Rock Slope Stability Analysis of a Metro Station Excavation

R Rajaraman and Gundeti Sumanth Kumar

1 Introduction

In the present-day of urban transportation infrastructure, demand for utilization of underground space surges at a high speed. Construction of stations in metro projects in urban environments involves huge challenges to the safety and creates an impact on adjacent structures. In a metro project in southern part of India, excavation of a station involves deep rock excavation of around 20 m. A vertical cut was proposed without any bench due to space constraints imposed on the area by adjacent major buildings and structures. All types of failure likely to occur for rock slopes were considered for analysis. Analysis has been conducted using empirical, analytical, and numerical methods so that the most suitable support system is recommended with application of adequate engineering judgement.

2 Site Description and Geology

The excavated bedrock was of fresh and hard strong to very strong granitic gneiss. The discontinuities were tight in nature with low persistence making lower portion of the excavation stable and massive in nature. Damp to moist conditions of seepage were noticed mainly. Outcrops were assigned Rock Mass Rating (RMR) of 42–79. The outcrops observed during excavation are presented below (Fig. 1).

R. Rajaraman · G. S. Kumar (✉)
EDRC Metro, Heavy Civil Infrastructure, L&T Construction, Chennai, Tamil Nadu, India
e-mail: sumanth-gundeti@lntecc.com

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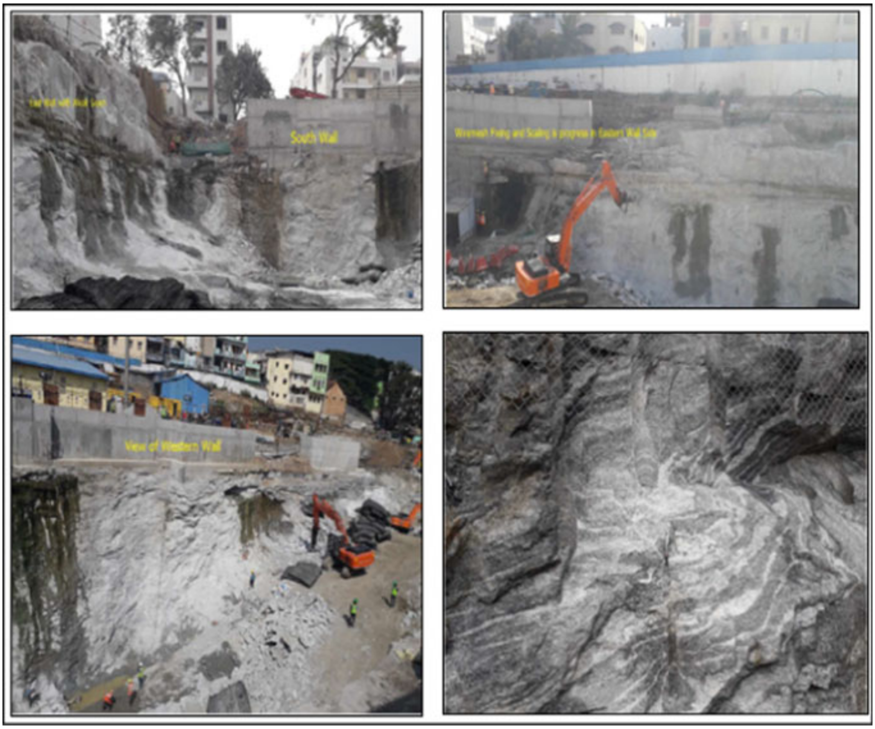

Fig. 1 Outcrops observed during excavation

2.1 Major Discontinuities

The discontinuity data collection has been carried out for the rock outcrops, and joints considered for kinematic analysis are presented in Table 1. All these joints are not discreet and distinct. For a conservative estimation, all these joints have been considered for kinematic analysis.

3 Geotechnical Parameters for Analysis and Design

3.1 Rock Mass

Hoek–Brown strength criterion is used to determine the principal stress and normal-shear strength plots for the rock mass at various depths. The equivalent Mohr–Coulomb parameters have been calculated by fitting the linear Mohr–Coulomb relationship. Typical plots derived by RocData software for one such case is presented in Fig. 2.

Table 1 Details of joints mapped at site

Sl. No.	Joint No.	Dip (degrees)	Dip Direction (degrees)
1	J1	20	360
2	J2	80	40
3	J3	18	180
4	J4	80	310
5	J5	65	230
6	J6	70	135
7	J7	30	100
8	J8	85	110
9	J9	35	310
10	J10	45	125
11	J11	30	110
12	J12	80	10
13	J13	80	210
14	J14	29	102
15	J15	25	320
16	J16	85	210

Table 2 Summary of geotechnical parameters of rock mass

Case No	Height of rock (m)	GSI	UCS (MPa)	Cohesion (kPa)	Friction angle (degrees)	Modulus of Deformation (MPa)
1	5	15	33	31	45	418
2		30	75	81	60	1416
3		40	162	193	67	4952
4	10	15	33	49	40	418
5		30	75	121	56	1416
6		40	162	252	64	4952
7	15	15	33	63	37	418
8		30	75	155	53	1416
9		40	162	304	62	4952
10	20	15	33	76	35	418
11		30	75	186	51	1416
12		40	162	353	60	4952

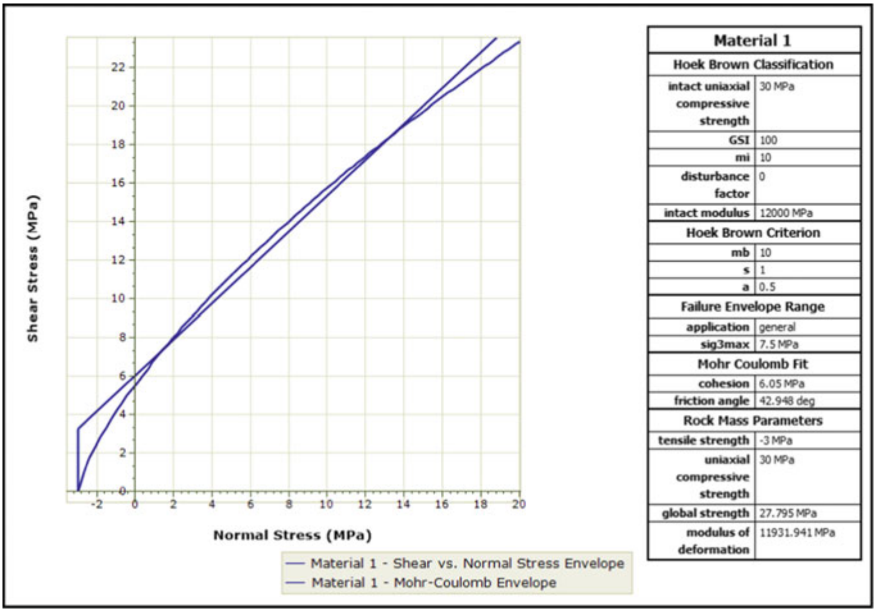

Fig. 2 Typical plot derived by RocData software

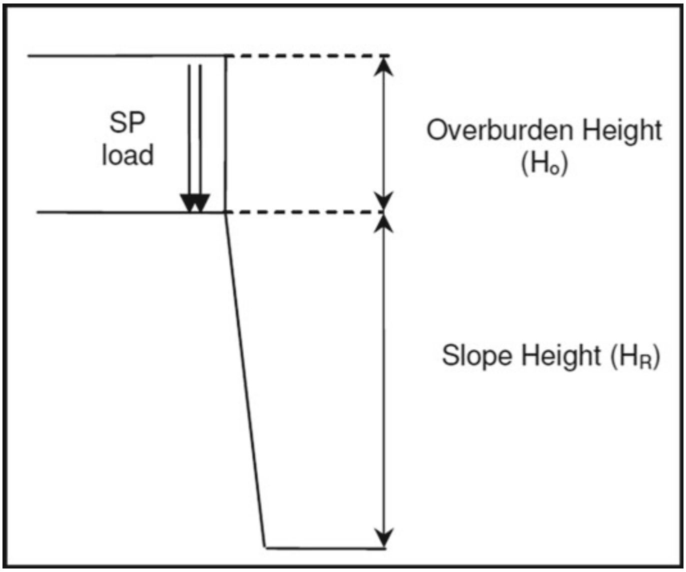

Fig. 3 Idealised Model of Slope

Analyses have been carried out for three rock mass classes represented by GSI ranges; GSI > 40; 20 < GSI < 40 and GSI < 20. These class ranges represent the possible conditions of rock mass likely to be encountered at site. The geotechnical parameters as estimated are presented in the following (Table 2).

3.2 Rock-Rock Joints

Shear strength parameters of rock-rock joints are to be assigned empirically in the absence of field test results. The joint characteristics have been defined by the geologist during site mapping. Accordingly, shear strength parameters of the rock-rock joint in accordance with IS 13365 Re-excavation [1] and Hoek [2] are assigned as below in Table 3.

4 Design Methodology

Analysis of rock slope stability and design of support system will only be holistic when there is a proper link between design and geological assessment at the site. The design must cater to all types of rock mass classes likely to be encountered at site. The flexibility of selection of support system must be entrusted upon the site geologist. Considering this, slope support system has been designed for three rock mass classes characterized by GSI. Analyses have been carried out for various ranges of overburden height and height of rock slope for each GSI range to determine the factor of safety for each scenario as shown in Fig. 4. Rock dowels of varying dimension and spacing are employed to improve stability where static FoS greater than 1.3 and seismic FoS greater than 1.1 was not achieved.

Table 3 Shear strength parameters of rock-rock joints

Material	Cohesion (MPa)	Friction angle (degrees)
Rock-Rock joint	0.05	35

Table 4 Estimated values of SMR at site

S.No	Face	Wall direction	SMR (Range)	Recommended support system
1	Face 1	East	35–79	Important corrective measure–occasional supports
2	Face 2	South	42–77	Systematic supports–occasional supports
3	Face 3	West	57–79	Systematic supports–occasional supports
4	Face 4	North	18–60	Re-excavation[a]–systematic supports

[a]This face needs special attention and effective support system to be provided depending upon site conditions

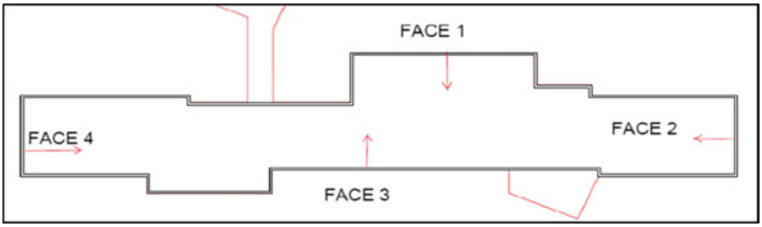

Fig. 4 Details of cut slopes at metro station

The bonded length is a function of the bar diameter, hole diameter and allowable bond stress. An allowable bond stress of 500 kPa has been selected for all anchorages based on typical conservative values presented by Duncan [3]. The unbonded length and required tension force per meter width of slope, T, has been provided where the FoS of either a sliding or rotating block does not reach 1.3. The tension force T is anticipated to be spread between several individual anchorages to distribute the load across the sliding block. For the rock mass assessment in the GSI > 40 range, the application of the secant pile (SP) loading causes local failure below the toe of the pile only, with the remainder of the slope remaining stable. A single row of rock dowels below the secant pile footing is generally adequate to prevent localized failure.

In order to analyse the stability of rock slope, possible four modes of failure were checked, i.e., Wedge failure, Planar failure, Toppling failure and Circular failure.

5 Analysis

5.1 Assumptions and Loading

As per available geotechnical investigations at the metro station location, it has been found that ground water table varies between 1.8 and 6.40 m deep from ground level. To stabilize the slope, drainage holes are recommended for the rock slope at regular intervals. Hence, zero uplift pressure has been considered on the joint plane conforming to Clause 10.2 of IS 14448 [4].

Pseudo static analysis has been undertaken to model the effects of earthquake. Seismic coefficients have been used as follows:

$A_h = 0.12$ has been used for design horizontal seismic co-efficient.

$A_v = 0.08$ has been used for design vertical seismic co-efficient.

Live loading from vehicles, e.g., cranes, at ground surface has been incorporated into the secant pile loading applied at rock head level. Details of various faces of cut slope considered for analysis are presented in Fig. 5.

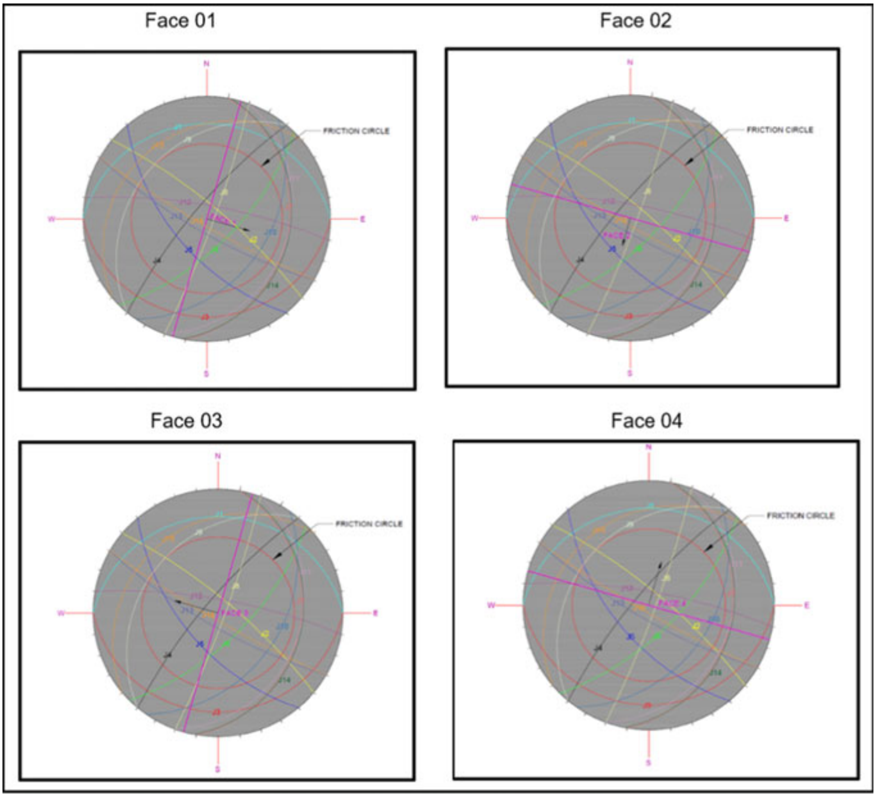

Fig. 5 Stereographic projection of joints mapped at site

5.2 *Analytical Method*

Identification of modes of failure. The stereographic projection of joints has been used for identifying the mode of failure of slope as in Fig. 6.

Wedge Failure. In order for the wedge failure to occur, three primary conditions are to be satisfied. They are as follows:

- Two planes will always intersect in a line.
- The plunge of the line of intersection must be flatter than the dip of the face and steeper than the average friction angle of the two slide planes.
- The line of intersection must dip in a direction out of the face for sliding to be feasible.

Planar Failure. A plane failure is a comparatively rare sight in rock slopes because it is only occasionally that all geometric conditions required to produce such a failure occur in an actual slope. Still, the possibility of such failure has been studied. In order for the planar failure to occur, three primary conditions are to be satisfied. They are as follows:

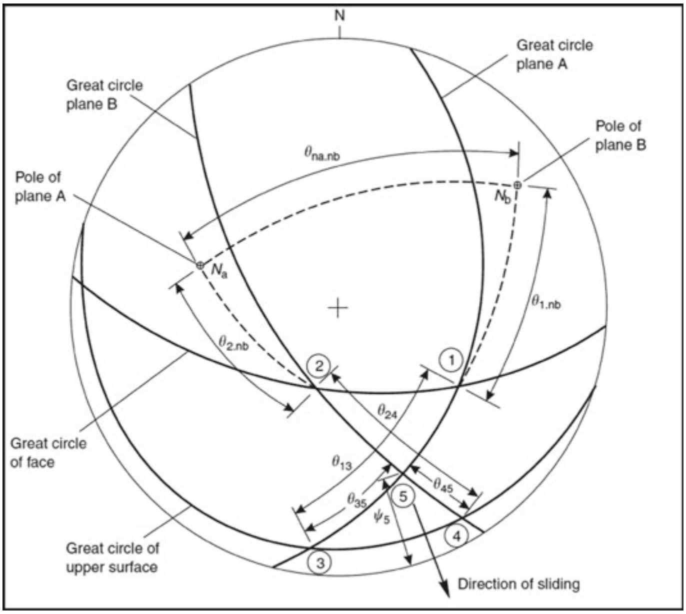

Fig. 6 Stereoplot of data required for wedge stability analysis

- Joint plane dipping out of the face.
- Dip of the joint must be less than the dip of slope face.
- The plane on which sliding occurs must strike parallel or nearly parallel (within approximately ±20) to the slope face.

*Toppling Failure***.** In order for the toppling failure to occur, two primary.

conditions are to be satisfied. They are as follows:

- Joints dipping into the face must be within about 10°.
- $(90° - \psi f) + \phi j < \psi p$, given by Goodman and Bray [5].

Where ψf is the cut slope angle, ϕj is the friction angle, and ψp is the dip of joint.

Circular Failure. In the case of a closely fractured or highly weathered rock, a strongly defined structural pattern no longer exists, and the slide surface is free to find the line of least resistance through the slope. Observations of slope failures in these materials suggest that this slide surface generally takes the form of a circle, and most stability theories are based upon this observation.

Assessment of factor of safety. Factor of safety of slopes identified in the kinematic analyses are further analysed using the formula below given in IS 14448 [4]

$$FOS = \frac{c_j A + \tan\Phi\left(\mathrm{W}\cos\Psi_p - \alpha_h \mathrm{W}\sin\Psi_p - V\sin\Psi_p - U\right)}{\mathrm{W}\sin\Psi_p + \alpha_h \mathrm{W}\cos\Psi_p + \mathrm{V}\cos\Psi_p} \tag{1}$$

FOS = Factor of safety.
c_j = Cohesion of rock-rock joint.
ψ_p = Dip of joint plane.
A = H cosec ψ_p.
Φ = Friction angle of rock-rock joint.
W = Weight of wedge = ½ Υ H^2 (cot ψ_p – cot ψ_f).
V = 0 (in the absence of tension crack).
α_h = 0.1 (for earthquake condition).
U = Uplift on joint plane = 0 (as per Clause 10.2 of IS 14448[4] for drained slope).

Hoek et al. [6], gave a formula for Factor of Safety (FoS) against wedge failure as given below:

$$\frac{3}{\Upsilon_r H}(c_A X + c_A Y) + \left(A - \frac{\Upsilon_W X}{2\Upsilon_r}\right)\tan\Phi_A + \left(B - \frac{\Upsilon_W Y}{2\Upsilon_r}\right)\tan\Phi_B \qquad (2)$$

Υ = Unit weight of rock = 26 kN/m3.
H = Cut slope Height.
$c_A = c_B$ = Cohesion of rock-rock joint = 50 kPa.
$\Phi_A = \Phi_B$ = Friction angle of joint = 35°

$$X = \frac{\sin\theta_{24}}{\sin\theta_{45}\cos\theta_{2,na}} \qquad (3)$$

$$Y = \frac{\sin\theta_{13}}{\sin\theta_{35}\cos\theta_{1,nb}} \qquad (4)$$

Υ = Unit weight of water = 9.81 kN/m^3.

Effect of water pressure has not been considered in the analysis as the drainage.
arrangement is proposed. A and B are derived from Friction only charts available in Duncan [3]. Other parameters are defined in Fig. 7.

The factor of safety against circular failure has been determined with the aid of circular failure charts, given in Duncan C [3]. These were produced by running a search routine to find the most critical combination of slide surface and tension crack for each of a wide range of slope geometries and ground water conditions. Circular chart corresponding to fully drained slope has been adopted for stability analysis.

5.3 Empirical Method

Slope Mass Rating (SMR). As per IS 13365 [1], Slope Mass Rating (SMR) can be used for preliminary assessment of the stability of rock slopes. The approach is based on modification of RMR system using adjustment factors related to discontinuity

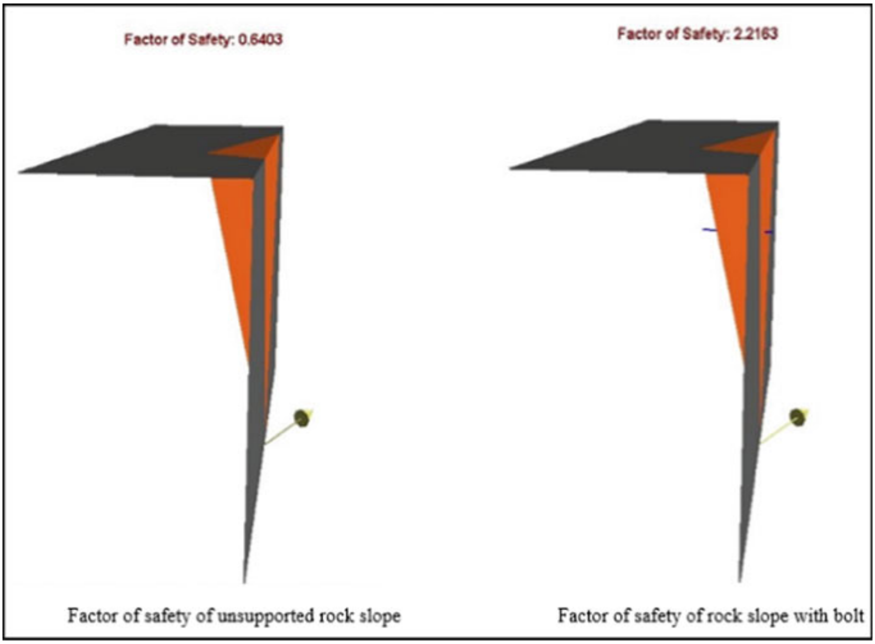

Fig. 7 Typical results of SWedge analysis of metro station excavation

orientation with reference to slope as well as failure mode and slope excavation methods.

$$\text{Slope mass rating (SMR)} = \text{RMR}_{\text{basic}} + (\text{FI} \times \text{F2} \times \text{F3}) + \text{F4} \quad (5)$$

The values of F1, F2, F3, and F4 have been taken from IS 13365 [1]. The adjustment rating for joints and excavation in rock slopes depends on the following factors:

F1: Factor which is dependent on parallelism between the slope and the discontinuity.

F2: Factor which is dependent on the dip of discontinuity.

F3: Factor which is dependent on the relationship of dip of discontinuity and inclination of slope.

F4: Factor which depends on whether the slope under investigation is a natural one or excavated by pre-splitting, smooth blasting, mechanical excavation or poor blasting (Table 4).

Northern side has been found as critical and the support system shall be finalized only after a proper assessment of joint conditions by site geologist.

5.4 *Numerical Analysis*

Analyses to assess the overall stability of the slopes have been carried out using softwares—SWedge, RocPlane, RocTopple and RS^2.

Wedge failure analysis. SWedge is a software developed by Rocscience Inc. This is based on the method developed by Goodman and Shi [7]. Factor of safety of the identified wedges has been evaluated using SWedge software. Analysis has been considered by incorporating both static and dynamic conditions for both supported and unsupported wedges. Typical results of SWedge are presented in Fig. 8.

Planar failure analysis. Identified cases of Planar failure have been further analysed using RocPlane software of Rocscience Inc. Factor of safety has been determined for all planar failures likely to be formed in the slope. Analysis has been performed for both static and dynamic loading conditions considering the reinforcement as required. Typical results of planar failure are presented in Fig. 9.

Toppling failure analysis. RocTopple software of Rocscience Inc. has been used to determine the factor of safety of identified toppling blocks. This software is based on the limit equilibrium analysis. Typical result of toppling failure analysis is presented in Fig. 9.

Circular failure analysis. Detailed circular analysis has been carried out by RS^2 software [8] of Rocscience Inc. for various combinations of rock slope height and overburden possible at the site. Critical Strength Reduction Factor (SRF), equivalent to factor of safety has been evaluated for all possible scenarios. Factor of safety has been determined in both static and dynamic loading conditions to ensure the safety of the rock slope in adverse conditions. Typical result of failure analysis is presented in Fig. 10.

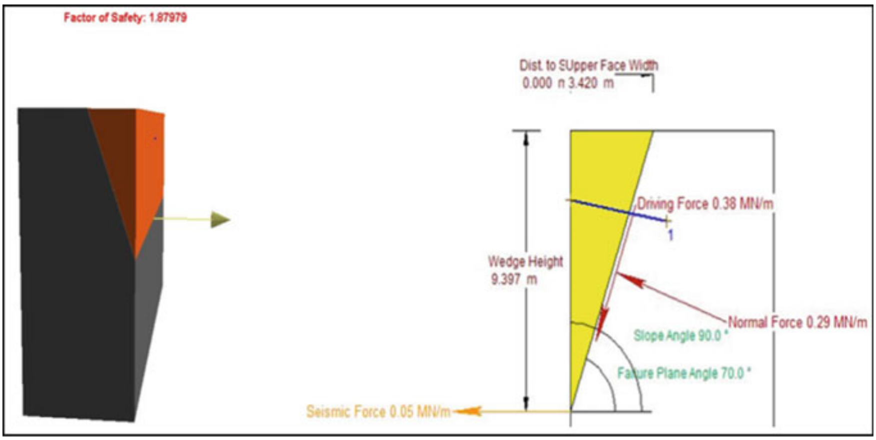

Fig. 8 Typical results of RocPlane failure analysis of metro station excavation

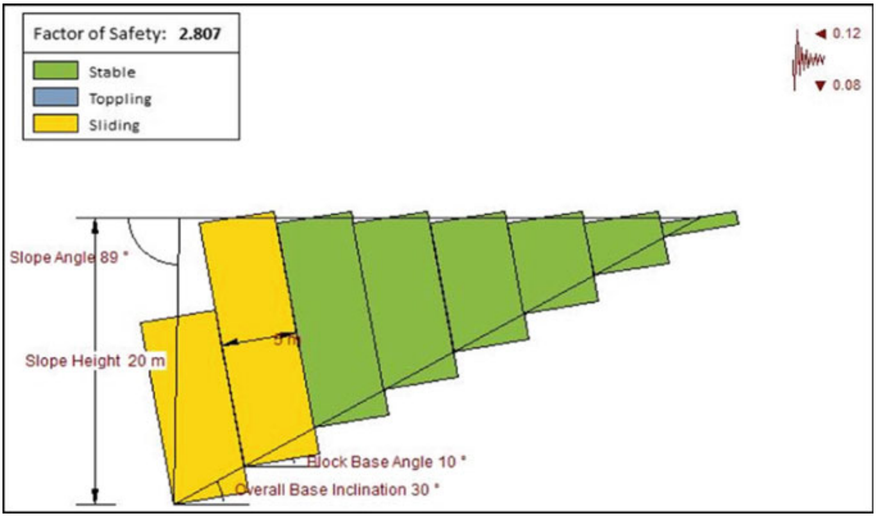

Fig. 9 Typical result of RocTopple failure analysis of metro station excavation

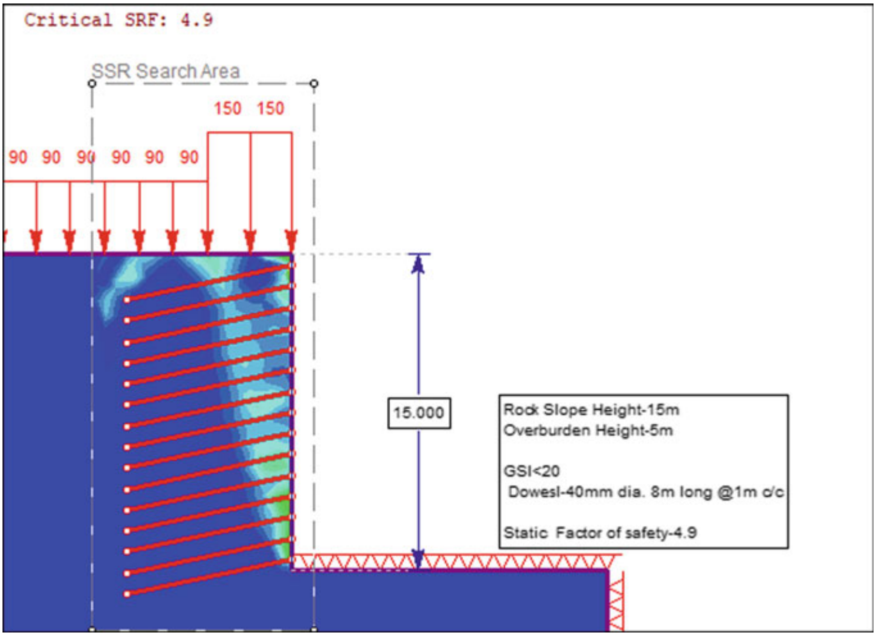

Fig. 10 Typical result of circular failure analysis of metro station excavation

Hoek Brown failure criterion has been used to represent the strength of the rock mass. Analyses have been carried out for three rock mass classes represented by GSI ranges: GSI > 40, GSI 20 to 40 and GSI < 20 and for all possible ranges of overburden against height of rock slope to determine the factor of safety for each case. Overburden thickness above rock head is applied as a surcharge at rock head level.

From the analysis, it has been estimated that spot bolting length of 4 m is required for slope stability. The length and spacing of rock bolts required for systematic bolt has been determined from the analysis for various cases of rock mass quality and depths of overburden and rock mass.

6 Instrumentation and Monitoring

Instrumentation and Monitoring forms an integral part of design. This has been recommended during the excavation of station to verify/alert the designer/contractor about the recorded values. In this regard, optical targets were installed on the rock slope to measure the 3D deformations of soil and rock mass within the excavation. Load cells are also installed at some dowels to understand whether the actual forces are same as anticipated forces or not.

7 Comparative Analysis

Methods for rock slope stability can be categorized into three: (1) Analytical methods by limit equilibrium analysis, (2) Empirical methods and (3) Numerical methods. In limit equilibrium analysis, factor of safety of the slope is calculated with unique procedures for wedge, plane, circular and toppling failures. Slope Mass Rating (SMR) is an empirical method developed to use in slopes as a sequel of Bieniawski's Rock Mas Rating (RMR) system by Romana [9]. Guidelines were proposed based on SMR for application of remedial measures.

Numerical analysis by RS^2 of Rocscience Inc. has been used to estimate a critical strength reduction factor which is equivalent to factor of safety of the slope. The basic concept is to reduce the strength parameters by a certain factor and to compute finite element stress analysis. The process is repeated for different values of Strength Reduction Factor (SRF) until the model becomes unstable, which corresponds to the critical Strength Reduction Factor (critical SRF) of the slope.

All methods have their own advantages and disadvantages. Empirical method by means of SMR can be used to predict the support system from rock mass classification and geological characterization of rock-rock joints. This method helps us to predict the probable type of failure and to alert the designer and site engineers about the critical face of excavation. Analytical methods by limit equilibrium analysis do not help us to predict the mode of failure. But this method helps to determine the stability of slope by calculating the factor of safety. Unlike other methods, numerical analysis considers more geotechnical parameters and results of the analysis are to be interpreted by keeping in mind degree of uncertainty involved in estimation of parameters. Reliability of results of numerical analysis is dependent upon the quality of input parameters and the assumptions involved.

8 Conclusions

Rock cut slope of the metro station has been analysed for wedge failure, planar failure, toppling failure and circular failure, and appropriate support arrangement is recommended. Analysis performed leads to the conclusion that the planned cut slope will be safe with the recommended support system varying from spot bolting arrangement to systematic bolting arrangement depending upon the site conditions encountered.

From the analysis, it has been estimated that spot bolting length of 4 m is required for slope stability. The length and spacing of rock bolts required for systematic bolt has been determined from the analysis for various cases of rock mass quality and depths of overburden and rock mass. Since the rock slope is vertical, shotcrete of 100–150 mm thickness with wire mesh is recommended to be applied for safety during excavation and to prevent erosion and weathering of rock-rock joints. 75 mm dia. 3 m long drainage holes have been recommended to release the water pressure from the rock slope.

During excavation, no slope more than 2.5 m depth was not kept unsupported. The extent of blasting matched with the rock condition so as not to over fracture the rock mass. Wire mesh was always fully covered with shotcrete to prevent corrosion by wetting and drying cycles. Whenever the joint conditions assumed for the analysis did not match with design assumptions, slope support system was revised to suit to the site conditions encountered.

Rock slope stability involves estimation of parameters of rock mass and rock-rock joint. The parameters shall be applied with adequate engineering judgement and after elaborate discussion among the stakeholders. The design approach must be followed in such a way that all the available methods shall be exploited to the core. Scenarios corresponding to each and every probability shall be forecasted. The parameters are to be assigned by considering the safety of the metro station. Risks involved in the omission and admission of all the scenarios shall be identified and mitigation system shall be derived accordingly.

References

1. IS 13365-Part 3: Quantitative Classification System of Rock Mass—Guidelines: Determination of Slope Mass Rating. Bureau of Indian Standards, New Delhi (1997)
2. Hoek, E.: Practical Rock Engineering (2006)
3. Wyllie, D.C., Mah, W.: Rock Slope Engineering, 4th edn. CRC Press (2004)
4. IS 14448: Code of Practice for Reinforcement of Rock Slopes with Plane Wedge Failure. Bureau of Indian Standards, New Delhi (1997)
5. Goodman, R.E., Bray, J.W.: Toppling of rock slopes. In: Proceedings of the Specialty Conference Rock Engineering for Foundations and Slopes, pp. 201–234. American Society of Civil Engineers, Boulder, CO. (1976)
6. Hoek, E., Bray, J., Boyd, J.: The stability of a rock slope containing a wedge resting on two intersecting discontinuities. Quart. J. Engng Geol. **6**(1), 22–35 (1973)
7. Goodman, R., Shi, G.: Block Theory and its Application to Rock Engineering. (1985)
8. RS^2 v9.0. Elasto-plastic Finite Element stress analysis program for underground or surface excavations in rock or soil. (2016)
9. Romana, M.: New adjustment ratings for application of Bieniawski classification to slopes. In: Proceedings of the International Symposium on the Role of Rock Mechanics in Excavations for Mining and Civil Works. International Society of Rock Mechanics, pp. 49–53, Zacatecas. (1985)
10. IS: 1893-Part 1: Criteria for earthquake resistance design of structures. Bureau of Indian Standards, New Delhi (2002)
11. RocData Version 5.007, Rocscience Inc