Determination of Mass Moduli for Slope Design

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SUMMARY Cl assification of rock masses is correlated with in situ moduli measured from jacking tests and theoretically derived moduli using composite elastic theory. A means of improving the in situ moduli by rock bolting is then considered. In order to assess the effectiveness of the remedial measure pseudo elastic theory is used to analyse the specific problem of slope stability. The results of the moduli changes due to rock bolts are checked by in situ jacking tests.

1 INTRODUCTION

This paper deals with the stability of soft-hard rock slopes where systems of joints and faults control the behaviour of the slopes. If the material is a soil or soft rock the various plastic design approaches varying from the Swedish Slip Circle to Janbo's wedge analysis are generally used. If the material is a hard intact rock an elastic design approach is used. In turn, if the material is neither plastic nor elastic in its deformational response, an elastic plastic analysis or pseudo plastic or pseudo elastic approach is used. When these latter methods are used the definition and determination of deformation moduli require careful consideration. Namely because in the deformational process the mechanisms of slip and rotation occur between and across the joint and fault systems. However, when stress or load gradients are minimal, Chappell (7), these latter mechanisms are also minimal and the representation of the jointed rock mass as a composite material in pseudo elastic theory is valid.

It is generally recognized that the deformational moduli obtained from relatively small intact rock samples are very much different to the deformational moduli of the jointed rock mass from which the intact rock samples are retrieved. Consequently, in situ testing techniques developed to measure the mass deformational moduli, are an important and essential part of the investigation program. In order to determine the moduli defining the constitutive relation some assumption or coefficient is applied to the measuring instrument, be it a flat jack, dilatometer, or plate bearing device. This coefficient is required in that it defines the boundary conditions and when used in conjunction with the elastic moduli gives the stress distribution. In a continuous or discontinuous material the constitutive relation is defined by the elastic modulus or pseudo elastic modulus. Consequently these moduli are very much dependent on the boundary interactions, and if the boundary conditions are ill defined and the mechanism causing deformation undefined then the evaluated coefficient of the measuring instrument is suspect.

Here the deformational response of the rock slopes is neither elastic nor plastic. With these aspects in mind a simple field test is devised to measure the deformational moduli of the jointed and faulted rock masses which have zones of hard and soft intact rock. Many of the slopes excavated in this material had failed and others were still to be excavated. Remedial measures to ensure the stability of existing and future slopes required careful assessment and design. A comparison of the measured moduli before and after slope stabilisation was the control criterion used to measure the effectiveness of the remedial measures and confirm the design process.

The rock material where the in situ moduli were measured was classified and collated with the various geological structural zones. From this the designed remedial rock bolt patterns for specific rock mass classes were allocated and documented. Consequently, the rock mass type encountered in the field was easily classified and the appropriate slope treatment defined.

2 SLOPE DESCRIPTION

2.1 Material Types

The geology of the site is not reported in detail here but the general characteristics of the material and profile of the slopes are given.

Because of the complexity of the geology and material in which the slopes are excavated it is important to classify the different rock zones encountered. An important addition of the classification used here was the measurement of the deformational response. From this the required remedial measures in the form of rock bolt patterns were determined by using a pseudo elastic design process. Three classification systems were considered namely those described by Barton et al (2) Bieniawski (3) and Wickham (4). Of these Bieniawski's RMR, rock mass rating, was used to define and assess the behaviour of the structural rock zones related to the excavated slopes.

The area in which the slopes were excavated is Jurrasic in age and is highly crushed and contorted. Low grade pressure and temperature metamorphism occurs which causes a wide variation in the material types and fault-joint systems. The resultant materials are derived from sandstones and basalts giving quartzitic sandstone, greenstone, jasper and calcite. Superimposed on these metamorphosed material types are the geological structural features and resultant gouge material. The gouge is a breccia and mylonite material with seams of calcite, chlorite, kaolinite and montmorillonite being quite common.

When classifying both the material and structural zones (1), the rock mass is defined on a rating scale of class 1, 2, 3, 4 or 5 which in descriptive terms is excellent, very good, good, bad and very bad respectively. The slopes considered here were excavated in rock masses defined as class 4 and 5 with some class 3 in the upper regions of the slope. The heights of the slopes varied from 50 m to 130 m.

2.2 Material and Slope Profile

In many situations in practice the rock profile is generally such that the strength and deformational characteristics of the rock mass improve with depth. This is accepted as being consistent with the weathering profile and stress environment generally encountered. Here, however, there is a general improvement to a depth of about 30 m and then below this depth the strength and deformational characteristics markedly decrease with an increase in depth.

This means that as the depth of excavation is increased the toe of what was a stable slope becomes softer and deforms more readily. Though the toe of the slope does not necessarily fail, excessive deformation occurs. This excessive deformation causes the loosening of the joint and fault system in the stiffer upper regions of the slope and this in turn causes unravelling and local wedge failures. From this general slope failure ensues. A typical excavation profile is depicted in Figure 1.

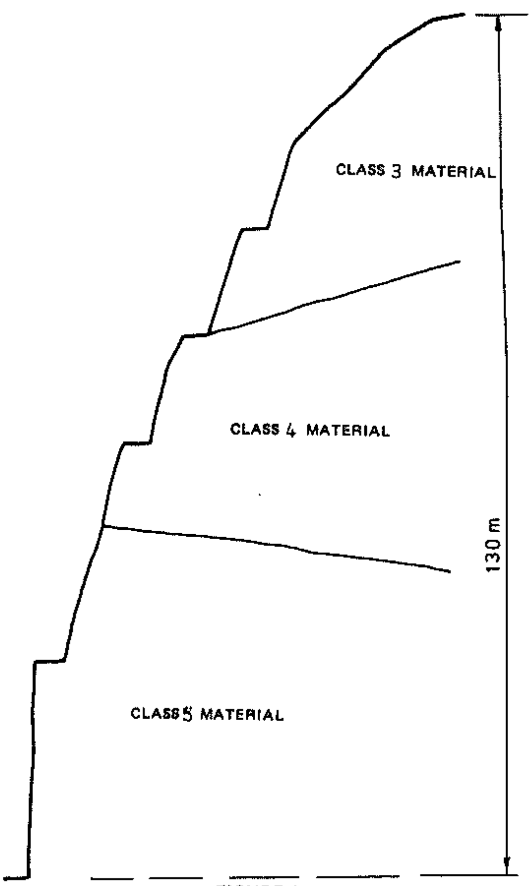


FIGURE 1

TYPICAL SECTION OF SLOPE

In order to prevent a recurrence of these events some means of reducing the deformational response especially in the region of the toe is required. Rock bolts which impose a confining stress and kinematic restraint on the rock mass not only reduces the deformational response but also markedly increases the strength of the rock mass. Though deformation and strength are inextricably related it is felt especially in a discontinuous material that an understanding of the deformation process must be acquired before the strength can be defined.

With these aspects in mind a series of ungrouted rock bolt plate bearing type tests are devised and performed where the deformational response of the loosened and then prestressed rock mass is measured.

It should be noted that the prestressed and then subsequently grouted bolt serves in the main two functions. As an active prestressed bolt the normal forces across the joint sets are increased and consequently the magnitude of the deformational modulus is increased. In addition as the depth of excavation is increased the magnitude of deformation of the rock mass is inhibited by the passive interaction of the grouted bolt within the jointed rock mass.

3 MODULI DETERMINATION

3.1 Deformational Moduli

In order to perform a pseudo-elastic plastic analysis knowledge of the deformational response and strength parameters is required. The deformational response is given in terms of a constitutive relation. Smart (5) and Singh (6) and others consider the discontinuous rock material as a multi-phase composite material. Deformational mass moduli are evaluated using elastic multi-phase continuous models. These approaches are fraught with many dubious assumptions (7) especially if the mechanisms of slip and rotation are occurring. By using the criteria of compatibility and equilibrium Hill (8) shows that the upper and lower bounds of deformational moduli for an elastic multi-phase continuum are obtained. In turn by measuring the mass moduli in situ plus composing the mass composite moduli from component parts and then comparing these results with the upper and lower evaluated bounds an appreciation of discontinua is obtained. This latter approach is valid when the mechanisms of slip and rotation are absent and this is so if the stress or load gradients are small.

By knowing the separate deformational response of the two phases making up the jointed rock mass namely the intact rock and joint material the mass deformational modulus is compiled (9). These material characteristics related to deformational response and strength were determined by standard laboratory techniques. While performing the joint tests both normal and shear stiffnesses are also measured. In addition to the above information the core retrieved from the rock bolt moduli test holes was classified.

The in situ deformational response of the jointed rock mass was measured by noting the movements of both the anchor and face plate of a loaded hollow rock bolt. This allowed the determination of the deformational modulus of both the destressed surface rock and inner confined rock mass. The difference between the magnitude of these two moduli gives the effect of the stress environment in a jointed rock on the deformational response. Initially, as the rock mass making up the slope is unloaded and as there is no surrounding rock bolt which pre-loads or kinematically constrains the contiguous rock mass, the mass deformational response relates to an unloaded rock mass. After determining the unloaded deformational response the region was preloaded by loading a nearby rockbolt and the deformational response at the same location was repeated. This gives the decrease in deformational response due to the active preloading across the joint systems. It does not measure the kinematic passive constraint inhibiting dilation of the rock mass as further deformation of the rock mass occurs.

3.2 Joint Moduli

Using the Hoek direct shear box machine the apparent cohesion and friction angle of the joints were measured. In addition to this the shear stiffness was determined. Tables 1 and 2 show the results of the intact and joint properties plus stiffnesses for the joint system.

Sample No.	Intact Modulus Ei GPa	Apparent Poisson's Ratio	Unconfined Comp. Str. MPa	Description
IC–1	55.6	0.32	75.7	Greenstone
IC–2	38.0	0.23	85.6	Greenstone
3C–1	84.0	0.28	87.5	Greenstone
4C	49.3	0.29	74.5	Greenstone
6C	71.0	0.19	15.5	Greenstone
8C	23.0	0.51	38.2	Jasper
9C	13.0	0.98	10.0	Jasper
2C–2	32.1	0.24	55.0	Quartzite
3C2	55.5	0.21	64.5	Quartzite

TABLE 1
Intact Rock Properties

E_V upper and E_V lower are the upper and lower bound composite moduli,
E_1, E_2, etc. are the component moduli of phases 1, 2, etc.
V_1, V_2, etc. are the percentage volumes of phases 1, 2, etc. in relation to the total volume considered.

Upper and lower bound moduli are given in Table 3 for the various frequencies of joints per metre thickness of material.

3.4 In situ field moduli

There are a number of ways of determining the deformational response in a discontinuous rock mass. It is difficult, however, to achieve consistent or repeatable values of deformational response from the various methods generally used (10), (11), (12). From experience gained measuring the deformational response with devices such as flat jacks, plate

Test No.	Description	Location	Normal Stress MPa	Residual Angle of Fric	Normal Stiffness MPa/mm	Shear Stiffness MPa/mm
1	Greenstone on Greenstone	South side	8.0	35	3.86	2.7
2.	Greenstone with Chlorite infill	South side	4.76	19	2.44	0.84
3	Chlorite on Chlorite	South side	2.0	19	2.7	0.93
4	Jasper on Jasper	South side	3.7	29	0.83	0.46
5	Chlorite on Chlorite	South side	2.9	17	5.56	1.7
6	Greenstone on Greenstone	North side	1.9	19	2.45	0.75
7	Greenstone on Jasper	North corner	4.62	12	5.6	1.2
8	Jasper on Jasper	Centre	3.33	26	3.03	1.48

TABLE 2
Joint Properties

3.3 Combined Moduli

When combining the component parts of multi-phase material by composite elastic theory an important assumption is that normal loads do not induce shear forces. This does not apply in a jointed material where the joints are in any way staggered, (7). This however, is not significant if the mechanisms of slip and rotation do not occur.

The formula used for determining the upper and lower bound deformational moduli are evaluated from (7),

$$E_{v\,upper} = E_1V_1 + E_2V_2 + \cdots \qquad (1)$$

$$\frac{1}{E_{v\,lower}} = \frac{V_1}{E_1} + \frac{V_2}{E_2} + \cdots \qquad (2)$$

bearing, overcoring, extensometers and pressuremeters it was decided to use a rock bolt type plate bearing test, or often termed the jacking test.

The test performed here used a hollow rock bolt anchored from 3 m to 9 m into the rock mass, Figure 2. Using an oil jack the rock bolt is loaded and the deformations of both the bearing face plate and anchored extension rod are measured, Figure 3. From this the deformation of the rock loaded between the face plate and anchor is known. Initially, difficulty was experienced in achieving adequate anchorage in the class 4 and 5 rock. This was overcome by creating a cavity at the anchor end with a small quantity of explosive using a detonator. Good anchorage was achieved and the size of anchorage was determined by measuring the quantity of grout used for anchorage and the depth of drill hole before and after grouting the anchor.

Intact Material Type	Intact Mod. GPa	Joint Type	Int Thick mm	Int Mod. MPa	1 Joint/m Eupp	1 Joint/m Elow	1 Joint/m Eu/El	5 Joints/m Eupp	5 Joints/m Elow	5 Joints/m Eu/El	10 Joints/m Eupp	10 Joints/m Elow	10 Joints/m Eu/El
Greenstone	60	Greenstone	2.5	8	59.85	3.04	19.7	59.25	0.63	93.3	58.5	0.32	184
Greenstone	60	Chlorite	5	3	59.7	0.59	100	58.5	0.12	488	57.0	0.06	950
Greenstone	60	Jasper	10	0.5	59.4	0.05	1 188	57	0.01	5 700	54.0	0.005	10 800
Jasper	18	Greenstone	2.5	8	17.96	2.72	6.6	17.78	0.62	28.8	17.55	0.315	56
Jasper	18	Chlorite	5	3	17.91	0.58	30.8	17.55	0.12	147	17.1	0.06	286
Jasper	18	Jasper	10	0.5	17.82	0.05	356	17.1	0.01	1 710	16.2	0.005	3 240

TABLE 3
Ratios of Upper & Lower Bound Moduli

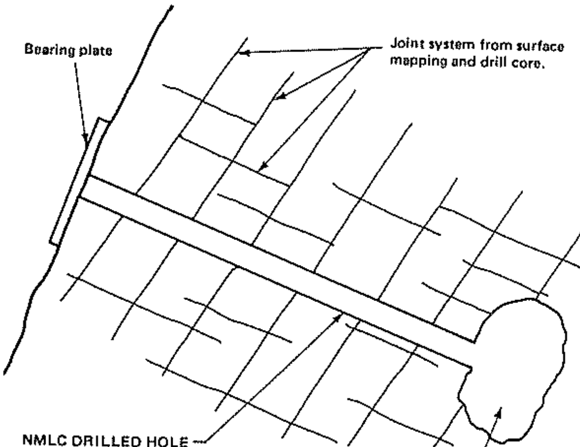


FIGURE 2

TYPICAL STRUCTURAL MODEL DETERMINED TO MEASURE IN SITU DEFORMATION MODULUS

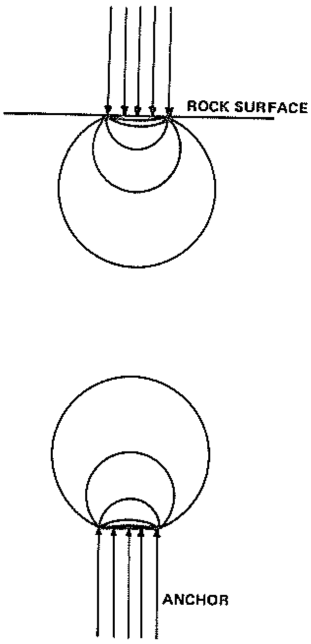


FIGURE 4
CONTINUOUS STRESS DISTRIBUTION

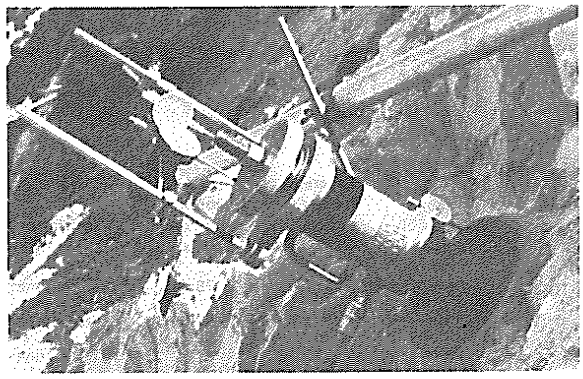


FIGURE 3

The load distribution between the face plate and anchor is largely conjectural. However because the stress distribution for a continuous material is as shown in Figure 4 and the stress distribution for a discontinuous material is as shown in Figure 5, the stress distribution between the plate and the anchor is assumed to be uniform. Assuming the stress distribution as given in Figure 5 depicting the results of the geological and computer model and the in situ deformational response Figure 6 the mass deformational response is evaluated. This gives the deformational modulus which is the constitutive parameter used to relate load and deformation, Table 4.

Table 4 also gives the modulus of the same material measured from the same rock bolt after loading the material with a nearby rock bolt. The nearby rock bolt is located within a radius of 1/2 the length of the initial rock bolt, and is also equal to the length of the initial rock bolt.

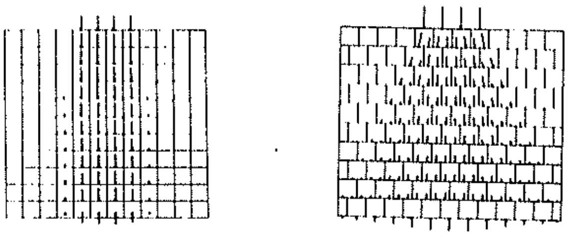
Moduli determined from the deformation of the face plate and anchor are also given in Table 4. These values of moduli show the effects of destressing the surface or skin of the slopes and the increase of moduli due to the confining or stressing effects of the inner rock mass. When determining the surface or skin moduli the elastic theory formula

$$E_v = \frac{P(1-\nu^2)}{2\rho a}$$

was used, where E_v is the deformational modulus P is the applied force applied to a rigid punch, ν is Poisson's ratio, ρ is the measured displacement of the rigid punch of radius a.

Bore Hole No.	Hole Diameter mm	Length m	Face Plate Modulus GPa	Absolute Modulus GPa	Reinforced Modulus GPa
1	75	6.96	0.25	6.26	
2	45	1.9	0.38	3.68	
3	45	1.58	0.15	1.62	
4	45	2.43	0.98	1.82	
5	75	6.96	0.16	5.13	
6	45	1.8	0.2	3.01	
7	45	1.88	0.67	3.15	11.3
8	75	6.34	0.16	7.43	
9	45	1.35	0.08	0.53	3.15
10	45	2.14	0.07	0.66	
11	75	8.0	0.45	12.7	
12	45	1.65	0.35	2.1	5.47

TABLE 4
In Situ Measured Moduli



LOAD DISTRIBUTION
FIGURE 5

3.5 Pseudo Elastic Analysis

Table 4 shows that there is difference between the surface moduli and anchor moduli due to the effects of joint constraints and confining stresses. The rock mass deformational modulus increases with depth from the surface of the excavation. In order to account for this the absolute deformations measured between the face plate and anchor are used to evaluate the rock mass deformational moduli. This value to the measured depth namely 6 m is used for the skin value of the slope stability analysis while the deformation of the anchor is used to determine the modulus of the inner rock mass. From these moduli the analysis of the slopes were performed using a pseudo elastic finite element program. The result of this was that tensile or much reduced compressive stress zones which were generated in the upper regions of the slope were eliminated after the remedial rock bolts were applied to the toe regions of the slopes.

By increasing the magnitude of skin moduli the tensile stress zones were eliminated in the upper destressed regions of the slope. The two main modes of decreasing the deformational response of a rock mass are by grouting and/or prestressing with anchors or rock bolts. In this particular instance grouting would not be effective in that the joints were infilled with gouge which would prevent the penetration of grout and much of the excavation had already been performed. Of consequence rock bolting was used to improve the deformational characteristics of the slopes. The situation considered here is that the rock mass without the rock bolts generally expands as the excavation is created and the loads redistribute. By using rock bolts, slip and rotation

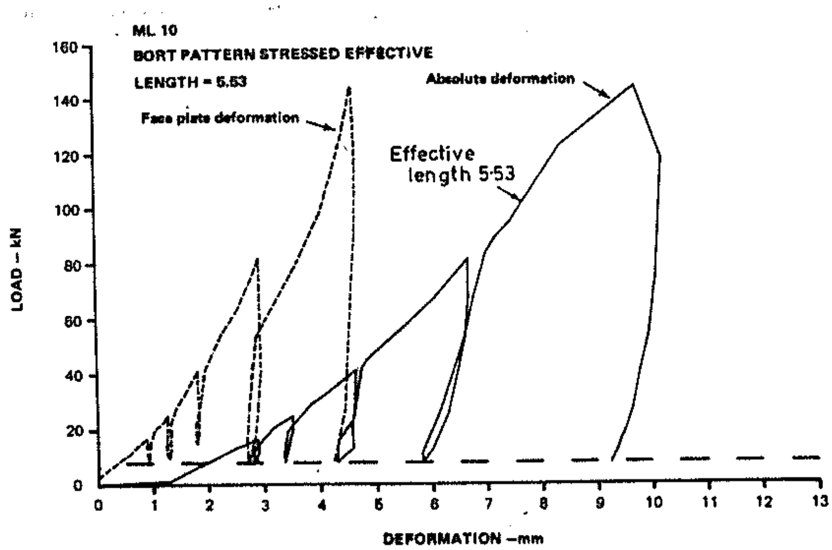


FIGURE 6
MEASUREMENT OF BEARING PLATE AND ANCHOR DEFORMATIONS

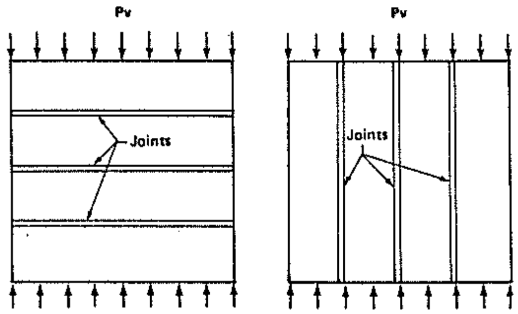


FIGURE 7

JOINT SETS ORIENTATED PERPENDICULAR AND PARALLEL TO IMPOSED LOADS

of the joint and fault sets is inhibited and as these mechanism of slip and rotation are the main processes of loosening and causing the consequent loss of rock mass strength the value of the rock bolt is evident. Besides this, however, the rock bolts also increase the normal loads across the joint system. This increases the stiffness of the joint which in turn increases the deformational modulus of the rock mass. With these aspects in mind the measured in situ moduli were used in a pseudo elastic plastic analysis. Where any zones of tensile or small compressive stresses occurred, the rock mass was pre-loaded with patterns of rock bolts so placed that zones of doubtful stress conditions were eliminated.

With the above approach the soft toe areas were pre and post stressed and the upper regions of the slope consequently stabilised. A very difficult and precarious condition is stabilised by measuring in the field the mass rock moduli before and after loading with rock bolts.

By combining the intact modulus of the parent rock and the modulus of the joint the upper and lower bound moduli are determined. A simplistic picture of the effect on these bound moduli caused by the orientation of the joint system relative to the direction of imposed load is given in Figure7&8. When the joints are perpendicular to the direction of the load a lower bound value of modulus is obtained when

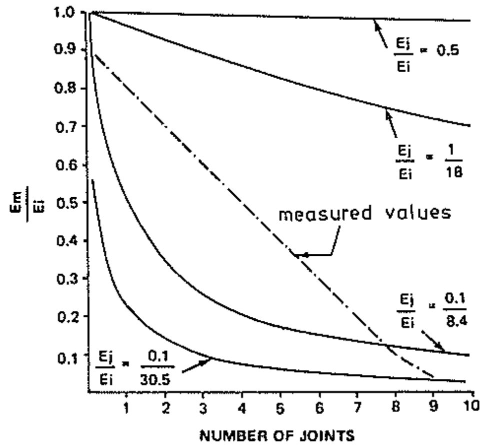


FIGURE 8

parallel an upper bound value results. This introduces an anisotropy into the material which is characterised by the orientation of the joints and their physical properties, Table 3. It is evident that here the intact rock controls the upper bound modulus and the joints control the lower bound modulus. Therefore if the intact rock is soft the upper bound modulus would be low in magnitude and have a high classification value depicting bad rock mass even if the mass had no joints. On the other hand if the intact rock were hard but had numerous soft joints the rock mass would still have a high classification rating and be considered bad. It is therefore evident that joint properties and the associated orientation relative to the imposed loads has an important effect on the rock mass classification and the consequent deformational response.

The measured in situ rock bolt mass moduli varied from 0.53 GPa to 30 GPa. This was collated with the rock mass classification defined below.

Class	3	4	5
Modulus	≥ 10 GPa	< 10 GPa but ≥ 1 GPa	< 1 GPa

The moduli measured before and after rock bolting showed that the bolts in effect increased the classification rating of the rock mass. A characteristic not investigated here but nevertheless important is that the rock bolt inhibits loosening of the rock mass thus preventing load redistributions and dilation. This increases the strength characteristics of the rock mass.

Table 5 shows the percentage reduction of the deformational response of the intact rock modulus caused by the joint-fault systems, John (13). The intact moduli of the Greenstone and Jasper were taken as 60 and 18 GPa respectively and the measured mass moduli are those related to the classified rock mass.

By comparing Tables 3 and 4 it is seen that the measured in situ mass moduli lie within the upper and lower bounds of the evaluated moduli. The effects of the rock bolts on the mass deformational response is to improve the rock mass classification by at least one class and in some cases two. That is if the rock mass was class 5 before installing the rock bolts it is improved to at least a class 4 and in some cases a class 3.

4 CONCLUSIONS

When the mechanisms of slip and rotation in a jointed rock mass are not part of the deformational process, the rock mass is considered a composite elastic material. From this the pseudo elastic theory is used to analyse the stability of the rock slopes. In order to perform the analysis the constitutive relation between load and deformation is required. In situ field jacking tests using hollow anchored rock bolts are found suitable for determining the deformational modulus of the rock mass. A geological model of the rock material making up the jacking test zone, is constructed and this coupled with the results of the in situ deformational response give the required deformational muduli required for the pseudo elastic analysis.

Laboratory tests are performed on the NMLC core retrieved from the holes drilled for the jacking tests. Standard laboratory testing give the moduli of the intact rock material and stiffnesses of the joint-fault sets. The results of the testing are incorporated in the geological structural model to determine the in situ pseudo elastic moduli or used in composite elastic theory to give the upper and lower bound moduli of the rock mass.

Using an appropriate classification system the rock zones are classified; which in this instance are class 3, 4, and 5 being fair, bad and very bad rock masses respectively. The deformational moduli associated with these rock zones are carefully correlated. This means that when encountering a rock of a specific class the appropriate rock bolt patterns and remedial measures are applied.

The possibility of connecting the classification system with design by measuring the concomitant deformational moduli has wider aspects than those reported here. The quantification of classification with design has great potential in documentation of both specifications and contracts associated with earth material.

TABLE 5

	Rock Mass Class (Classification) 3	4	5
Greenstone % Modulus reduction	<16.7	>16.7 but < 1.7	> 1.7
Jasper % Modulus reduction	< 55.6	>55.6 but < 5.6	> 5.6

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