 65 m has the minimum factor of safety. In this section , this slope section at 65m chainage, has been analyzed under different pseudo-static loading conditions along with the gravitational forces using FEM based Phase2 software.

Table 4 Factor of safety of the slope section with corresponding K_h value

Horizontal Seismic Coefficient (K_h)	Factor of Safety
0 (Static condition)	1.75
0.03	1.54
0.08	0.99
0.1	0.94
0.16	0.48

The pseudo-static analysis was carried out for 4 cases with different seismic loading conditions along with the gravitational load, for the stability analysis of the hill slope at 65 m chainage. The result of the analysis is tabulated in Table 4.

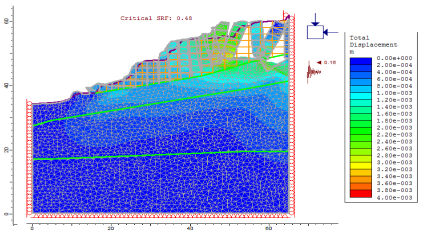

Fig. 5 Max. Deformed shear strain plot of 65 m chainage section with $K_h = 0.16$

6 STABILIZATION OF THE SLOPES

From the above analysis it is clear that the slope is failing at K_h=0.16 as prescribed by I.S. 1893.1.2002 for seismic zone 3 areas. Hence, there was a need to provide optimum support system in order to stabilize the slope. The hill slope at 65 m chainage was therefore stabilized with shotcrete and fully bonded bolts so that the factor of safety can be reached greater than one in dynamic condition. Various trials had been done with different reinforcement to stabilize the slope at the corresponding loading condition.

The optimum support measures were provided using shotcrete (f_{ck} = 35MPa and 100mm thickness) and fully bonded bolts (32mm Diameter, 10m length with 3x3m spacing) due to which the Factor of Safety of the most critical section(at 65 m chainage) was raised from 0.48 to 1.2 at pseudo-static and gravitational loading conditions. The same support was provided to all the sections and the corresponding FOS were 1.82, 1.61, 1.2, 2.5 and 2.44 for slope sections at chainage 15 m, 35 m, 65 m, 90 m and 110 m respectively.

7 CONCLUSION

The field work was carried out at the hilly site of Cumballa Hill in Tardeo, Mumbai, to get the joint mapping and other site parameters. Four different types of borehole rock samples were collected from various already drilled boreholes drilled 22.5 to 50 m below the Existing Ground Level (EGL).

The laboratory testing was carried out to get the physical and strength parameters of the rock material. The summary of geotechnical properties is given in Table 1 and 2. Then the borehole data was studied and the slope sections were prepared at chainage 15 m, 35 m, 65 m, 90 m, and 110 m. Since, the joint pattern in the bottom three rock layers was not exposed, the rock mass property was estimated from the structure and surface condition of the rock mass using GSI Index chart.

From the numerical modeling done using RS2 software on all the prepared sections for gravitational loading in dry condition, it was found that the slopes are in stable condition. The slope section at chainage 65 m was found to have the lowest Factor of Safety = 1.75. This slope section was further analyzed under pseudo-static loading along with gravitational load. It was found that the slope comes in critical condition at horizontal seismic coefficient (K_h) = 0.08. Since, the site is located at Mumbai which comes under seismic zone 3 area, hence the slope was analyzed for K_h = 0.16 as recommended by I.S.1893.1.2002. The slope was found to be failing with factor of safety = 0.48. Hence, there was a need of providing support measures for slope stabilization. The optimum support measures were provided using shotcrete (f_{ck} = 35MPa and 100mm thickness) and fully bonded bolts (32mm Diameter, 10m length with 3x3m spacing) due to which the factor of safety for the most critical section (at 65 m chainage) was raised from 0.48 to 1.2 at the dynamic loading conditions. The same support was provided to all the slope sections and all the corresponding factor of safety was found to be 1.82, 1.61, 1.2, 2.5 and 2.44 at chainage 15 m, 35 m, 65 m, 90 m and 110 m respectively.

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