Stress and tunnel geometry effects on deformation modulus derived from plate load tests

Alireza Agharazi
Department of Civil & Environmental Engineering – University of Alberta, Edmonton, Alberta, Canada
Dwayne Tannant
School of Engineering – University of British Columbia - Okanagan, Kelowna, BC, Canada
Ahmad Jafari
Department of Mining Engineering – University of Tehran, Tehran, Iran

ABSTRACT
Interpretation of plate load test data commonly assumes that the rock mass is a homogenous linear elastic half space. However, when plate load tests are performed in small tunnels, the tunnel geometry affects the measured deformations and excavation-induced stress redistribution creates zones of varying rock mass stiffness around the tunnel. These effects should be considered when interpreting plate load test data. Data from the Sazbon dam site in Iran illustrates zones of increasing rock mass stiffness with distance away from a test adit and finite element modelling illustrates the importance of geometric and stress-dependent stiffness effects.

RÉSUMÉ
L'interprétation des essais de charge de plat suppose généralement que la masse de roche est demi d'espace élastique homogène. Cependant, quand des essais de charge de plat sont réalisés dans de petits tunnels, la géométrie de tunnel affecte les déformations mesurées et excavation-induit la redistribution d'effort crée des zones de rigidité de masse variable de roche autour du tunnel. Ces effets devraient être considérés en interprétant des essais de charge de plat. Les données de l'emplacement de barrage de Sazbon en Iran illustrent des zones de rigidité de masse croissante de roche avec la distance loin d'un accès d'essai et modeler fini d'élément illustre l'importance des effets géométriques et soumettre à une contrainte-dépendants de rigidité.

1 INTRODUCTION

Knowledge of the rock mass deformation modulus is important in any rock-engineering project that involves analysis of deformations, such as tunnel lining design, or dam foundation analysis. Different methods have been proposed for measuring or estimating the deformation modulus. These vary from in situ tests (Heuze & Salem, 1977) to modulus estimation using rock mass classification systems (Hoek & Diederichs, 2006).

The accuracy and reliability of in situ tests depend on the quality of test execution and consistency of the theoretical assumptions with the real rock mass conditions. Plate load tests, dilatometer tests, and flat jack tests are often used in rock engineering projects. Back calculation of tunnel convergence measurements involves a much larger volume of rock and this technique is sometimes used.

Performing in situ tests is generally very expensive and time consuming. Hence, these tests are performed when more precise and reliable estimation of deformation modulus is required. Interpretation of in situ tests results is usually based on the theory of elasticity. The rock mass is often assumed to be a continuous, homogeneous, isotropic, linear elastic, (CHILE) medium. Inconsistency between these assumptions and the actual characteristics of the rock mass is an important source of error in the interpretation of in situ test results.

In this study, the plate load test is reviewed and the effects of confining stress level and the size of a test tunnel on test results are presented using a finite element model and results from plate load tests performed at the Sazbon dam site in Iran.

2 PLATE LOAD TEST

The plate load test is a common test for determining the rock mass deformation properties in large dam construction projects. This test involves loading of two opposite walls of an exploration adit by hydraulic jacks and loading plates and measuring the induced deformations at surface and at depth (Figure 1).

Two types of this test can be distinguished based on the method of applying the load to the rock. Flexible loading by a flat jack causes a ‘uniform stress’ boundary condition, while hydraulic jacks and rigid plates produce a ‘uniform displacement’ at the plate-rock contact. The assumption of flexible loading is most applicable for stiffer rock masses while rigid loading conditions can be achieved in softer rocks and soils. Based on Lama and Vutukuri (1978), the loading plates should be at least two times stiffer than the rock mass for the rigid loading assumption to be valid.

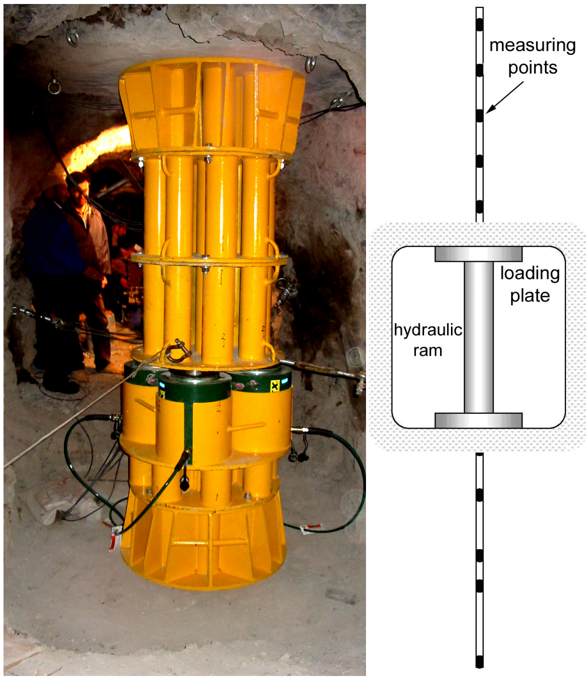

Figure 1. Plate load test at Bakhtiari dam site in Iran

Deformation measurements are usually taken at different depths beneath the load plate. Relying only on surface measurements of deformation can give erroneous estimation of modulus because of the disturbances produced in the rock close to surface during the excavation and test site preparation stages.

The maximum applied load is usually achieved after several, e.g. five, loading–unloading cycles. The intermediate cycle load level is usually selected to approximate the anticipated design load and last cycle load, i.e. maximum load, is set to approximately 1.5 to 2 times of the design load.

According to the report of the 1975 ISRM commission on terminology (Unal, 1997), two alternative definitions for rock mass modulus are:

- modulus of deformation, E_m = ratio of applied stress to induced strain including both elastic, W_e and non-elastic, W_d deformation (Figure 2)
- modulus of elasticity, E_{em} = ratio of applied stress to induced strain including just elastic deformation, W_e.

It is common to calculate the modulus of deformation from plate load data as this is a more relevant parameter needed for design when estimating dam foundation settlements during construction and impounding.

Rock mass modulus often is calculated based on measured deflections, using Boussinesq's equation for a semi-infinite, isotropic, homogeneous, elastic medium, loaded by a distributed pressure on the boundary. Assuming a uniform stress boundary condition, the following equation is used (ISRM 1979):

$$E = \frac{zq(1+\nu)}{w}\left[1 - z\left(a^2+z^2\right)^{-\frac{1}{2}}\right] + \frac{2q(1-\nu^2)}{w}\left[\left(a^2+z^2\right)^{\frac{1}{2}} - z\right]$$

where:

w = deflections measured along axis of plate
z = depth at which deflection measurement is made
q = applied stress at the rock face
ν = Poisson's ratio
a = loading plate radius
E = modulus.

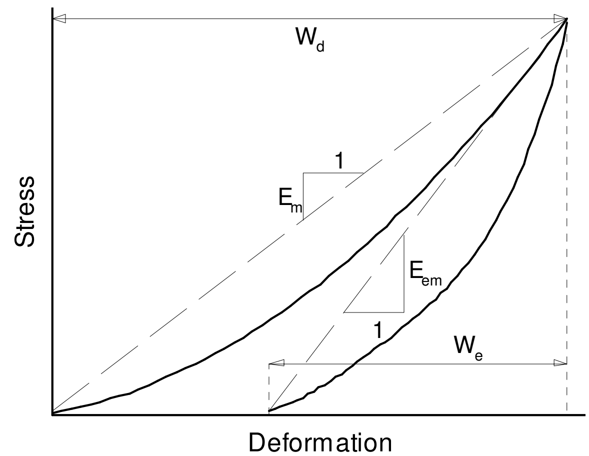

Figure 2. Generic plate load test curve showing elastic and total deformation and corresponding moduli

3 TEST RESULT INTERPRETATION

The interpretation of plate load test data assumes a semi-infinite medium, while in practice, plate load tests are performed in small exploration adits. The geometry and dimensions of the adit relative to the size of the loading plate, causes the measured displacements to be smaller than they would be if a semi-infinite medium existed. ISRM (1979) and Boyle (1992) suggest that the diameter of the prepared rock pad should be at least 1.5 to 2 times of loading plate diameter to reduce this effect. A study carried out by Van Heerden and Maschek (1979) using a finite element model of a plate load test showed that this ratio should be at least 6 to make this effect negligible. In practice, the adits usually have dimensions smaller than 3 m, so when using a 1 m diameter loading plate, this ratio can be 3 at most.

Palmstrom and Singh (2001) and Serafim and Guerreiro (1968) reported that measureable induced deformations usually occur to a depth of about three times of the loading plate diameter at a stress level of 5 to 6 MPa. However, in a massive stiff hard rock, the deformations can drop below the resolution of the measurement devices at even shallower depths.

Factors that introduce errors in plate load tests can be classified into two main categories. First are factors related to the quality of test execution including resolution of measurement equipment, quality of the site preparation and its effect on the in situ condition of rock mass, and proper installation of test apparatus.

The second category includes the factors that introduce calculation errors that are related to the differences or discrepancies between theoretical assumptions of the test and the real site conditions. In many cases in practice, assumptions of semi-infinite homogeneous medium and isotropic linear elastic behaviour are not met. These factors can be classified as "theoretical factors", which are independent of the test execution quality.

Of the factors related to the test procedure, the disturbance of the rock at tunnel periphery due to excavation-induced damage or stress release have a dominant effect on the test results. Singh and Rajvansi (1996) studied this factor by conducting a series of plate load tests in Lakhwar dam project in India. In this study, two plate load tests were conducted on the same rock mass, one in an undisturbed section of the adit, where no blasting had been used, and the other one in a nearby blasted section. Comparison of these results proved the considerable effect of blasting damage on plate load test results (the blasted section had moduli that were 5.4 to 9.6 times lower than the other section). To minimize the effect of excavation disturbance on calculated moduli, Palmstrom and Singh (2001) recommend not using displacements measured within 0.5 to 0.8 m of the rock surface.

The effect of theoretical factors on calculated moduli can vary from negligible to considerable depending on the level of consistency between assumptions made and real rock mass conditions. For example, in a layered sedimentary or foliated rock mass, the assumptions of isotropy and sometimes linear elasticity are not met. In layered rock masses, with beddings or other joint sets, the non-linear behaviour of the joints dominate the overall behaviour of the rock mass. This non-linear behaviour makes rock mass modulus a stress level dependent property and gives rise to confusion and uncertainties in interpretation of plate load test results. For a non-linear rock, no unique modulus exists.

4 STRESS DEPENDENCY OF ROCK MASS MODULUS

The ISRM (1979) suggested equation for calculating the rock mass deformation modulus is based on the theory of elasticity for a linear elastic homogenous medium. This assumption necessitates having the same modulus values at different depths (different stress levels), but this is not the case in many plate load tests, as an increasing trend of moduli with depth is observed.

Several authors have pointed out this stress dependency of modulus in jointed rock masses. Based on the back calculation of the tunnel closure measurements, Verman et al. (1997), showed the dependency of the rock mass modulus on depth of overburden. They concluded that modulus increases with increasing confining stress and this stress dependency is more pronounced in weaker rocks and is almost absent in strong and brittle rocks.

Boyle (1992) examined this increasing trend of modulus with depth by simulating plate load test with a finite element model. He reported an increase of modulus with depth, calculated using ISRM suggested method even for a linear elastic model. He proposed a statistical approach to obtain a unique modulus value for a given plate load test. However, his approach is basically an averaging (least square) of the values calculated by ISRM suggested method. Consequently it suffers from the same defects and does not address the nature of problem.

Stress dependency of the modulus is an inherent property of many jointed or fractured rock masses that should be taken into account. This stress dependency originates from the non-linear deformation behaviour of the discontinuities present in the rock mass. Figure 3 shows the behaviour of rock joints established by Singh (2000) based on physical models. The joint becomes stiffer as the normal stress acting on the joint increases.

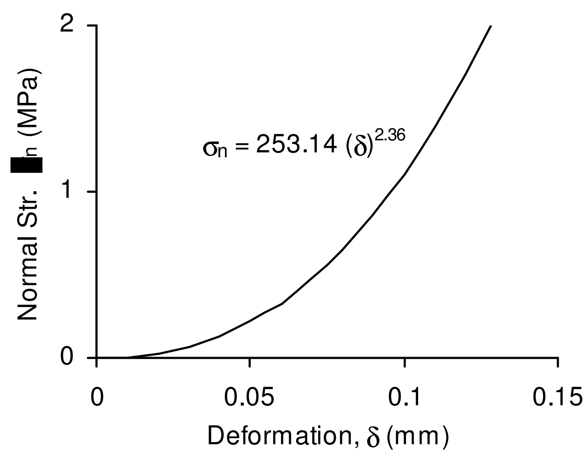

Figure 3. Non-linear deformation response of joints based on physical model developed by Singh (2000)

In weak and jointed rock masses, non-linearity may be caused by non-linear response of intact rock and joints. In an exploratory adit where plate load tests are usually conducted, the confining stress at the adit periphery is low due to stress release resulting from excavation and gradually increases to its in situ state with increasing distance from excavation wall (Figure 4). The zone of stress relief around the adit is coincident with the zone that is loaded during a plate load test. The variation of confining stress in this zone results in a variation of joint stiffness and consequently creates a situation where rock mass moduli increases with depth from the adit.

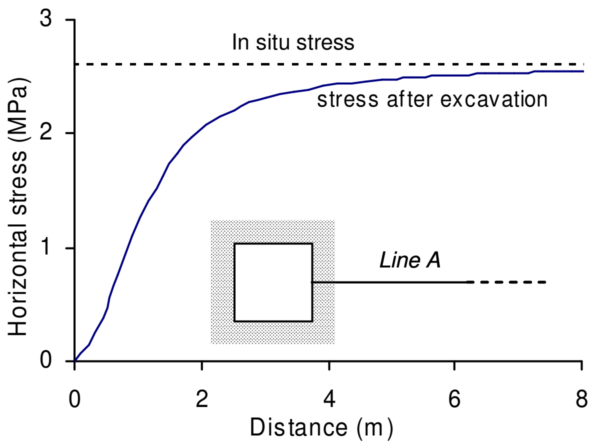

Figure 4. Horizontal stresses along line A obtained from a Phase2 finite element model of a 2 m square excavation in a CHILE rock subject to lithostatic stress

A similar phenomenon is sometimes observed when the plate load test is conducted in cycles of increasing load. At higher loads, the joints become stiffer and the calculated rock mass moduli increases.

The stress dependency of rock mass modulus is only one mechanism to explain the observed increase in modulus with distance away from a loading plate. Plate load tests conducted in adits have geometric boundary conditions that differ from the elastic half-space model implicit in the ISRM (1979) approach to interpreting the test data that results in apparent increases in moduli away from the loading plate.

5 PLATE LOAD TESTS AT SAZBON DAM

The Sazbon dam site is located in Ilam province of Iran on crystalline limestone of the Asmari formation, which is classified as strong and lightly jointed rock. As part of a comprehensive rock mechanics study program, 10 plate load tests were conducted in exploratory adits that had an approximate span of 2 m and were excavated in the dam abutments.

One metre diameter plates and eight hydraulic jacks were used to apply the load to the rock. Displacement measurements were made at surface and four points at depth using multiple point borehole extensometers, with measuring resolution of 0.002 mm. The displacement measurements were taken with respect to a 6 m deep anchor, which was assumed to be fixed.

Five load increments were applied to a maximum stress of 10 MPa, followed by a creep test at the maximum load (Figure 5).

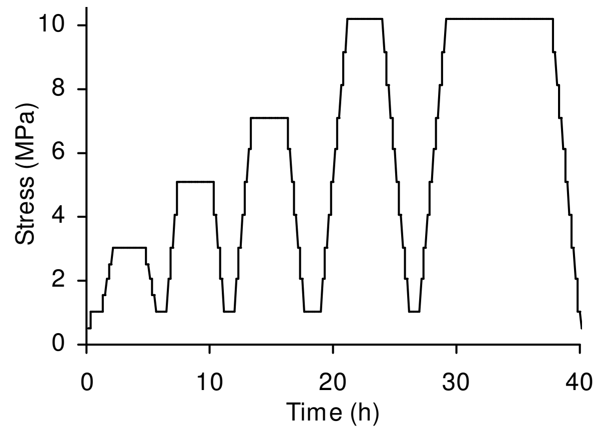

Figure 5. Stress – time curve for a plate load test at the Sazbon dam site

Figure 6 shows the stress versus deformation curves measured at five locations beneath the loading plate. The non-linearity observed for the surface deformations may be attributed to excavation-induced disturbances near the adit periphery as well as stress-dependent stiffness changes. Non-linear behaviour almost disappears for the measurements taken at depth, which indicates that the excavation disturbances are limited to the rocks located between adit wall and the first measurement depth (0.5 m). For any given measurement point, the ratio of deformation moduli, i.e. the slope of stress-deformation curves, of subsequent loading cycles can be used as an indicator of hardening response of the rock mass to increase of applied load. Any ratio of greater than unity indicates increase of the rock mass stiffness with the level of applied load. For the surface measurement point, the ratio of the deformation modulus calculated in cycle two to the modulus calculated in cycle one is $E_2/E_1 = 2.6$, while this ratio is 1.4 for the measurements taken at depth. For these points, this ratio trends to unity for the last four loading cycles, which is characteristic of undisturbed lightly jointed to massive hard rocks.

Lower values of deformation moduli calculated at surface, can be attributed to the closure of cracks generated or opened at adit periphery during excavation and site preparation stage. However even at the surface, after two loading and unloading cycles, a close to unity ratio is achieved for the moduli calculated in last three cycles (E_4/E_3 & E_5/E_4).

In the calculation of deformation modulus at the Sazbon dam site, the ISRM (1979) suggested equation for uniform stress loading is used. Poisson's ratio is assumed as 0.25 for the rock mass. In these calculations, a diameter of 100 mm has been considered for a central borehole through the loading plate, which was used to access the extensometer borehole.

An increase in the deformation moduli with depth (Figure 7) is observed for the test results shown in Figure 6. The calculated modulus based on the deflection measurements taken at deepest point, i.e. 2.5 m, is approximately 35 GPa, which is about 6 times higher than the calculated moduli for the surface measurements. A similar trend occurs for other plate load tests done at the Sazbon dam site.

As mentioned earlier, geometry effects and stress-dependency of stiffness are two important factors that can cause the calculated moduli to increase with depth below the loading plate. For the Sazbon plate load test, the relative significance of geometric effects or stress-dependent modulus effects is unknown. In the volume of rock that has been affected by the test, a significant stress-dependent modulus response is not expected because few joints were observed in the rock mass around the test adit. Thus, it is likely that the observed increase in deformation moduli with depth is associated with violations of the semi-infinite medium assumption. One way to explore these two effects is to perform finite element modelling as discussed in the next section.

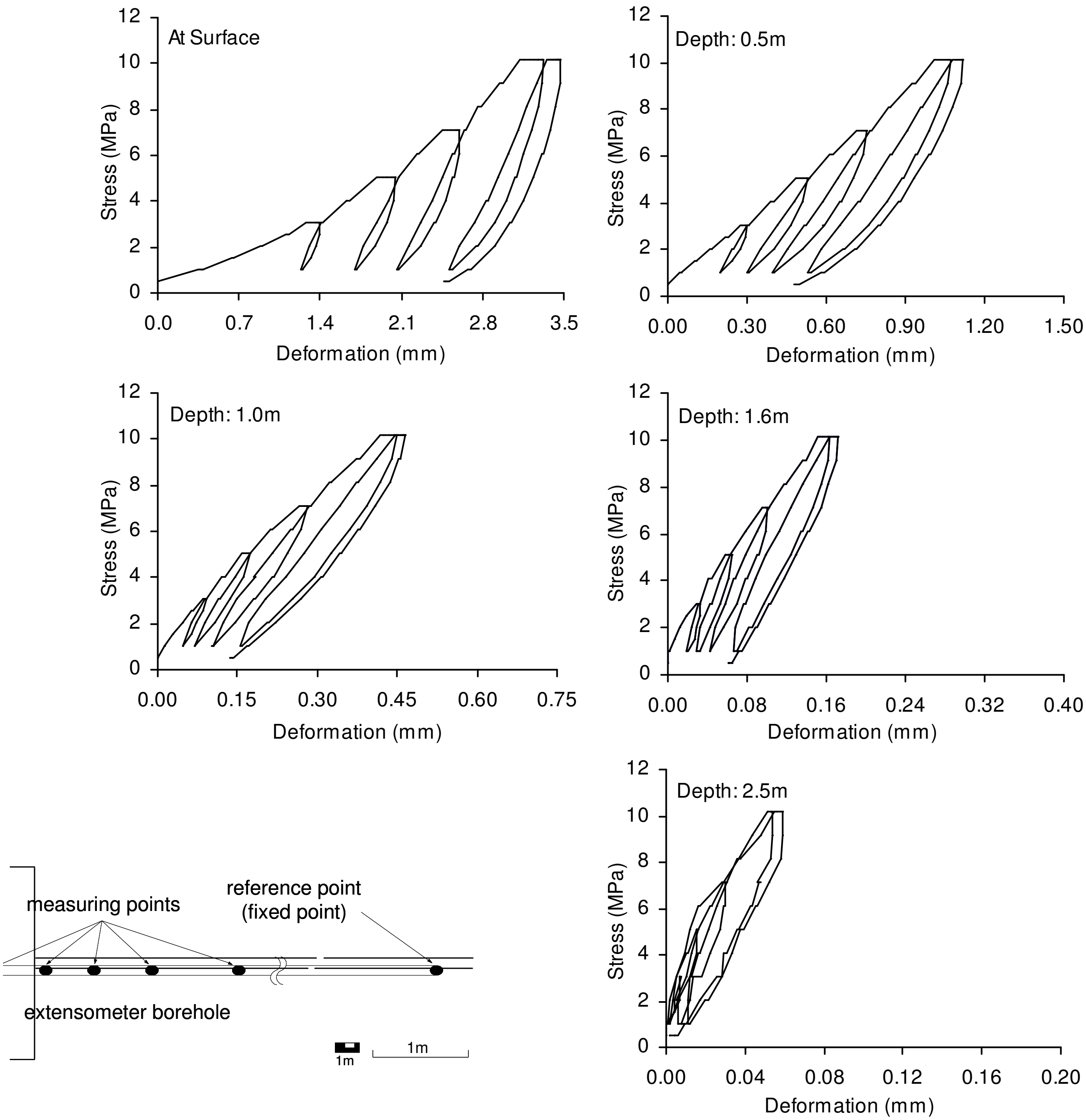

Figure 6. Plate load test stress–deformation curves for a test conducted at Sazbon dam site

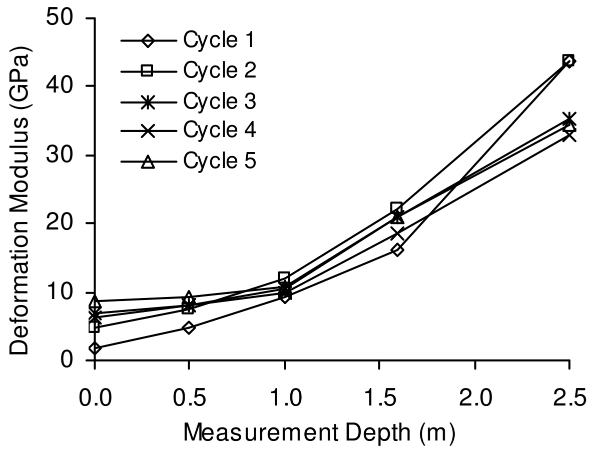

Figure 7. Increasing modulus with depth for a plate load test at Sazbon dam site

6 FINITE ELEMENT MODELING

Finite element modelling was performed to illustrate the effects of tunnel geometry and stress-dependent moduli on the measured displacements during a generic plate load test. Displacements obtained from the models were used with the ISRM (1979) equation to calculate the modulus. The calculated moduli from different models were then compared with the actual moduli used in the models. The modelling results presented here are loosely based on the Sazbon plate tests.

Phase2 (RocScience, 2008) was used to construct axisymmetric models with a 1 m diameter circular loading plate simulated by a uniform applied stress of 15 MPa. Three different axisymmetric models were created (Figure 8). The boundaries of the finite element mesh were extended far enough from the loading area to neglect boundary effects on the results. The model dimensions were 100 m radius by 200 m height for models 1 and 2. Model 3 had a radius of 50 m and a height of 100 m.

The first model simulated a homogeneous, isotropic, linear elastic half space loaded by a circular flexible plate. In this model, the rock's modulus of elasticity and Poisson's ratio were 15 GPa and 0.25, respectively.

The second model had the same geometry as the first model but it incorporated five 1 m thick layers of rock with progressively lower moduli toward the loading plate. The softest rock (E = 2 GPa) was located within 1 m of the rock surface and the stiffest rock (E = 15 GPa) was located below depths of 4 m in the model. The intent in this model was to simulate a stress-dependent modulus without explicitly linking the state of stress around the tunnel to the rock mass modulus through a specific functional relationship. This model might represent the case of a rock mass with one joint set oriented parallel to the loading plate.

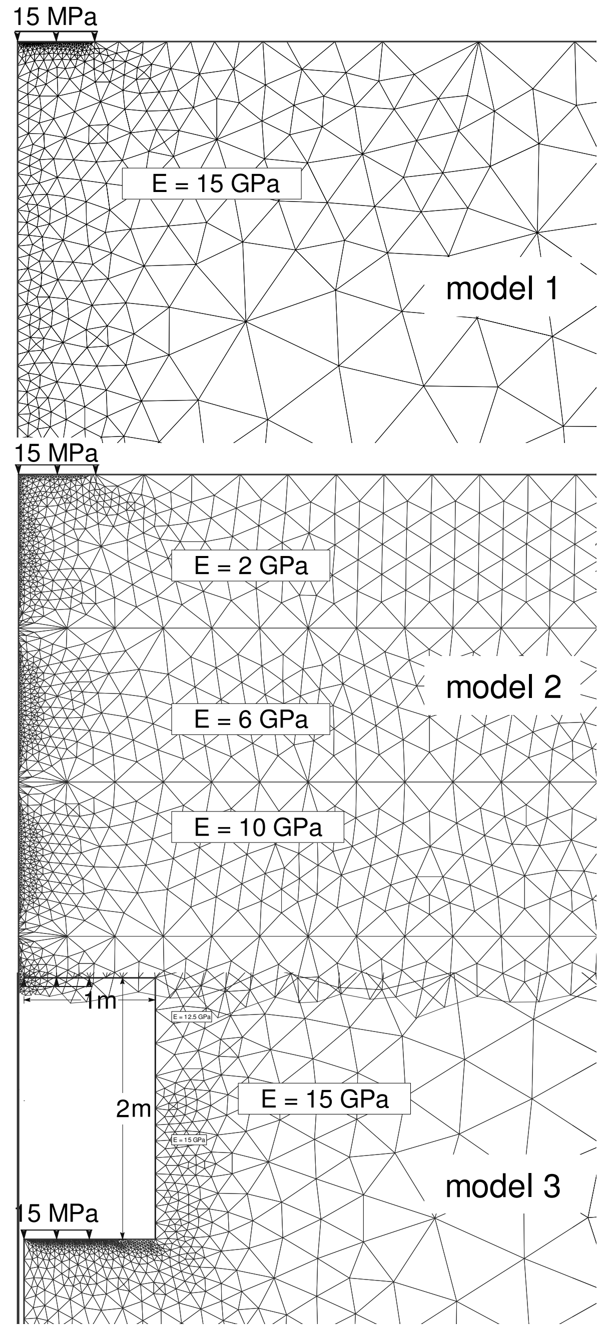

Figure 8. Axisymmetric finite element models (only portion of model closest to loading plate is shown)

The third model used a constant modulus of 15 GPa everywhere but the model incorporated the approximate effects of different excavation span to loading plate diameter ratios. The model was used to examine the influence of geometric effects on the interpretation of plate load test displacement data.

Figure 9 compares the assigned modulus distribution in models 1 and 2 with the rock mass modulus calculated using the ISRM (1979) suggested equation. When an elastic half-space model with constant modulus is used (model 1), the calculated moduli using displacements from the model are slightly higher than the value that was assigned to the model. A possible reason may be associated with the size of the model and its boundary conditions.

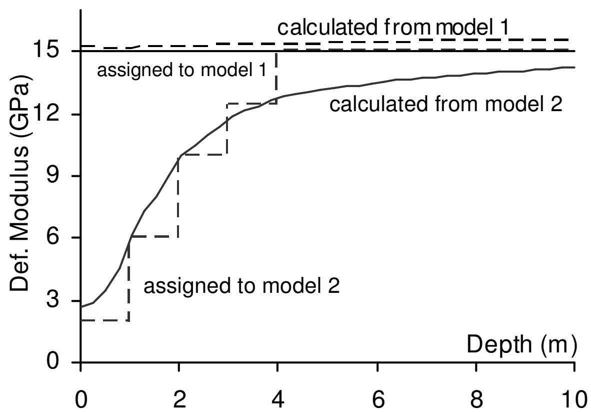

Figure 9. Variation of modulus with depth for plate load test using models 1 and 2

When zones of increasing stiffness away from the rock surface are used in the model (model 2), the ISRM suggested relation over-estimates the modulus for near-surface rock layers and at some point this trend turns to under-estimation of the modulus for deeper rock layers. In this model, the rock has the same elastic properties in tension and compression. Decreasing the modulus of rock near the surface causes more tensile deformations sub-parallel to rock layers and transfers more stress to deeper rocks. More stress transferred to deeper layers in model 2 causes the larger deformations in the deeper rock layers compared to model 1 and consequently under-estimation of modulus for that rock.

In practice, the existing discontinuities in the rock mass limit development of tensile stresses and strains in the rock mass. This will further influence the distribution of deformations beneath a loading plate and thus the calculated moduli.

Figure 10 compares the assigned modulus in model 3 with the rock mass modulus calculated using the ISRM (1979) suggested equation. The calculated moduli are higher than those assigned to the model because the geometry of the model affects the displacements that occur beneath the plate. As the excavation size increases relative to the plate size, the model approaches a half space geometry and thus the calculated moduli become closer to the assigned rock mass modulus. For typical ratios of tunnel width to loading plate diameter (2 to 3) the influence of the tunnel geometry on the measured displacements below the plate is significant. Clearly the assumption of an elastic half space associated with the ISRM (1979) suggested equation is easily violated when performing plate load tests in adits and calculated moduli using this relationship can be much higher than their true values.

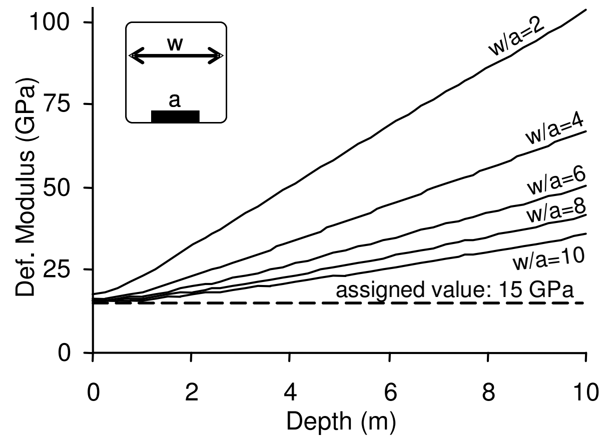

Figure 10. Variation of modulus with depth for plate load test using model 3 with varying excavation sizes

7 DISCUSSION AND CONCLUSION

At dam sites, plate load tests are usually performed in small exploration adits. The calculated rock mass moduli are then used for analysis of structures of different geometry and under different in situ and induced stress conditions, such as dam foundations or tunnels. For a dam foundation, confining stresses are usually low so non-linear rock mass behaviour can be anticipated resulting from progressive closure of open discontinuities as load increases. On the other hand, for a deep tunnel excavated in the same rock mass, discontinuities should be tight and consequently a stiffer rock mass modulus with only minor non-linearity is likely to prevail. Hence, in situ or laboratory test programs should be designed and aimed at defining stress-dependent moduli for the rock mass instead of a unique value.

The interpretation of plate load test data needs to consider the effects of the excavation geometry. The displacements occurring below a loading plate can be influenced by the size and geometry of the adit in which the tests are performed and interpretation of these displacements to determine the rock mass modulus should account for the excavation geometry. Simply assuming an elastic half space can yield moduli that are higher than the true values. The use of numerical models incorporating the excavation geometry is recommended as a means to account for the geometric influences on the displacements. The model can be used to back-calculate the modulus or modulus distribution that best matches the deformations measured at different depths.

The stiffness of a rock mass containing discontinuities depends on the state of stress acting on the rock mass. The relationship between stress and rock mass stiffness is non-linear. The presence of an exploration adit creates an altered stress distribution around the excavation and hence the rock mass moduli are expected to vary at different locations around the adit too. Low confining stress occurs near the adit periphery and hence lower rock mass modulus can be expected at this location. Non-linearity in deformation response is an inherent property for a discontinuous rock mass and should be considered in interpretation of plate load test data.

The ISRM suggested equation for calculation of rock mass modulus does not allow recognition and separation of the excavation geometry or stress-dependent stiffness effects. Hence, determining the rock mass modulus from plate load tests conducted in exploration adits using this approach may result in erroneous results.

ACKNOWLEDGEMENTS

Mahab Ghodss Consulting Engineers Co. and Khak & Sang Geotechnical Consulting Co. provided the Sazbon dam site plate load test results.

REFERENCES

Boyle W.J. 1992. Interpretation of plate load test data. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.,* **29**(2): 133-141.

Heuze F.E. & Salem A. 1977. Rock deformability measured in situ - problems and solutions. Proc. *Int. Symp. on Field Measurements in Rock Mechanics*, Kovari (Ed.), **1**: 375-387.

Hoek E. & Diederichs M.S. 2006. Empirical estimation of rock mass modulus. *Int. J. Rock Mech. Min. Sci.*, **43**: 203-215.

ISRM 1979. Suggested methods for determination in situ deformability of rock. International Society for Rock Mechanics, Commission on Standardization of Laboratory and Field Tests. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.,* **16**(2): 143-146.

Lama R.D. & Vutukuri V.S. 1978. Handbook on Mechanical Properties of Rocks, Vol. 3, Trans Tech Publications.

Palmstrom A. & Singh R. 2001. The deformation modulus of rock masses – comparisons between in situ tests and indirect estimates. *Tunnelling and Underground Space Technology*, 115-131.

RocScience. 2008. Phase^2 6.0 user guide and tutorial

Serafim, J.L., Guerreiro, M., 1968. Shear strength of rock masses at 3 Spanish dam sites. Proc. Int. Symp. Rock Mech., Madrid, pp. 147-157

Singh M. 2000. Applicability of a constitutive model to jointed block mass. *Rock Mech. Rock Engng.*, **33**(2): 141-147.

Singh R. & Rajvansi U.S. 1996. Effect of excavation on modulus of deformation. Proc. *Conf. on Recent Advances in Tunnelling Technology*, 57-67.

Unal E. 1997. Determination of in situ deformation modulus: new approaches for plate loading tests. *Int. J. Rock Mechanics Min. Sci. & Geomech. Abstr.*, **34**(6): 897-915.

Van Heerden W.L. & Maschek R.K.A. 1979. In-situ modulus determination, a case history, Elandsberg Scheme, South Africa. Proc. *4th Congr. Int. Society of Rock Mechanics*, 705-710.

Verman M., Singh B., Viladkar M.N. & Jethwa J.L. 1997. Effect of tunnel depth on modulus of deformation of rock mass. *Rock Mech. Rock Engng*, **30**(3): 121-127.