Rock Mass Slope Stability Analysis Under Static and Dynamic Conditions in Mumbai, India

Ahmed Shaz, Endalu Tadele Chala and K. Seshagiri Rao

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, groundwater conditions, rainfall, and dynamic forces like earthquake. If these factors, coupled with deep weathering of the rock mass, complicate the entire behavior of the rock slope and makes the analysis of rock slopes challenging task. For the present work, fieldwork 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 modeling 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.

Keywords Rock slope stability · Rock mass properties · Numerical modeling RS2 · Stabilization

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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 affects 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 and Feng (2007) provided a flowchart for numerical modeling of rock engineering. It is a common practice to model the behavior 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, and 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. Chala 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 (Figs. 1 and 2).

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 fieldwork such as joint mapping and laboratory investigations. Summary of the geotechnical properties of rocks is presented in Tables 1 and 2.

Fig. 1 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

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
σt (MPa)	0.005	0.007	0.01	19.670
E (MPa)	517.14	514.0	892.1	34,937
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)	29,700
Shear stiffness (MPa/m)	12,270

3 Making Sections from Fence Diagram

Five sections of the hill slope were plotted for chainage 15, 35, 65, 90, and 110 m. After studying, the core samples were obtained from already drilled borehole (22.5–50 m 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).

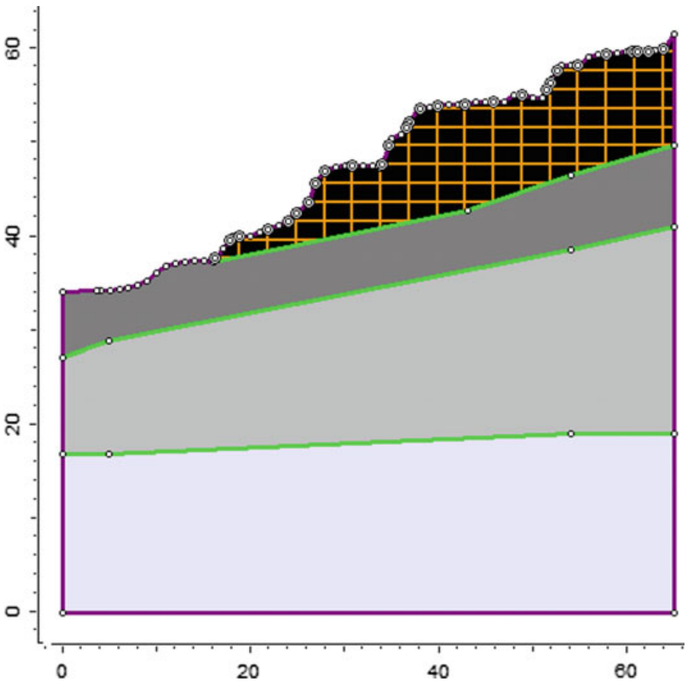

Fig. 3 Geometry and formation of the four rock masses at 65 m chainage

The angle of the slope varies throughout the slope section, and the average angle of the slope was found to be 30°.

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 ($\sigma_c = 68.85$ MPa and $E = 34.937$ GPa) and joint property had been calculated in the laboratory and used in the analysis given (Tables 1 and 2). Normal stiffness and shear stiffness were taken from Kainthola et al. (2014).

Since none of the underlaying strata was exposed for the joint mapping, the equivalent rock mass properties were calculated based upon GSI Index chart (Marinos and Hoek 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 ($\sigma_c = 26.1$, 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 provided in Tables 1 and 2.

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 Phase2 model. Then the boundary conditions were assigned to the model (Fig. 4).

The ground surface of the slope was made free in both *X*- and *Y*-direction. The vertical boundaries of the section were 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.

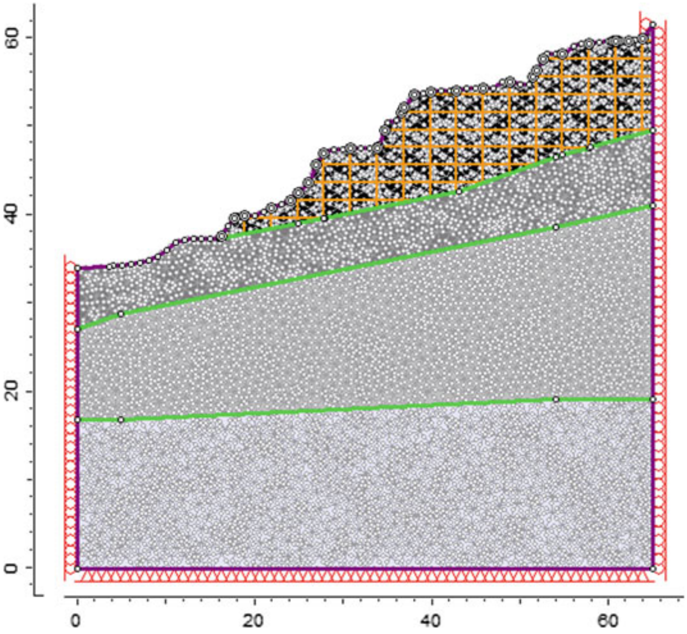

Fig. 4 Discretized Phase2 model of slope section at 65 m chainage

5.1 *Static Analysis*

Model of the slope sections was loaded with field stresses by taking gravitational load from actual ground surface. All the analyses were done using Mohr–Coulomb failure criterion (Table 3).

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 (Fig. 5).

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

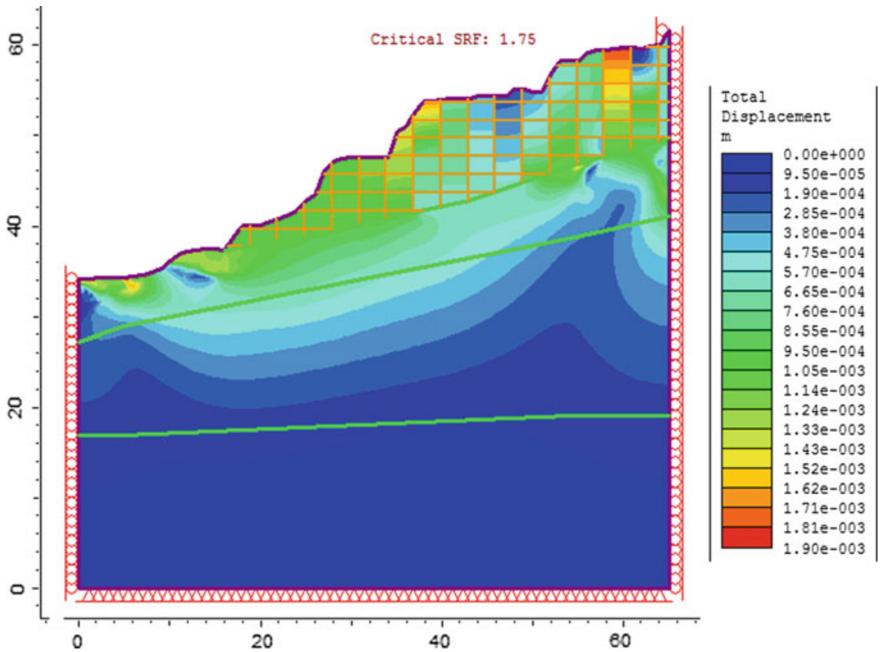

Fig. 5 Total displacement contour plot at critical SRF of section at 65 m chainage

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 65 m chainage has been analyzed under different pseudo-static loading conditions along with the gravitational forces using FEM-based Phase2 software (Fig. 6).

The pseudo-static analysis was carried out for four 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 provided in Table 4.

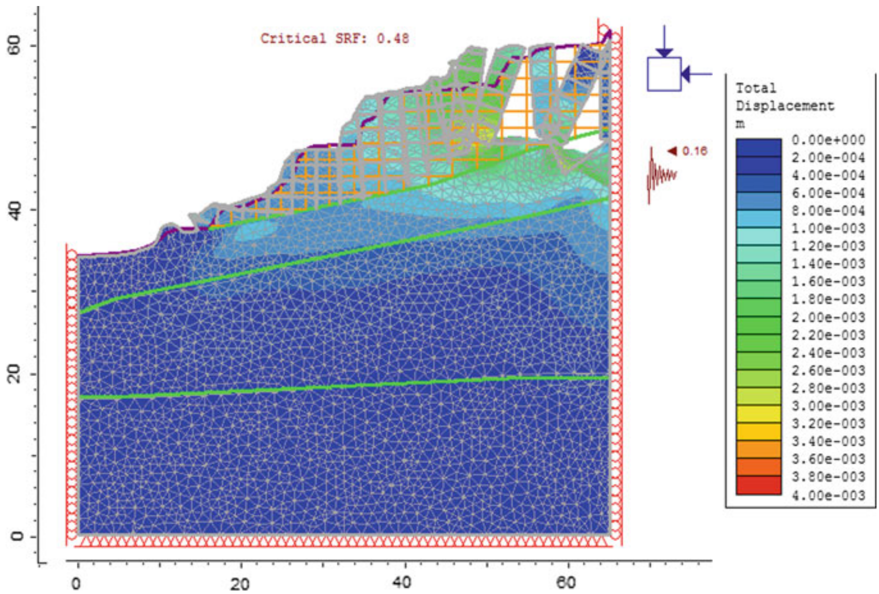

Fig. 6 Maximum deformed shear strain plot of 65 m chainage section with K_h = 0.16

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

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} = 35 MPa and 100 mm thickness) and fully bonded bolts (32 mm Diameter, 10 m length with 3 × 3 m 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 was 1.82, 1.61, 1.2, 2.5, and 2.44 for slope sections at chainage 15, 35, 65, 90, and 110 m, respectively.

7 Conclusion

The fieldwork 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 22.5–50 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 Tables 1 and 2. Then the borehole data was studied, and the slope sections were prepared at chainage 15, 35, 65, 90, 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} = 35 MPa and 100 mm thickness) and fully bonded bolts (32 mm diameter, 10 m length with 3×3 m 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, 35, 65, 90, and 110 m, respectively.

References

Bhasin, R., & Kaynia, A. M. (2004). Static and dynamic simulation of a 700-m high rock slope in western Norway. *Engineering Geology, 71*, 213–226.

Chala, E. T., Seshagiri Rao, K., & Malik, A. (2016). Estimating engineering properties of weathered rocks from Deccan trap. In *Indorock 2016 Proceedings, 6th Rock conference,* IIT Bombay, June 17–18.

Hoek, E., Carranza-Torres, C., & Corkum, B. (2002). Hoek-Brown failure criterion-2002 edition. In *Proceedings of NARMS-Tac* (Vol. 1, pp. 267–273).

Hudson, J. A., & Feng, X. T. (2007). Updated flowcharts for rock mechanics modelling and rock engineering design. *International Journal of Rock Mechanics and Mining Sciences, 44*, 174–195.

Kainthola, A., Singh, P. K., & Singh, T. N. (2014). Stability investigation of road cut slope in basaltic rock mass, Mahabaleshwar, India. *Geoscience Frontiers, 6*(6), 837–845.

Marinos, P., & Hoek, E. (2000, November). GSI: A geologically friendly tool for rock mass strength estimation. In *ISRM International Symposium*.