Rock Modulus from In-situ Pressuremeter and Laboratory Tests

Le module du rocher au pressiomètre in situ et par essais au laboratoire

L.F. Cao, S.M. Peaker & A. Sirati
SPL Consultants Limited, Ontario, Canada

ABSTRACT : This paper describes in-situ rock pressuremeter (rock dilatometer) tests conducted in shale rock in Toronto, Canada. The deformation moduli of rock mass obtained from in-situ pressuremeter tests are compared with those measured from laboratory unconfined and triaxial compression tests. It is found that the deformation moduli of shaly rock mass measured from in-situ tests are generally higher than those measured on the rock samples in laboratory. The lower modulus obtained from laboratory testing could be due to swelling and weathering of rock samples as well as the release of the high in-situ horizontal stress in the bedrock after sampling. The use of modulus obtained from unconfined compression testing on intact rock sample will lead to a conservative approach in geotechnical design for the shaly rock.

RÉSUMÉ : Cet article décrit les essais au pressiomètre (le dilatomètre de roche) in-situ menées dans le schiste à Toronto, au Canada. Les essais de compression simple ou triaxiale en laboratoire et les modules de déformation de roche massive provenant d'essais pressiométriques in situ sont comparés. Il est constaté que les modules de déformation des roches schisteuses massives mesurés à partir d'essais in-situ sont généralement supérieurs à ceux mesurés sur les échantillons de roche en laboratoire. La valeur inférieure du module obtenue à partir d'essais en laboratoire pourrait être due à un gonflement et à l'altération après le prélèvement d'échantillons de roches, mais aussi à la libération de la contrainte horizontale élevée in situ dans le substratum rocheux. L'utilisation du module de compression simple obtenu sur l'échantillon de roche intacte correspondra donc à une étude prudente dans la conception géotechnique pour le rocher argilo-schisteux.

KEYWORDS : pressuremeter, dilatometer, unconfined compression, triaxial compression, shaly rock, modulus.

MOTS CLES : pressiomètre, dilatomètre, compression simple, compression triaxiale, roche argilo-schisteuse.

1 INTRODUCTION

Pressuremeter testing has been advanced considerably since its introduction by Menard in 1956 and is widely performed in soils to determine soil parameters. There are limited cases for the pressuremeter testing conducted in rocks.

This paper describes in-situ rock pressuremeter (also called rock dilatometer) testing in relation with a rock tunnel project in bedrocks in Toronto, Canada. In order to determine the deformation parameter of rock mass for design of the tunnel work, pressuremeter tests were conducted in four NQ-size coreholes at depths ranging from 19.3 m to 59.1 m below the existing ground surface. Unconfined compression and triaxial compression tests on rock core samples were also carried out in laboratory to determine the elastic moduli of the intact rock samples. The moduli obtained from in-situ pressuremeter tests are compared with those obtained from laboratory tests and their differences are discussed. Recommendations for the use of modulus in the design are provided.

2 SITE CONDITIONS

The site is situated at Lakeshore Boulevard East, approximately 200 m west of Coxwell Avenue, in the City of Toronto, Canada. The lands on both sides of Lakeshore Boulevard at this site include facilities associated with the Ashbridges Bay Water Treatment Plant, as well as parkland.

The existing site grade lies between Elev. 76.8 m to 79.4m, which is approximately 1.8 m to 4.4m above Lake Ontario level. Lake Ontario lies some 50m to the south of this site.

Field investigation consisted of borehole drilling and rock coring at periodic intervals along the proposed tunnel alignment, and revealed that the site stratigraphy was made up of 2.3 m to 6.2 m thick, very soft to very stiff clayey silt fill and/or loose to compact sandy silt to silty sand fill, over 0 to 3.9 m thick, very loose organic silt to silty sand, very soft to stiff organic clayey silt, and/or peat, underlain by laminated clayey silt to silty clay with some sand lenses/layers as well as sand and gravel, which in turn overlie clayey silt till or till/shale complex. Bedrock of the Georgian Bay Formation was encountered at depths ranging from 13.0 m to 17.8 m (Elev. 65.1 m to 61.6 m) below existing ground surface. The groundwater levels measured in the monitoring wells installed with the overburden soils ranged from 0.7 m to 3.0 m (Elev. 76.4 m to 75.4 m) below the existing ground surface. The groundwater levels measured in the monitoring wells installed within the bedrock ranged from greater than 0.6 m above ground surface (higher than Elev. 78.3 m) to 3.8 m (Elev. 73.6 m) below the existing ground surface. Groundwater level installed at shallow depth within the bedrock in one monitoring well was artesian with respect to existing grade.

The Georgian Bay Formation consists of typically highly weathered to fresh, grey, fine to very fine grained fissile, weak to medium strong shale with widely spaced jointing and sub-horizontal bedding planes, interbedded with slightly weathered to fresh grey, fine grained strong to extremely strong calcareous siltstone and limestone layers (hard layers). Figure 1 shows the photographs of rock cores taken from one corehole (Location 4) at depths from 14.2 m to 59.5m. The total core recovery (TCR) below the upper weathered zone (1 to 2 m below the bedrock

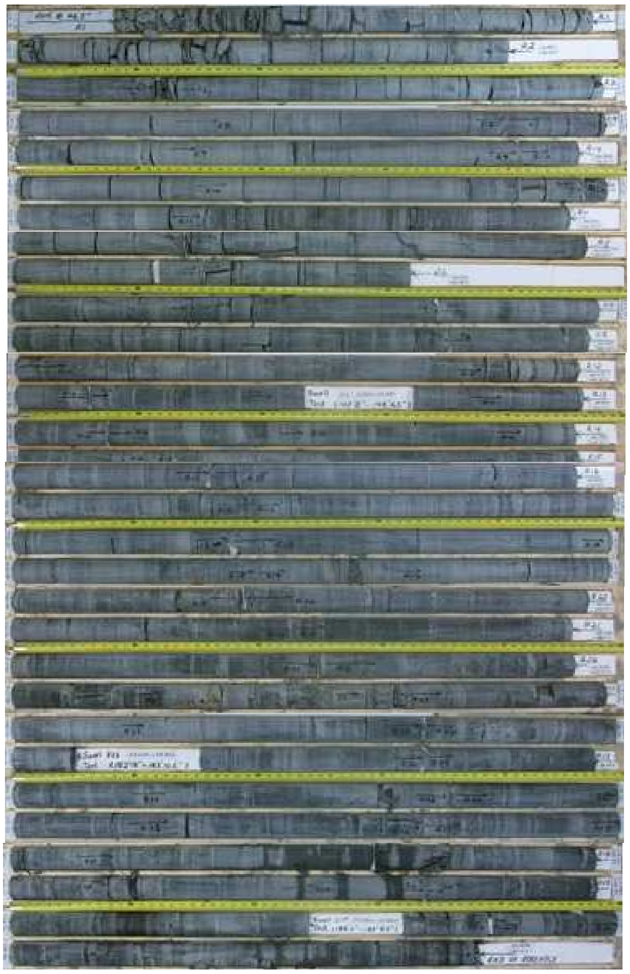

Figure 1. Photographs of rock cores taken at depths of 14.2 m to 59.5 m below the existing ground surface at Location 4

surface) was 100%. Generally, poorer core recovery was experienced only near the bedrock surface, where the formation is more weathered. The rock quality designation (RQD) below the upper weathered zone ranged between 70% and 100%. Low RQD was recorded at the upper weathered zone. Based on the visual examination of the rock cores, an attempt was made to identify and record the thickness and percentages of the relatively harder siltstone and limestone layers. The percentage of the "hard layers" per core run ranged between 4% and 38%, averaging approximately 17%. The thickness of these layers varied but was generally less than 100 mm. Figure 2 shows the profiles of TCR, RQD and hard layer percentage at the four locations where the rock pressuremeter tests were conducted. The shaly bedrock formations are subjected to high in-situ horizontal stresses and the measured horizontal stress ranged between 1.7 MPa and greater than 6.9 MPa (Lo et al. 1979).

3 TESTING PROGRAM AND RESULTS

Rock pressuremeter tests (RPMT) conducted at four locations at the site using PROBEX borehole dilatometer. The four boreholes were advanced through the overburden using HW casing down to about 16 m to 19 m below ground surface. The tip of the HW casing was set firmly into the top of the rock mass to prevent the migration of soil particles into the corehole. The coreholes were advanced down to about 59 m below ground surface using triple tube wireline rock coring equipment. The nominal size of the rock coring equipment used for RPMT was NQ.

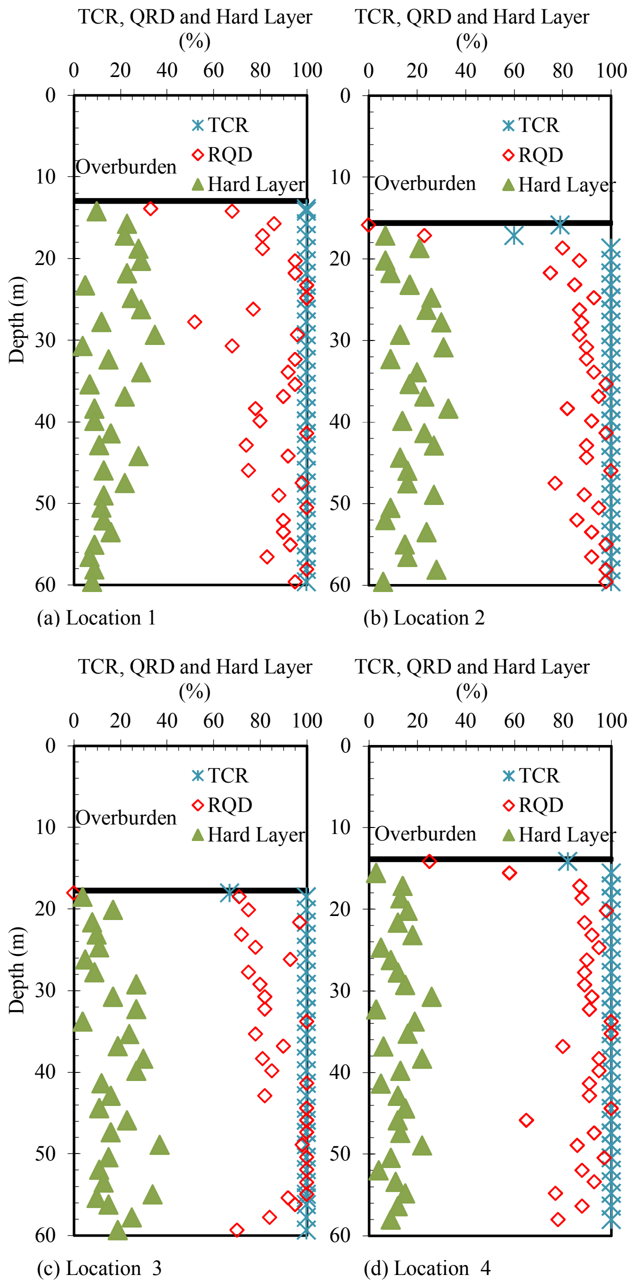

Figure 2. Profiles of TCR, RQD and hard layer percentage at four location

Rock cores were continuously sampled at approximately 1.5 m intervals. Rock coring was advanced using fresh/clean water. At the end of coring, the holes were flushed with fresh water in order to eliminate rock cuttings (also known as "rock flour") from the test holes. On completion of coring, rock core logs were examined to identify rock weakness areas or cavities present on the rock mass in an effort to prevent potential membrane rupture in such cavities. RPMT testing was conducted to determine deformation moduli of the rock mass at the tested depths/locations in preparation for design of tunnel work at the site. In addition, coring of the rock with HQ-2 size double tube wireline equipment was conducted near the pressuremeter test holes to take rock core samples for laboratory testing.

In-situ RPMT testing procedures were performed as suggested by the equipment's manufacturer, Roctest Ltd.

Testing procedures were also in general conformance with the testing procedure recommended by US Bureau of Reclamation. The dilatometer probe was positioned at the target depths using NQ rods being hoisted with the main winch in the drill rig. Tests were completed at approximately 3 m intervals. RPMT test equipment, including probe, tubing, control unit, and data logger were checked and calibrated on site prior to perform the tests, as recommended by the manufacturer. The testing procedure consists of membrane dilation on a monotonic pressurization pattern. No unloading-reloading cycle was performed. For the tested shale formation, the pressurization loading consisted of approximately 3,000 kPa steps, suitable for rather soft to medium hard rock types, up to a maximum pressure of about 24,000 kPa. Volume and pressure readings were taken with a 60-second delay after start of each pressure step. It should be noted that RPMT testing considers two test variables, volume and pressure. As such, estimated rock deformation moduli represent average properties of the rock mass surrounding the probe, which has nominal length and diameter or 457 mm and 73.7 mm, respectively.

Based on in-situ test data and in calibration parameters, values of the rock modulus, E_r are calculated using the following expression:

$$E_r = 2(1+v_r)(v_o + v_m)\frac{(\Delta p_b - \Delta p_i)}{\Delta v - c(\Delta p_b - \Delta p_i)} \tag{1}$$

where v_r is Poisson's ratio of the rock; v_o is the nominal initial or "at rest" volume of the deflated probe (this volume is approximately equal to 1950 cc); v_m is the mean additional volume (up to the selected pressure range midpoint) injected into the probe from the "at rest" condition; Δv is the additional volume injected into the dilatable membrane, correspond to the applied pressure increment, Δp_b; Δp_i is the change of the pressure of inertia of the dilatable membrane corresponding to the applied pressure increment Δp_b; and c is the volume correction factor of the dilatometer.

For each test, the modulus of deformation is measured as the steepest slope of the pressure/volume curve. For this purpose, two data points are manually selected, then the proprietary software, "Probex Companion" by Roctest, calculates and reports the deformation modulus of the tested rock mass. Values of Poisson's Ratio for the shaly rock measured from the unconfined compression tests ranged from 0.10 and 0.29. A value of 0.2 was used in the E_r calculation for RPMT.

RPMT on Locations 2, 3 and 4 was completed within a week period of time after completion of coring at each coring location. In contrast, the corehole at Location 1 was tested after two weeks of coring. Figure 3 shows a RPMT conducted at a depth of 59 m below the existing ground surface at Location 1. Before the applied pressure reached to 2.3 MPa, the probe was not fully attached with the corehole. When the applied pressure was greater than 3.7 MPa, a linear relationship between load increment and volume increment was observed. The deformation modulus was determined from the linear slope. It is noted that the dilatometer probe cannot induce failure in the tested rock mass, therefore, no failure parameters can be inferred from this test. The deformation modulus of rock mass obtained from RPMT ranged from 0.9 GPa to 12.8 GPa with an average value of 7.4 GPa. As shown in Figure 4, the RPMT deformation modulus generally increases with depth with the exception at a depth of 34.6 m below grade at Location 1 and at a depth of 37.8 m below grade at Location 3, where low values of modulus were obtained. At these depths, sub-horizontal joints/beddings were observed.

Rock core samples were selected for unconfined

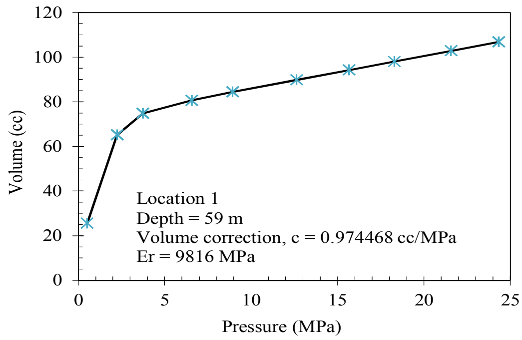

Figure 3. A typical result of RPMT

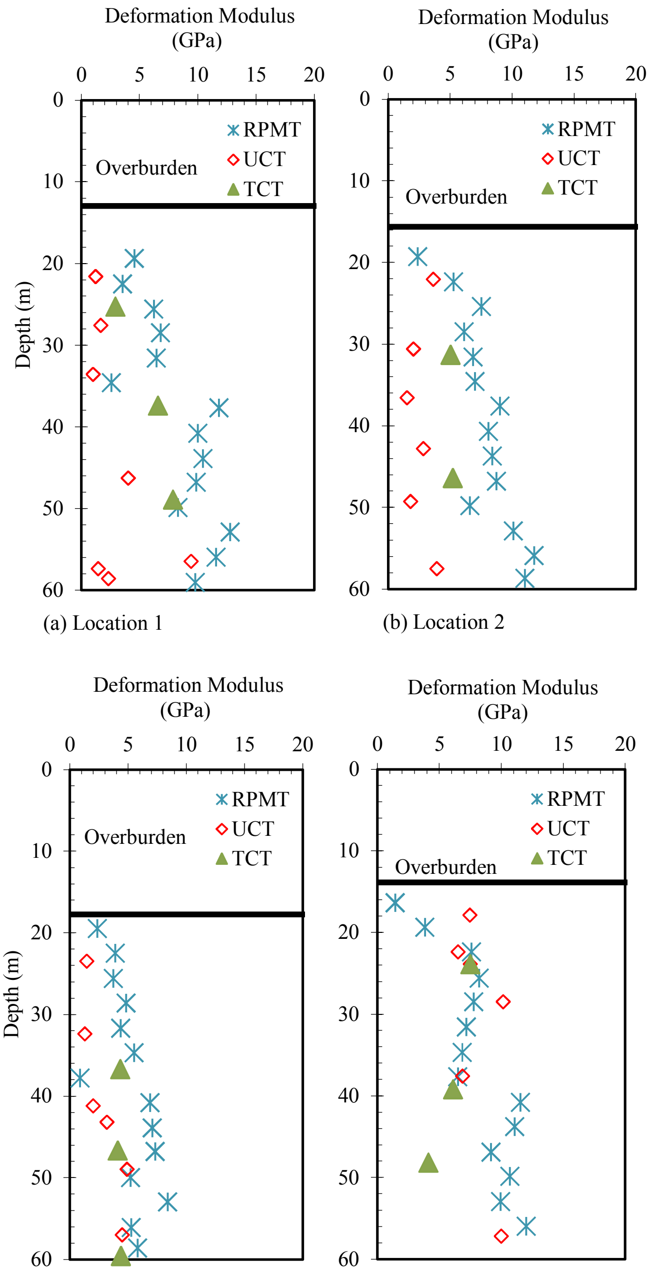

Figure 4. Deformation modulus obtained from RPMT, UCT and TCT

compression tests (UCT) and triaxial compression tests (TCT) to determine the elastic modulus of rock core sample. The sample ends were diamond-sawed to prepare a cylindrical sample having nearly parallel end faces and diamond-lathed to prepare samples faces parallel to within ±0.025 mm for the UCT and TCT. The confined pressure σ_3 used in the TCT ranged from 3 MPa to 6 MPa, corresponding to the in-situ horizontal stress. The UCT and TCT were performed under axial stain control at rates approximating 10^{-5} per second, and simultaneous recording of axial force, axial deformation and circumferential deformation was conducted, from which determination of the sample compressive strength and elastic modulus.

Table 1 summarizes the results of the UCT. The values of elastic modulus E_r obtained from UCT on ranged from 1.0 GPa to 20.8 GPa with an average value of 4.8 MPa. The unconfined compressive strength (UCS) ranged from 8.0 MPa to 72.0 MPa. The bulk density γ ranged from 2.52 to 2.69 g/cm^3. It is noted that the higher values are obtained from the limestone samples and lower values from the samples of shale bedded with limestone or siltstone.

Table 1. Results of unconfined compression tests.

Location	*Depth (m)*	*UCS (MPa)*	E_r *(GPa)*	v_r	γ *(g/cm^3)*	*Rock Type*
1	21.6	16.3	1.231	0.17	2.68	Shale
1	27.6	8.0	1.691	0.10	2.52	Shale
1	33.6	8.8	1.015	-	2.58	Shale
1	46.3	34.4	4.034	0.10	2.62	Shale
1	56.5	60.6	9.453	0.10	2.63	Limestone
1	57.4	10.9	1.457	0.22	2.63	Shale
1	58.6	19.0	2.334	0.12	2.60	Shale
2	22.1	20.8	3.657	-	2.62	Shale
2	30.6	13.4	2.064	-	2.59	Shale
2	36.6	11.4	1.552	-	2.61	Shale
2	42.8	23.0	2.866	-	2.62	Shale
2	49.3	13.4	1.827	-	2.60	Shale
2	57.5	18.0	3.942	0.13	2.64	Shale
3	23.5	10.8	1.467	-	2.51	Shale
3	32.4	12.8	1.308	-	2.52	Shale
3	41.2	20.1	2.001	-	2.58	Shale
3	43.2	10.7	3.203	0.12	2.58	Shale
3	49.0	24.8	4.932	0.12	2.62	Shale
3	57.0	15.4	4.509	-	2.62	Shale
4	17.9	27.7	7.473	0.29	2.63	Shale(*)
4	22.4	23.6	6.521	0.10	2.62	Shale(*)
4	23.9	23.0	7.503	0.12	2.59	Shale(*)
4	28.5	42.8	10.154	0.10	2.62	limestone
4	37.6	32.0	6.895	0.10	2.69	Shale(*)
4	43.6	72.0	20.774	0.14	2.65	limestone

(*) Sample with high percentage of limestone or siltstone

Table 2. Results of triaxial compression tests.

Location	*Depth (m)*	σ_1 *(MPa)*	σ_3 *(MPa)*	E_r *(GPa)*	γ *(g/cm^3)*	*Rock Type*
1	25.3	37.4	4.0	2.943	2.60	Shale
1	37.4	50.7	5.0	6.592	2.55	Shale
1	48.9	64.4	6.0	7.895	2.62	Shale
2	31.3	42.7(*)	3.0	5.049	2.41	Shale
2	46.4	25.9	5.0	5.222	2.60	Shale
3	36.7	39.0(*)	4.0	4.349	2.59	Shale
3	46.7	8.6	5.0	4.117	2.59	Shale
3	59.6	79.6	6.0	4.395	2.61	Shale
4	23.9	40.5	3.0	7.503	2.59	Shale
4	39.2	53.1(*)	4.0	6.113	2.64	Shale
4	48.2	37.9(*)	5.0	4.112	2.66	Shale

(*) Sample failure occurred along pre-existing foliation surface

Table 2 summarize the results of the TCT. The maximum (failure) pressure σ_1 ranged from 8.6 MPa to 79.6 MPa. The values of elastic modulus measured from the TCT for the samples of shale bedded with limestone or siltstone ranged from 2.9 GPa to 7.9 GPa with an average value of 5.3 MPa.

The values of elastic modulus obtained from the UCT and TCT are shown in Figure 4 for comparison. It is noted that at Locations 1, 2 and 3, the RPMT modulus is the upper bound of the moduli measured from the three types of tests, whereas the UCT modulus is the lower bound. At Location 4, the UCT modulus of limestone/siltstone samples or shale with high percentage of limestone or siltstone is close to or slightly greater than the RPMT modulus. This founding is against the conventional opinion that the modulus of intact rock sample is higher than that of rock mass which contains joints and beddings. The lower modulus obtained from laboratory testing could be due to swelling and weathering of rock samples as well as the release of the high in-situ horizontal stress after sampling.

The modulus obtained from the TCT with a confined pressure similar to the in-situ horizontal stress is close to or slightly smaller than the RPMT modulus, but greater than the UCT modulus. This means that the modulus is a pressure-dependent parameter.

4 CONCLUSIONS

The deformation moduli of shaly rock mass measured from in-situ pressuremeter tests are generally higher than those measured on the rock samples in the laboratory. The use of modulus measured from the unconfined compression test on intact rock sample will lead to a conservative approach in geotechnical design for the shaly rock.

This finding is important with respect to tunnel design and signifies the importance of in-situ testing vs laboratory testing.

5 REFERENCES

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