Deformability Characteristics of Garnetiferous Quartzo-Feldspathic Gneiss Rock Mass – A Case Study

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Abstract

Unlike soil, rock mass is a heterogeneous material containing discontinuities in the form of joints, faults, rock fragments and various degrees of weathering. Any structure involving rock requires the knowledge of deformability characteristics of intact rock and the rock mass containing discontinuities. There are many methods available for the determination of modulus of rock mass in the field. However, these methods have many limitations. The present paper is based on the deformability characteristics of garnetiferous quartzo-feldspathic gneiss rock mass in southern part of India.
Keywords: Modulus of Deformation, Modulus of Elasticity, Intact Rock, Rock Discontinuities, Weathering

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I. INTRODUCTION

A rock mass is heterogeneous and quite often discontinuous. Often, a rock mass is composed of a system of rock blocks and fragments separated by discontinuities forming a material in which all elements behave in mutual dependence as a unit (Matula and Holzer, 1978). Strength and deformability characteristics of rock mass are the important parameters required for the design of structures on or in the rock mass. Strength of rock mass depends on the strength of intact rock and discontinuities. Laboratory tests on rock cores give strength of intact core whereas field tests takes into account the combined effect of discontinuities present in the geological setup which signifies the need for evaluation of in-situ parameters. Several tests/instruments exist for the evaluation of modulus of deformation of rock mass such as pressure meter, dilatometer, plate jacking, plate loading, radial jacking, flat jack, cable jack and Goodman jack. Indirect methods can also be used to assess the modulus of deformation of rock mass but these methods have many limitations. Empirical correlations to estimate the strength and modulus of rock mass are mostly derived from different rock mass classification systems viz; Rock Quality Designation (RQD) (Deere, 1964), Rock Mass Rating (RMR) system (Bieniawski, 1973, 1976, 1989); the Q-system (Barton et al., 1974); and Geological Strength Index (GSI) system (Hoek and Brown, 1997). Many other correlations have also been suggested (Gardner, 1987, Serafim and Pereira, 1983, Johnson et al, 1980, Nicholson and Bieniawski, 1990). Palmstrom and Singh (2001) concluded that RMR and RMi systems gives better estimation of deformation modulus for jointed rocks. The authors further concluded that modulus values obtained by plate loading test (PLT) and Goodman Jack test are generally lower compared with those determined by plate jacking method (PJT). Modulus values by plate loading and Goodman jack tests are generally in agreement for results from manually excavated drifts and fresh drillholes (Hari Dev and Rajbal Singh, 2015). Present case study is based on the plate load tests conducted in quartzo-feldspathic gneiss rock at foundation level of spillway for an irrigation project in Southern India.

II. GEOLOGICAL SETUP

The rocks exposed in the area belong to Khondalite suite, trending in ENE – WSW direction of Eastern Ghat Mobile belt of Archean age. The prominent rock types in the area are garnetiferous quartzo-feldspathic gneiss, garnet biotite gneiss, charnockite and migmatite gneiss. The rocks are weathered to fresh, hard and competent in nature. The general foliation directions are south eastern with steep dips. Variation in foliation is also observed at places which are due to folding. The rock mass is intersected by two prominent joint sets and random joints. The prominent joints are: 310 to 320 -130 to 140(strike)/90^0, 070-250(strike)/90^0 and 090-270(strike)/90^0. The joints, in general, are tight to slightly open, moderately to widely spaced, rough, planar, irregular, continuous to discontinuous and straight to curvilinear in nature. UCS of the fresh rock varies from 80 to 100 MPa in dry and saturated conditions. Weathering is commonly observed in the area and varies from highly weathered to slightly weathered. However at places weathering is so intense that the rock is converted into reddish brown silty clay soil. The rock in the area also consists of pegmatite bands/veins of varying thickness ranging from few cms to a meter. It is also noticed that wherever these pegmatite bands/veins exist, generally they are sheared and crushed, affecting the adjoining rock mass. The spillway axis showing the test locations is shown in Figs. 1.

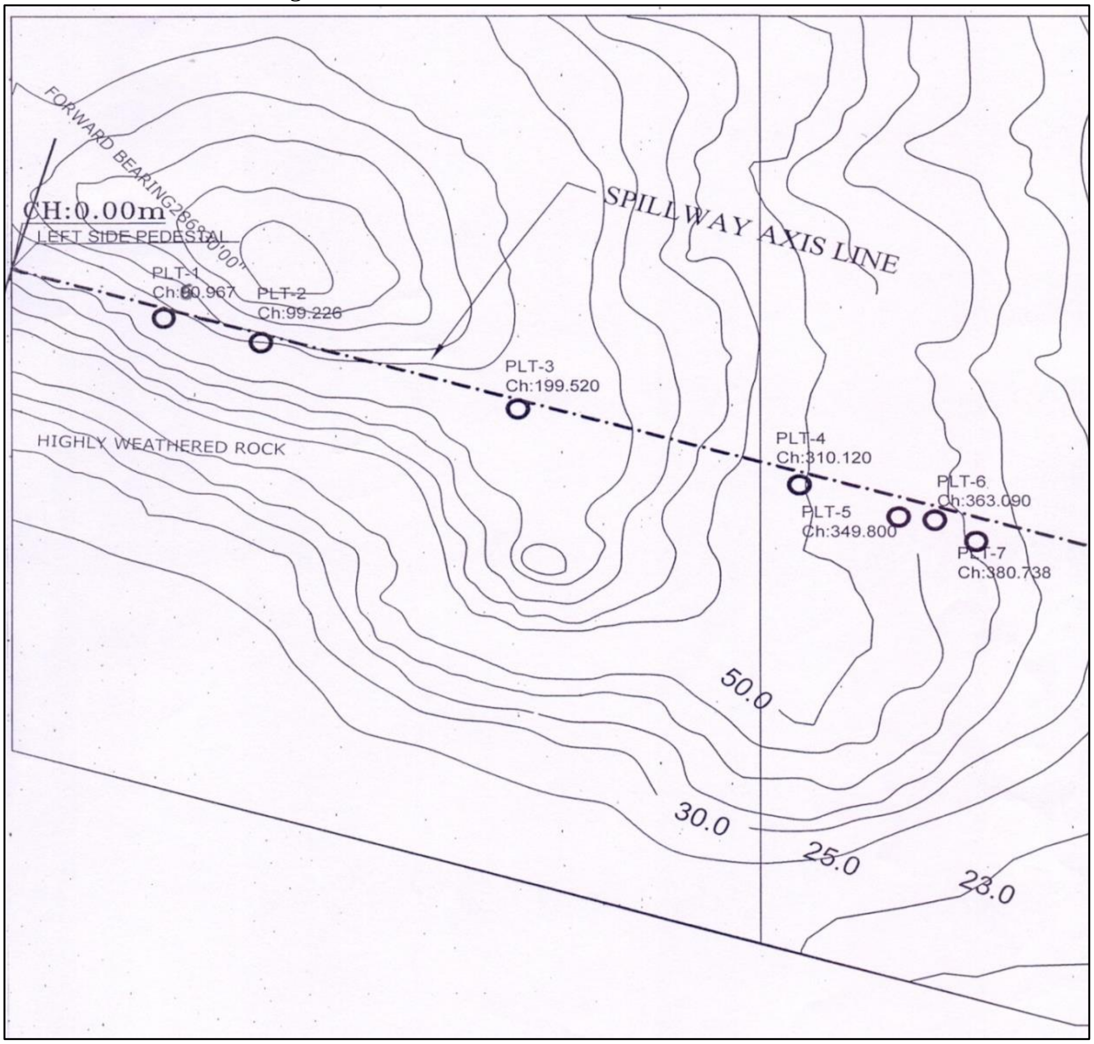

Fig. 1: Layout plan showing the plate load test locations

III. Plate Load Test

Seven locations were selected for carrying out plate load tests along the spillway axis from chainage 60 m to 380 m (RL 9.567 m to 10.345 m) along the spillway axis.

Surface preparation for the plate load tests was done carefully using electrically operated small cutting machines, grinders, chisels and hammers. The surface was made smooth with undulations less than 5 mm. Rock powder from grinding was used smoothening the minor undulations.

The maximum load was decided on the basis of stresses expected on the foundation and the provisions of IS 7317:1993. Excavator weighing 120 tonnes was used for providing the desired loading. For this, a beam consisting of ISMB 300 sections, stiffeners and flange plates was designed and fabricated in the project workshop and the same was welded on the bottom plate of the excavator with its centre matching with the center of gravity of the excavator. Hence, this mobile excavator of 120 tonne weight was considered adequate for providing the ultimate load of 72 tonnes during testing. Ramps were provided on both sides of the test location for making head room. This mobile loading arrangement was helpful in conducting as many as 7 tests in shortest possible time of 5 working days.

Step wise test procedure adopted while conducting plate load test is as following:

1) At the selected test site, the rock surface was prepared using electrically operated small cutting machines, grinders, chisels and hammers. The surface was made smooth with undulations less than 5 mm. These minor undulations were smoothened using the rock powder from grinding.
2) For Kentledge arrangements, a mobile excavator weighing 120 tonnes was selected. A beam comprising of ISMB 300 sections, stiffeners, flange plates on top and bottom was designed and fabricated and the same was welded to the bottom plate of the excavator with its center point at the center of gravity of the excavator.

3) The test was performed using 60 cm diameter plate. The load was applied by means of jack and pump and the test was completed in five loading and unloading cycles of 0.5, 1.0, 1.5, 2.0 and 2.5 MPa stress levels. The deformations were recorded using four dial gauges with an accuracy of 0.01 mm installed diagonally on the bottom plate.

Set up of in-situ plate load test assembly and deformation measuring arrangement is shown in Fig. 2.

The modulus of deformation for the loading cycle has been calculated by considering total deformation during a particular cycle, whereas, modulus of elasticity has been calculated by considering elastic deformation for the same cycle using the Eqn. (1).

$$E = \frac{Pm\,(1-\nu^{2})}{\delta\sqrt{A}} \qquad (1)$$

Where,
E = Modulus of deformation/elasticity
P = Applied load
ν = Poisson's Ratio of rock mass (ν = 0.20 for the Granite rock mass)
m = Constant depending upon the shape of plate
(m = 0.96 for circular plate)
δ = Deformation corresponding to load
A = Area of plate

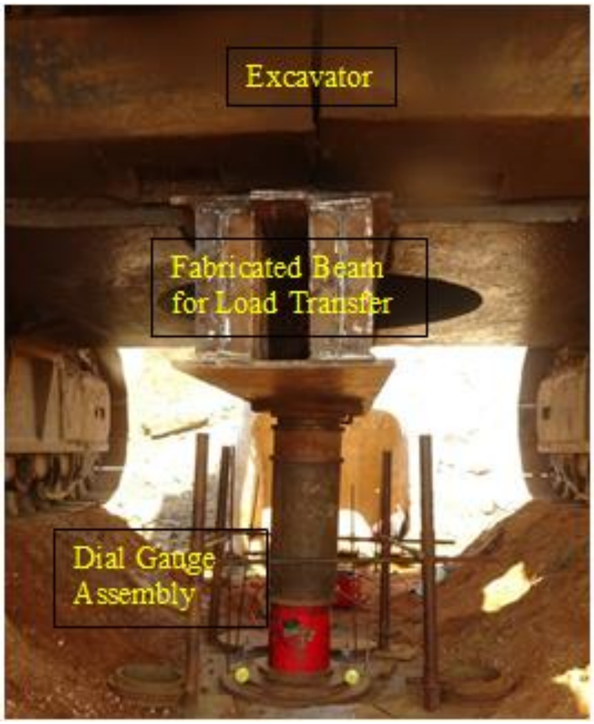

Fig. 2: Setup of plate load test assembly

The Eqn. 1 was used to calculate the values of moduli of deformation and elasticity using the value of total deformation of the loading cycle and elastic deformation of unloading cycles, respectively.

IV. RESULTS AND DISCUSSIONS

Seven plate load tests (PLT) were carried out at foundation level along the spillway axis at around RL 10 m in garnetiferous quartzo-feldspathic gneiss at 60.967 m, 99.226 m and 199.520 m, 310.120 m, 349.800 m, 363.090 m and 380.738 m. Though basic rock remained the same at all the locations, however the variation in weathering, jointing, infilling, seepage was observed from left to right flank. Figures 3, 4 and 5 show the variable geological conditions at PLT-1, PLT-5 and PLT-7, respectively whereas pressure versus deformation plots is shown in Figs. 6, 7 and 8 for corresponding tests.

Fig. 3: Exposed rock at chainage 60m (PLT-1)

Fig. 4: Exposed rock at chainage 200m (PLT-5)

Fig. 5: Exposed rock at chainage 350m (PLT-7)

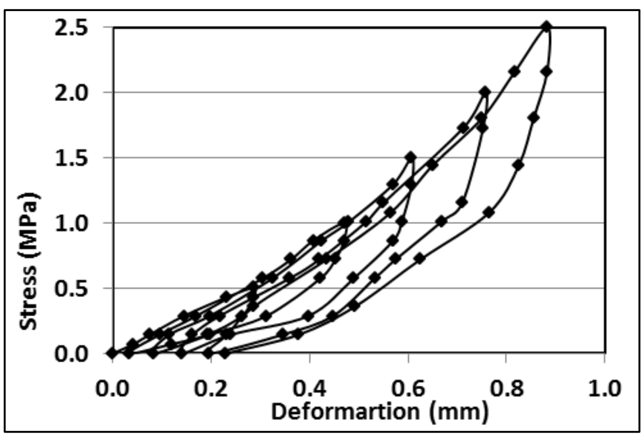

Fig. 6: Stress-deformation plot for PLT-1

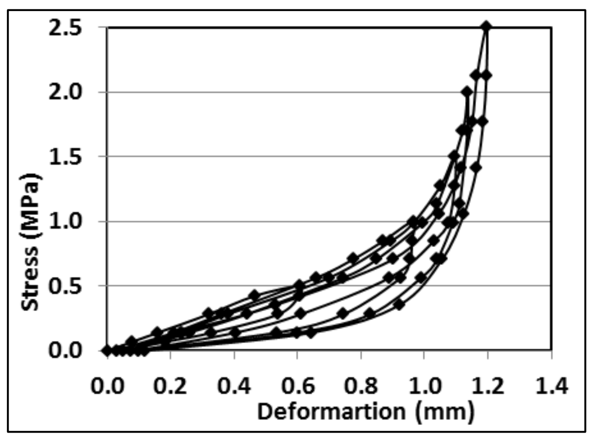

Fig. 7: Stress-deformation plot for PLT-5

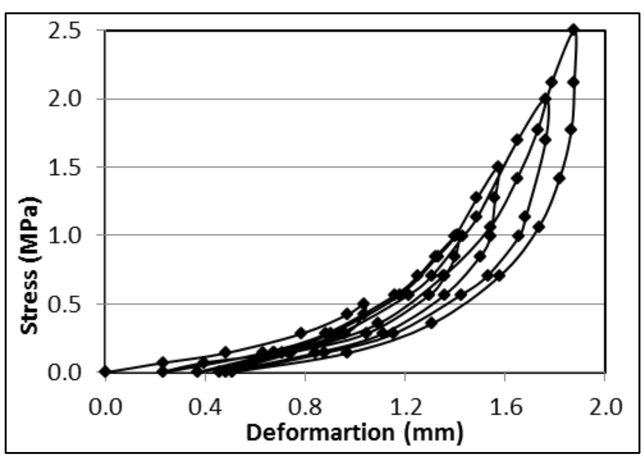

Fig. 8: Stress-deformation plot for PLT-7

The modulus of deformation values were determined for stress levels of 1.0, 1.5, 2.0 and 2.5 MPa. First cycle of 0.5 MPa stress level was intensionally omitted as the deformations in this cycle may not be true representative of actual rock mass. Table 1 presents the variation in modulus of deformation at various stress levels.

Table – 1
Variation in modulus of deformation with applied stress

Applied Stress, MPa	*Modulus of Deformation, GPa*						
	PLT-1	*PLT-2*	*PLT-3*	*PLT-4*	*PLT-5*	*PLT-6*	*PLT-7*
1.0	*1.142*	*0.929*	*0.753*	*0.724*	*0.534*	*0.451*	*0.423*
1.5	*1.427*	*1.223*	*0.973*	*0.861*	*0.717*	*0.580*	*0.622*
2.0	*1.618*	*1.586*	*1.223*	*1.019*	*0.943*	*0.634*	*0.764*
2.5	*1.817*	*1.951*	*1.469*	*1.178*	*1.141*	*0.759*	*0.894*

Rock mass at PLT-1 and PLT-2 was found to be massive (Fig. 3) whereas fractured rock mass conditions were observed in PLT-6 and PLT-7 (Fig. 4). Moderately jointed rock mass was encountered in between i.e. at PLT-3 to PLT-5 locations (Fig. 5). Total deformation in PLT-1, PLT-5 and PLT-7 were of the order of 0.88 mm, 1.20 mm and 1.88 mm, respectively after fifth cycle of loading i.e. at 2.5 MPa applied stress. The modulus of deformation obtained at various locations justified these geological variations with values in PLT-1, PLT-4 and PLT-7 obtained as 1.817, 1.261 and 0.894 GPa, respectively. Rock class has been designated as class I, III and IV/V in PLT-1, PLT-5 and PLT-7, respectively.

V. CONCLUSIONS

Large variation in modulus of deformation of rock mass was observed in modulus of deformation values from left to right flank between chainage 60.967 m and 380.738 m at foundation level of spillway. Massive rock conditions towards the left bank were justified with higher values of modulus whereas modulus reduces significantly towards the right bank. A significant reduction of modulus values of the order of 60% was observed with variation in rock from massive to highly fractured conditions.

The variation in modulus values suggest that it is not justified to recommend unique value of modulus of deformation for use in design of the whole foundation. Each test should be seen in the light of recorded deformations and the design of structure should proceed accordingly.

REFERENCES

[1] Barton, N., Lien, R. and Lunde, J. (1974): Engineering classification of rock masses for the design of tunnel support. Rock Mech., 6, 189-236.
[2] Bieniawski, Z.T. (1973): Engineering classification of jointed rock masses. Trans. S. African Instn. Civ. Engrs., Vol., No. 15, Dec. 1973, pp. 335-344.
[3] Bieniawski, Z.T. (1976): Rock mass classification in rock engineering, Proc. Symp. Exploration for Rock Engineering, ZT Bieniawski (Ed.), AA Balkema, Rotterdam, 1:97-106.
[4] Bieniawski, Z.T. (1978): Determining rock mass deformability: experience from case histories. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. 15, 237-248.
[5] Deere, D.U. (1964): Technical description of rock cores for engineering purposes. Rock Mech. and Rock Engg., 1, 107-116.
[6] Gardner, W.S. (1987): Design of drilled piers in the Atlantic Piedmont. foundations and excavations in decomposed rock of the Piedmont Province, GSP, Ed: R.E. Smith RE, ASCE, 9, 62-86.
[7] IS 7317 (1993): Indian Standard code of practice for uniaxial jacking test for modulus of deformation of rock, Bureau of Indian Standards, New Delhi.
[8] ISRM (2007), "The Complete ISRM suggested methods for rock characterization, testing and monitoring: 1974-2006", Blue Book Edited by Reşat Ulusay and John A. Hudson
[9] Johnson, I.W., Donald, L.B., Bennett. A.G., and Edwards, J. (1980): "The testing of large diameter pile rock sockets with a retrievable test rig", Proc. 3rd Australia-New Zealand Conf. on Geomech. Wellington, 1, 105-108.
[10] Hari Dev and Rajbal Singh (2015), "An Analysis of in-situ test data for deformation modulus of weak pyroclastic rock mass", Journal of Rock Mechanics and Tunnelling Technology, Vol. 21, Number 1, January 2015.
[11] Hoek E. and Brown E.T. (1997): Practical estimates of rock mass strength, Int. J. Rock Mech. Min. Sci. GeomechAbstr., 34:8:1165-1186.
[12] Matula M. and Holzer R. (1978): Engineering topology of rock masses. Proc. of Felsmekanik Kolloquium, Grundlagen ung Andwendung der Felsmekanik, Karlsruhe, Germany, 1978, pp. 107-121.
[13] Nicholson, G.A. and Bieniawski, Z.T. (1990): A nonlinear deformation modulus based on rock mass classification. Int. J. Min. Geo. Engrg., 8, 181-202.
[14] Palmström Arild and Singh Rajbal (2001), ' The deformation modulus of rock masses - comparisons between in situ tests and indirect estimates', Tunnelling and Underground Space Technology, Vol. 16, No. 3, 2001, pp. 115 - 131.
[15] Serafim JL and Pereira JP (1983): Consideration of the geomechanics classification of Bieniawski, Proc. Int. Symp. on Engg. Geology and Underground Construction, Lisbon, Portugal, pt. II, pp. 33-44.