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Assessing rock mass properties for a hydropower projectusing deformability tests

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Abstract

Rocks encountered at hydro power project sites in the Himalayas are generally jointed, anisotropic and show scatter in their engineering response. While the intact rock strength can be obtained easily by testing specimens till failure in laboratory, assessment of the ultimate strength of rock mass by stressing it up to failure is extremely difficult and not feasible. Alternative approaches are therefore preferred to assess the rock mass strength. This article advocates that rock mass strength can be obtained from the deformability of the mass which can be obtained by stressing the rockmass to a very low level of stress. This hypothesis is based on outcome of large number of UCS tests performed on jointed block mass specimens.

The hypothesis is utilised to assess the rock mass properties for a hydro power project located in higher Himalayas. Comprehensive laboratory tests were performed to get the intact rock properties and their probability distributions. Several cyclic plate load tests were performed at the site to get the elastic modulus of the rock mass. These modulus values were then utilised to arrive at a set of UCS values of the rock mass. A non-linear strength criterion was then used to arrive at strength values under confinement. These results were used to simulate instabilities observed in a cut rock slope at the dam site.

It is concluded that deformability of rock mass, which is relatively easy to obtain in the field, along with laboratory test results for intact rock, can be used successfully to arrive at strength of rock mass.

Keywords

Intact rock, Rock mass, Strength, Deformability, Non-linear strength criterion.

1 Background

The Nikachhu Hydroelectric Project located inTrongsa, Bhutan was commissioned in 2024 with an installed capacity of 118 MW. The rocks encountered at the dam site are heavily jointed mica schist intermittently intruded with quartz and garnets.The strength of such a rock mass is not "measured" through direct tests, rather "derived" indirectly. Preliminary estimates of the strength are made through classification techniques (Q, RMR and GSI). However, it was envisaged to improve the estimates by incorporating results from Cyclic Plate Load Tests (CPLTs)for higher reliability. The CPLT does not stress the rock mass to its ultimate failure stress, however, the output the test in the form of deformation modulus, implicitly represents in-situ conditions.It is established in literature (Singh & Rao, 2005)that strength and modulus of rock mass are closely related with those of the parent intact rock. This correlation can be used to arrive at rock mass strength by inputting the field rock mass modulus, and laboratory UCS and modulus of intact rock.

The present study uses an integrated approach for assessing the rock mass strength by incorporating field modulus from CPLTsand UCS test results on intact rock specimens in the laboratory. The main objective has been to derive equivalent material shear strength parameters that better reflect the in-situ rock mass conditions. Initially the hypothesis is discussed and then it is used in assessing the equivalent material strength for a rock mass at a dam site.

2 The Hypothesis

2.1 UCS of Rock Mass

Singh and Rao (2005) suggested an empirical approach to estimate the UCS of jointed rock mass based on the outcome of large number of UCS tests performed on jointedrock mass specimens (Singh, 1997). The test specimens (Singh, 1997)had varying joint configurations exhibiting different levels of interlocking conditions thereby resulting in various failure modes encountered in nature. Typical failure modes observed in the study are shown inFig. 2. The tangent (elastic) modulus and ultimate strength values obtained from the tests were plotted on (Deere & Miller, 1966)classification chart (Fig. 1). It was observed that the jointed rock locations follow a straightlinestarting from point 'I' representing intact rock position on the chart. This empirical line indicates a strong correlation between the modulus and strength values of intact and rock mass. The strength of the rock mass can be obtained as follows:

$$\frac{\sigma_{cj}}{\sigma_{ci}} = \left[\frac{E_j}{E_i}\right]^{0.63} \tag{1}$$

Where $\sigma_{ci}$and σ_{cj}= UCS of intact rock and rock mass; E_i&E_j= Elastic (tangent) modulus of intact rock and rockmass respectively.

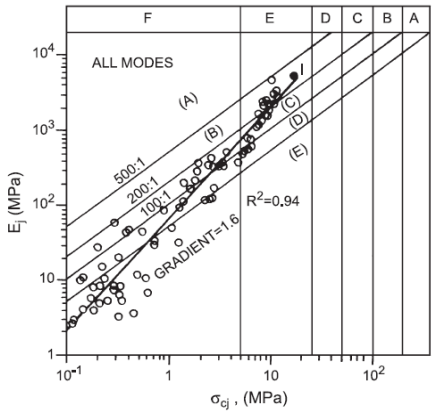


Fig. 1 Strength and tangent modulus of rock mass specimens plotted on DeereMiller classification chart (Singh, 1997)

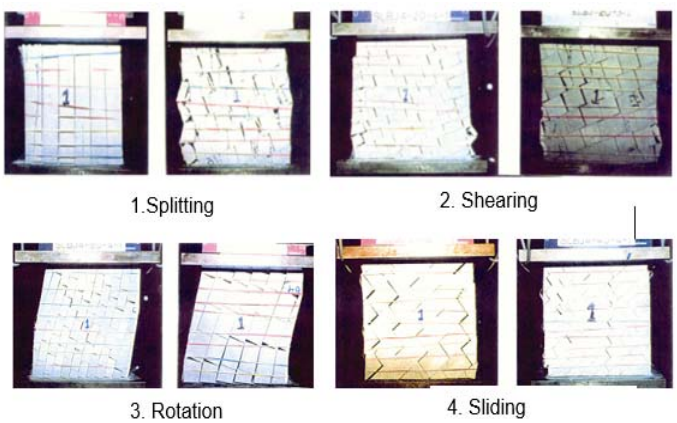


Fig. 2 Rock mass specimens showing various failure modes under uniaxial compression (Singh, 1997)

The Eq. (1) is used to determine the rock mass strength, σ_{cj} for given σ_{ci}, $E_i$and $E_j$values. The present study concerns slope instability of a rock slope at Nikachhu Hydroelectric dam site. The rock mass modulus in the field, $E_j$was obtained by conducting CPLTs. The CPLTs were performed at the base of a drift excavated at the level of the toe of slope. The rockmass under the test plate represents semi-infinite constrained rock mass with restriction on dilation during shear. On the contrary, the rock mass specimens tested by Singh (1997) (Fig. 2)were unconstrained on sides; and the loading platen covered the entire plan area of the rock mass. Consequently, the rock mass was unconstrained and free to dilate. The joint orientations play significant role in case of unconstrained, free to dilate rock mass. To apply the CPLT results to a rock slope problem, a correction needs to be applied to account for the joint orientation. In present study, the correction is applied to the computed rock mass strength by multiplying it with Joint Inclination Parameter 'n'(Ramamurthy & Arora, 1994; Singh et al., 2002) in Eq. (2). The joint inclination parameter accounts for the weakness brought to a jointed rock due to inclination of the joints. The inclination parameter 'n' is presented in Table 1. The corrected UCS of the rock mass is obtained as given below.

$$\sigma_{cj(corrected)} = n \times \sigma_{cj(computed)} \tag{2}$$

Where n = Joint inclination parameter

Table 1. Joint Inclination Parameter (Ramamurthy &Arora, 1994; Singh et al., 2002).

Joint dip	0	10	20	30	40	50	60	70	80	90
n	1.000	0.814	0.634	0.465	0.306	0.071	0.046	0.105	0.460	0.810

2.2 Triaxial Strength of the Rock Mass

Jointed rock masses exhibit large non-linearity in their triaxial strength behaviour. In present study, a non-near failure criterion, Modified Mohr Coulomb Criterion (MMC)(Singh & Singh, 2012)was used to model the non-linear strength response of the rock mass. As per this criterion, the non-linear triaxial strength of a jointedmass is defined as:

$$(\sigma_{1j} - \sigma_{3j}) = \sigma_{cj} + \frac{2sin\phi_{j0}}{1 - sin\phi_{j0}}\sigma_{3j} - \frac{1}{\sigma_{ci}}\frac{2sin\phi_{j0}}{1 - sin\phi_{j0}}{\sigma_{3j}}^2 for\ 0 \leq \sigma_{3j} \leq \sigma_{ci} \tag{3}$$

Where: σ_{cj} = UCS of the rock mass; σ_{3j}&σ_{1j}= minor and major principal stresses at failure for the rock mass; ϕ_{j0} =MMC parameter which represents internal friction angle of the mass at very low confining pressure ($\sigma_3 \rightarrow 0$),obtained as:

$$sin\phi_{j0} = \left\{\left(1 - \frac{\sigma_{cj}}{\sigma_{ci}}\right) + \frac{sin\phi_{i0}}{1 - sin\phi_{i0}}\right\} \Big/ \left\{\left(2 - \frac{\sigma_{cj}}{\sigma_{ci}}\right) + \frac{sin\phi_{i0}}{1 - sin\phi_{i0}}\right\} \tag{4}$$

ϕ_{i0}=MMC parameter for intact rock i.e. friction angle of the intact rock at very low confining pressure ($\sigma_3 \rightarrow 0$) (Singh et al., 2011). Triaxial test data on intact rock is required to computeϕ_{i0}. The following steps are used to compute ϕ_{i0}.

$$A_i = \frac{\sum(\sigma_{1i} - \sigma_{3i} - \sigma_{ci})}{\sum({\sigma_{3i}}^2 - 2\sigma_{ci}\sigma_3)}, \qquad 0 < \sigma_{3i} \leq \sigma_{ci} \tag{5}$$

$$B_i = -2A_i\sigma_{ci} \tag{6}$$

$$sin\phi_{i0} = B_i/\{2 + B_i\} \tag{7}$$

Where: σ_{ci} = UCS of the intact rock; σ_{3i}&σ_{1i}= minor and major principal stresses at failure for the intact rock.

3 Field and Laboratory Investigations

Detailed investigations were conducted in the field and laboratory to characterise the geological materials encountered at the site. A total of seven drill-holes were advanced in the dam area and intact rock cores were tested in laboratory to determine their physical and mechanical properties. The laboratory tests included UCS (20 numbers), triaxialstrength (11 sets) and Brazilian tests (18 tests). In addition, fourteen sets of direct shear testsalong the joint surfaces. In field, six number of cyclic plate load tests (CPLT) were performed in a drift along the dam axis. The intact rock properties were analysed statistically and the cumulative distribution functions of the UCS and the tangent moduli (E_{t50}) of the intact rock were determined. The summary of the results from geotechnical investigations is presented in Table 2.

Table 2 Properties reported for Nikachhu Project(GBR, 2017)

Property	Mean value	Standard deviation	Weibull shape parameter(α)	Weibull scale Parameter (β)
Unit weight, (kN/m^3)	27.00	0.75		
UCS, (MPa) (Saturated)	45	22.01	1.744	52.288
E_{t50}, (GPa) (Saturated)	34.19	13.91	1.763	41.089
P Wave velocity (Km/sec)	3.854	0.941		
Brazilian Tensile Strength (MPa)	7.31	3.20	1.939	8.502
Mohr-Coulomb, c_i (MPa)	2.53	1.14		
Mohr Coulomb ϕ_i(degrees)	42.27	7.79		

4 Rock Mass Properties

The following steps were used to arrive at the rock mass strength:(Fig. 3)

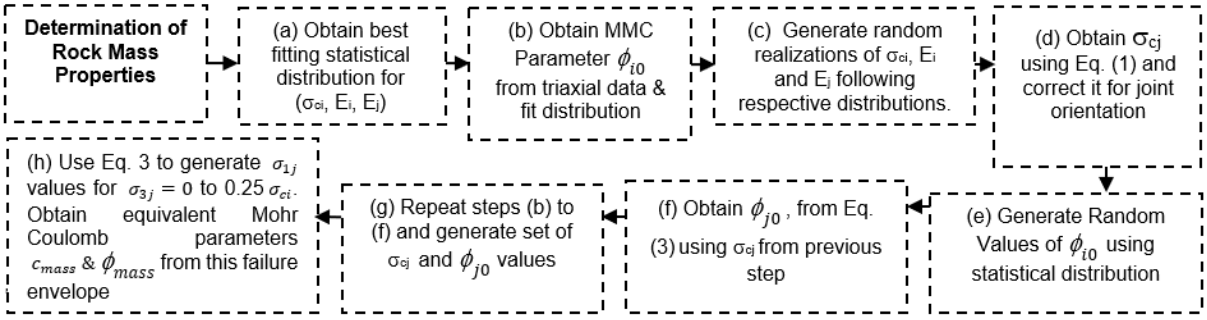


Fig. 3Steps involved in derivation of rock mass parameters

The distributions for equivalent material Mohr-Coulomb shear strength parameters are shown in Fig. 4.

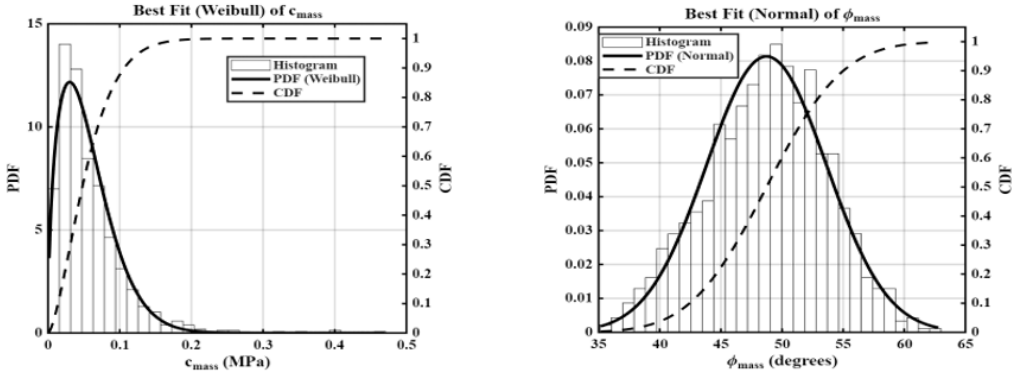


Fig. 4Consolidated distribution plots for Cohesion (MPa) and Friction angle of jointed rock mass

The slope mass at the site is intersected with several joints. The joint planes had low persistence, were not well developed and did not have distinct surfaces. The joint planes having dip direction within

±20° of slope dip direction had an average of dip of55.5°. The influence of these weaker planes wasincorporated through joint inclination parameter ‘n’.

4.1 Slope Stability Analysis

The equivalent material Mohr-Coulomb parameters obtained above are employed to simulate the instabilities observed at the rock slope on left abutment of the dam site.



Fig. 5Slope excavation section and location of instability

The rock slope was to be excavated for constructing the dam as shown in Fig. 5. The excavation was done by making benches of 3 m width 2.5m height respectively. The monsoon had just begun in the month of May 2018. When the excavation reached at El 2300m, tension cracks appeared at El 2309 m andthey propagated further with time. The work was put on hold to arrest the propagation of the tension cracks and infiltration of rainwater by sealing of crack openings with cement grout and installation of additional anchors. However, the slope couldn’t be stabilised, and it failed (Fig. 5). Later, the slope was redesigned, and re-excavation was carried out by shifting it backward. Mitigation measures were applied along with modified support systems.

The stability analysis of the section at the time of instability was done using SLIDE2 (Rocscience 2022) (Fig. 6). It is seen That the FOS is close to one which indicates instability. The mitigations measures were designed and the slope was stabilised.

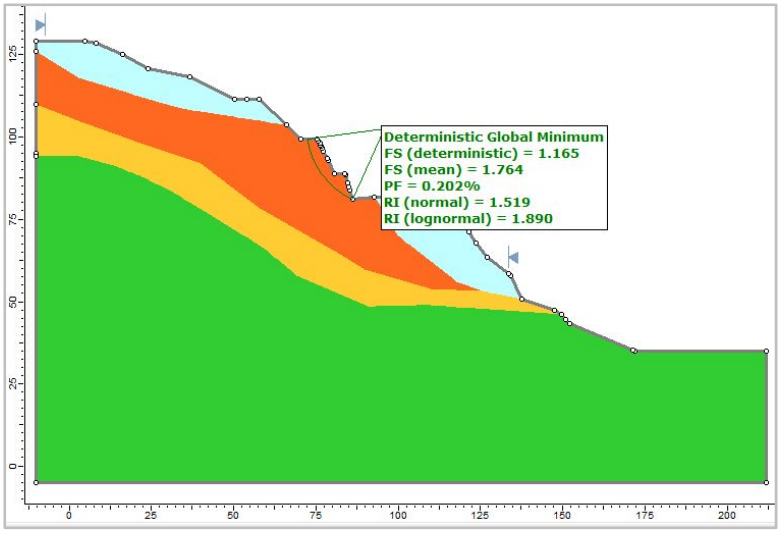


Fig. 6Slope stability analysis using SLIDE 2 (Rocscience, 2022)

5 Concluding Remarks

Shear strength parameters of rock mass are required for analysis and design of structures. The present study has focused on obtaining shear strength parameters for rock slope stability analysis. Orientation of weakness planes have great influence on stability of slopes. For situations where joints are not well developed and have low persistence, an equivalent material may be assumed, and analysis can be done by using equivalent shear strength parameters.

The present study advocates use of field modulus to assess the equivalent shear strength parameters of a rock mass in the field. The approach requires laboratory UCS (σ_{ci}), modulus (E_i) values of intact rock, and rock mass modulus (E_j) from the field which can be obtained through Cyclic Plate load tests. The UCS of the rock mass (σ_{cj}) may be obtained through correlation suggested in the study.

It is envisaged that rock mass modulus from the field will reflect influence of in-situ rock conditions. However, the influence of the joint orientation will not reflect fully as the CPLT is conducted on semi-infinite constrained mass contrary to the unconstrained rock mass in slope which has higher degree of freedom for slip, rotation of blocks and dilation of mass. A correction for joint orientation should be applied to the computed σ_{cj} before using it for slope stability analysis.

The applicability of the suggested approach has been demonstrated by applying it to the left abutment slope of a hydro power project. It is concluded that addition of input from field deformability may be used to enhance the confidence in the shear strength parameters.

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