Measurement of stiffness of rock from laboratory and field tests

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ABSTRACT
This paper compares the deformation modulus of rock measured from laboratory and field tests which were carried out as part of the site investigation works for a major project in Irbid, Jordan. Laboratory resonant column and torsional shear tests were performed at different confining pressures whereas ultrasonic velocity tests were conducted on unconfined rock specimens. In addition, empirical relationships were used for estimating the rock mass modulus employing the results of the uniaxial compression and point load strength tests. Field measurements comprised pressuremeter testing as well as seismic geophysical methods including down-hole and cross-hole techniques. The static and dynamic in-situ stiffness measurements were found to be reasonably in good agreement with the laboratory values from the dynamic tests as well as empirical methods for estimating rock mass stiffness from uniaxial compressive strength results.

INTRODUCTION

The modulus of deformation is undoubtedly the geomechanical parameter that best represents the mechanical behavior of rock mass. In particular, when it comes to underground excavations, this modulus becomes indispensable – whatever the type of design approach to be developed. Laboratory measurements have long been the reference standard for determining the mechanical properties of geomaterials. In addition, field tests to compliment the geotechnical investigation and laboratory testing has become an expedient and cost-effective way to determine the strength and stiffness parameters over an entire site.

The main purpose of this paper is to present and compare the stiffness obtained from static and dynamic tests determined in the laboratory and field from a comprehensive and integrated site investigation which was carried out by Arab Center for Engineering Studies (ACES) for a major project in Irbid, Jordan. The field tests comprised pressuremeter as well as seismic geophysical methods including down-hole and cross-hole techniques. On the other hand, the laboratory dynamic tests involved ultrasonic velocity, resonant column and torsional shear testing. In addition, uniaxial compression tests were carried out in which the stiffness was estimated from the compressive strength results employing well-established empirical relationships.

STIFFNESS

The deformation constants of a material are the most important parameters in any design and their determination involves the use of measuring techniques both for load and deformation. The amount of deformation that most of the rocks undergo is extremely small and its measurement requires special techniques. Deformation is defined as changes in shape (expansion, contraction, or other forms of distortion). It occurs usually in response to an applied load or stress, but it also may result from a change in temperature (thermal expansion or contraction) or water content (swelling or shrinkage). Deformability describes the ease with which rock can he deformed, and its inverse, stiffness, the resistance to deformation. Deformability, like strength, depends mostly on the porosity and the degree of jointing of the rock under test. Pores and joints are the weakest and most deformable elements in the rock. Other factors influencing rock deformability are drying and vibration effects from blasting.

The rock mass deformations are calculated by means of modulus of elasticity values as obtained from laboratory tests on rock core specimens. In general, laboratory rock specimen test results do not represent the in-situ properties of the overall rock mass. This limitation has led to the development of several static and dynamic field methods. To define the quality of the rock masses based on rock mechanics parameters, both field and laboratory test results should be used in the design. Depending upon the extent, amount and distribution of the joints and other defects in a rock mass, the modulus of deformation of rock may be quite different from its modulus of elasticity. Recall that the modulus of deformation is based on the total measured deformation (elastic plus inelastic). There are static and dynamic methods for determining the deformability of in-situ rock. In static ‘destructive’ tests, relatively large static loads are applied on the rock surface. In dynamic ‘non-destructive’ tests, the velocity of propagation of elastic waves (seismic or acoustic) is measured.

Probably the most commonly assumed behavior in practical geomechanics is that of isotropic linear elasticity (Clayton 2011). Characterization of an isotropic elastic solid requires the determination of only two material parameters (from four possible measurements, i.e. Young’s modulus E and Poisson’s ratio υ, or shear modulus G and bulk modulus K) for calculations of strain and deformation, and therefore an assumption of isotropic elasticity has the merit of simplicity. In the isotropic case, the relationship between the Young’s modulus, Poisson’s ratio and shear modulus can be obtained from

$$G = \frac{E}{2(1+\nu)} \tag{1}$$

Any measurement of stiffness, whether made in the field or in the laboratory, needs to be critically reviewed in the context of those factors that will control the stiffness of the ground around the structure. One of the main factors controlling the stiffness is the strain level, where stiffness parameters may be considered constant (i.e., linear) at very small strains (< 0.001%), but can be expected to reduce from the maximum value as strains increase above this level (Figure 1). Note that the strain levels around well-designed geotechnical structures such as retaining wall, foundations and tunnels are generally small (Burland 1989).

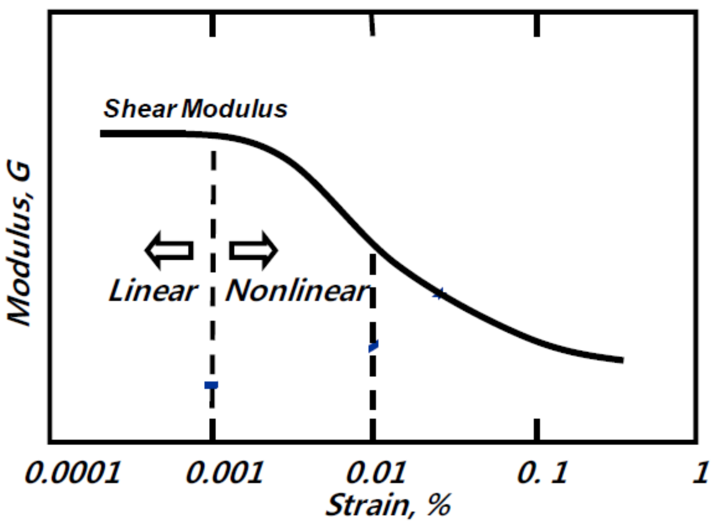

Figure 1: Nonlinear deformation characteristics

GEOTECHNICAL INVESTIGATION AND TESTING

The ground investigation was undertaken by ACES and consisted of drilling 38 boreholes, field testing and laboratory testing (including specialist testing) on selected samples. The boreholes were drilled to 30-150m deep using rotary drilling method. The scope of field testing is summarized as follows:

- Standard penetration testing (SPT)
- Permeability (constant head and packer) testing
- Plate load testing
- Field density testing
- Pressuremeter testing
- Geophysical (electrical resistivity tomography, seismic refraction, down-hole and cross-hole) testing.

Disturbed, undisturbed and split-spoon samples were obtained from the boreholes for laboratory testing. Continuous coring was carried out in rock whereas SPT was conducted at 1.5m intervals in soils. The undisturbed samples were obtained using double tube (T6-101 series – core diameter ~ 79mm) and wireline triple tube (HQ3 series – core diameter ~ 61mm) core barrels. Air flush was used at depths less than 60m whereas water flush was employed for greater depths. The laboratory testing included the following standard and specialist tests:

- Standard classification and index testing
- Strength (uniaxial, point load, unconsolidated undrained [UU] triaxial, direct shear) testing
- Compaction (Proctor, CBR) testing
- Chemical testing
- Dynamic (resonant column, torsional shear, ultrasonic velocity) testing.

GEOLOGY

The project site is covered by superficial (recent) deposits underlain by Muwaqqar chalk marl formation (B3). The superficial deposits comprise topsoil, calcrete, and pleistocene (plateau) gravel. The topsoil material is composed of reddish brown silty clay with few gravels of chert. Calcrete (calcareous, caliche) deposits are composed of hard re-crystallized crust of chalk (calcium carbonates) with some scattered chert fragments and some silty clay intrusions. Pleistocene gravel deposits are composed of weathered, medium dense to very dense, mixture of sandy gravel, gravel, cobbles and boulders of limestone, silicified limestone and chert with poorly to slightly cemented white calcareous (chalky to clayey marl) deposits and reddish brown silty clay intercalations. The superficial deposits thickness ranged from 10-15m at the site.

The Muwaqqar chalk marl formation (B3) is subdivided into the following divisions (from top to bottom): limestone materials; chalky to clayey marl materials; and bituminous limestone and marlstone materials. The limestone layer (upper Muwaqqar) consists of white to beige, silty texture, horizontally fractured, moderately weak to moderately strong limestone; intercalated with some thin bands of rosy to brown, very weak to weak, chalky to clayey marl and marlstone, and few thin bands of brown silty clay. This layer has an average thickness of 25-38m at this site. The chalky to clayey marl layer (lower Muwaqqar) consists of yellow, rosy and light brown, silty to clayey texture, massive, very weak, poorly cemented chalky to clayey marl, intercalated with weak, slightly cemented chalky to clayey marlstone and some moderately weak, chalky to marly limestone concretions (lenses). The average thickness of this layer is 35-50m. The bituminous limestone and marlstone layer (lower Muwaqqar) consists of light to dark gray, massive to thick bedded, moderately weak to moderately strong bituminous limestone, intercalated with dark gray to black, very weak to weak, thinly foliated bituminous marl and marlstone. The minimum thickness of this layer at the site is 80m. It should be noted that no groundwater was encountered in any of the boreholes.

FIELD TESTING PROGRAM

Pressuremeter Testing

Borehole expansion (high pressure dilatometer) tests were conducted in rock at depths ranging from 20-88m below ground level in accordance with ASTM D4719. The tests were performed using Elastmeter HQ Sound (Model-4180) which has 0-20 MPa pressure range. The test probe has expandable length of 700mm and deflated diameter of 74mm whereas the test was performed in a borehole section with nominal diameter of 76mm (prepared using smaller core barrel - T2-76 seires). The test pressure (applied in equal increments) was held for a minimum period of 60 seconds at each increment to allow for the deformation to stabilize. Loading was done using the high pressure hand pump and the displacement for the pressure applied was recorded. Two unload/reload cycles were performed before reaching the maximum pressure (~ 7 MPa) after which the membrane was deflated (end of test). The internal displacement calipers and the rubber membrane were calibrated as per the manufacturer's instructions and relevant standards. The instrument was calibrated before each use for both pressure and volume losses.

The typical parameters generally obtained from conventional pressuremeter tests include modulus of deformation, coefficient of lateral earth pressure, yield and limit pressures among others. Figure 2 shows a graphical presentation of the elastic shear and Young's modulus results calculated from unload/reload cycles performed during the test.

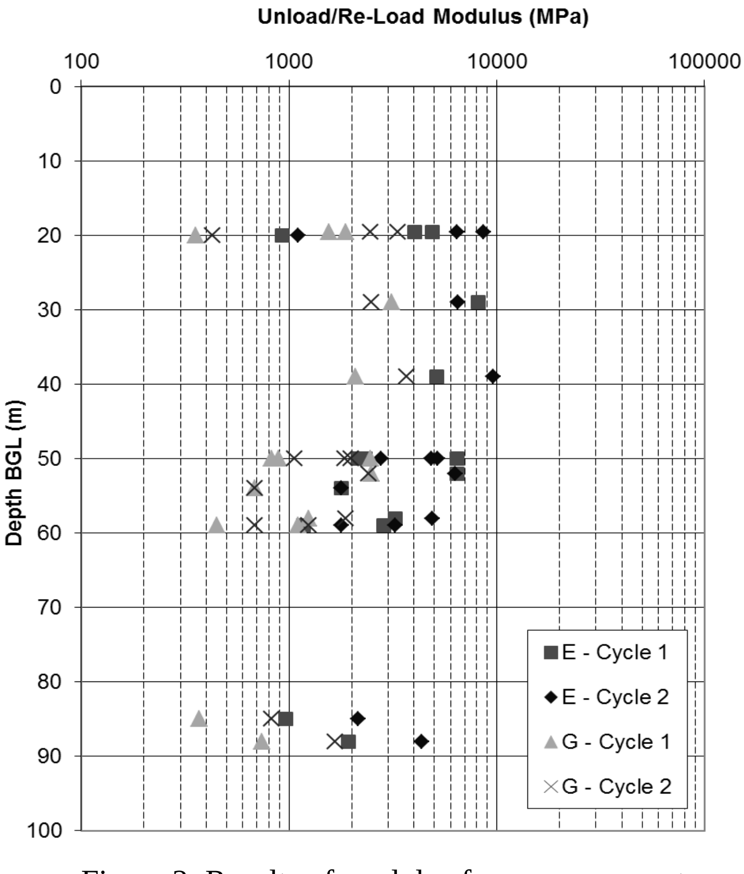

Figure 2: Results of modulus from pressuremeter

Down-hole Geophysics

Down-hole suspension P-S velocity logging was carried out to acquire primary compressional (P) and secondary shear (S) wave velocities as function of depth which in turn can be used to derive dynamic elastic soil properties such as Young's modulus, shear modulus and Poisson's ratio. The logging was performed every 1m interval and reaching a maximum of 91m depth. The single borehole probe encompasses the seismic source (generating P & S waves) and two receivers (three-component geophones) with spacing of 1m between the two receivers. This allows the travel time to be determined from waveforms detected at both sensors from the same hammer blow. The borehole was cased with PVC threaded pipes with one-way valve at the bottom end, and the annular space outside the PVC pipes was grouted with cement-bentonite grout with bottom-up grouting technique. The test was carried out in a borehole filled with water.

The shear modulus of the rock can be determined from the shear wave velocity using the following relationship

$$G = \rho V_s^{\ 2} \tag{2}$$

where ρ is the bulk density and V_s is the shear wave velocity. It should be noted that the density was assumed based on the laboratory results. From this, the elastic Young's modulus was determined using equation (1) and employing the Poisson's ratio values estimated from P & S wave velocities using the following relationship

$$\nu = \frac{(V_p / V_s)^2}{2\{(V_p / V_s)^2 - 1\}} \tag{3}$$

where V_p is the compressional wave velocity.

The results of the P-S logging measured using OYO Suspension System are provided in Figure 3. The figure shows graphs of P & S velocities, dynamic modulus (shear and Young), and Poisson's ratio versus depth.

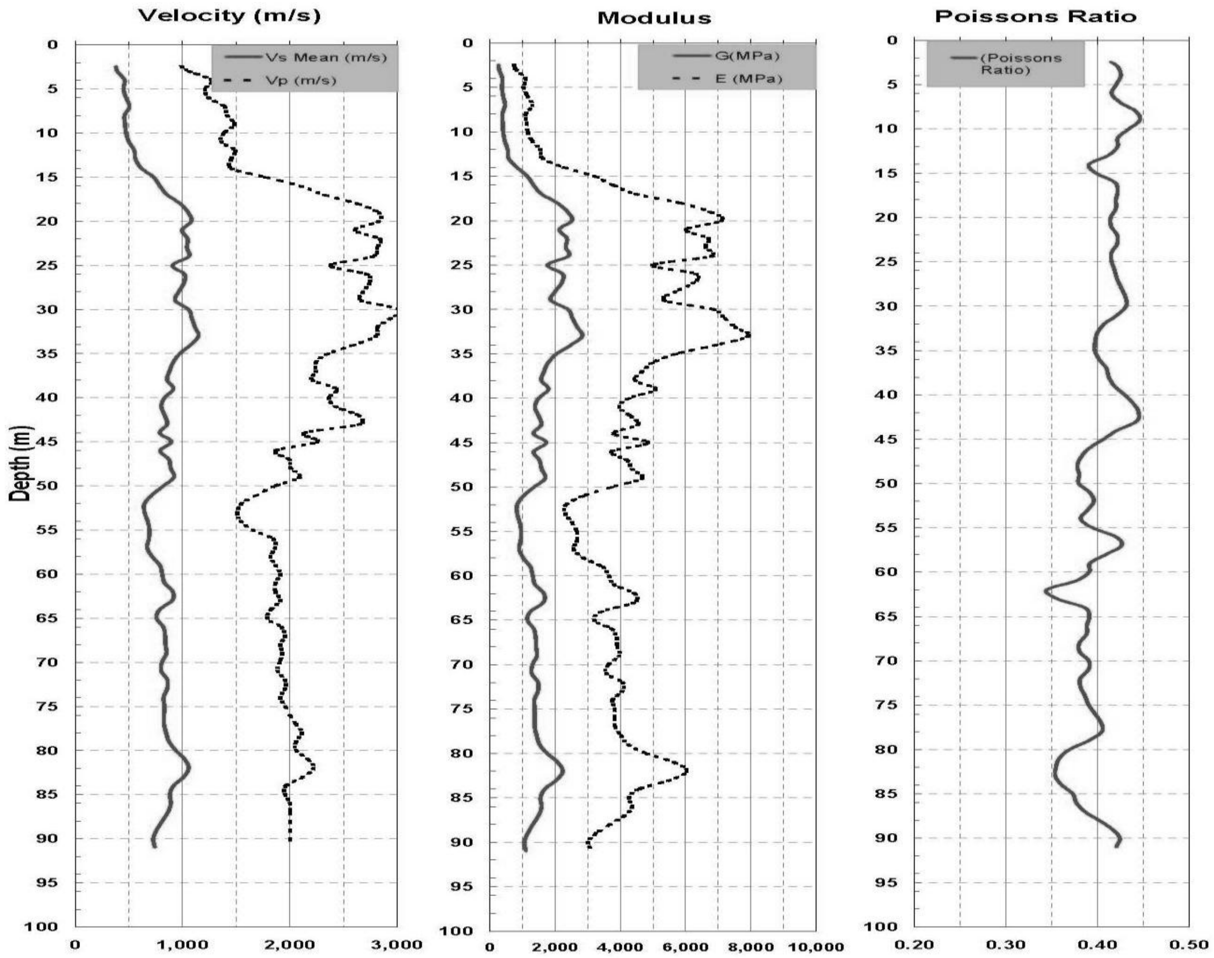

Figure 3: Results of down-hole test

Cross-hole Geophysics

The standard cross-hole seismic test was carried out to acquire shear and compressional wave velocities as a function of depth (1.0m logging interval) which in turn can be used to obtain dynamic soil properties (E, G, and v). The test was carried out in boreholes drilled to depth of 100m in accordance to ASTM D 4428. Three co-linear boreholes with 4.5m spacing were used. A borehole verticality survey was carried out in order to calculate the actual distance between boreholes at each test depth, since some deviation from vertical will have occurred during drilling and casing installation. A source was inserted in one of the boreholes to create seismic P & S waves whereas receivers (three-component geophones) were placed in the remaining two boreholes to measure the arrival of the seismic wave. The use of two sets of receivers avoids the issue of trigger accuracy, but increase the cost of this type of test. The inter-borehole distance is divided by the travel time at each depth to calculate the wave velocity. Like the PS suspension borehole, the cross-hole boreholes were cased with PVC threaded pipes with one-way valve at the bottom end, and the annular space outside the PVC pipes was grouted with cement bentonite grout with bottom-up grouting technique.

The shear and Young's modulus as well as the Poisson's ratio were calculated using equations (1), (2) and (3) above. The results of the cross-hole logging are provided in Figure 4. The figure show graphs of P & S velocities, dynamic modulus (shear and Young), and Poisson's ratio versus depth.

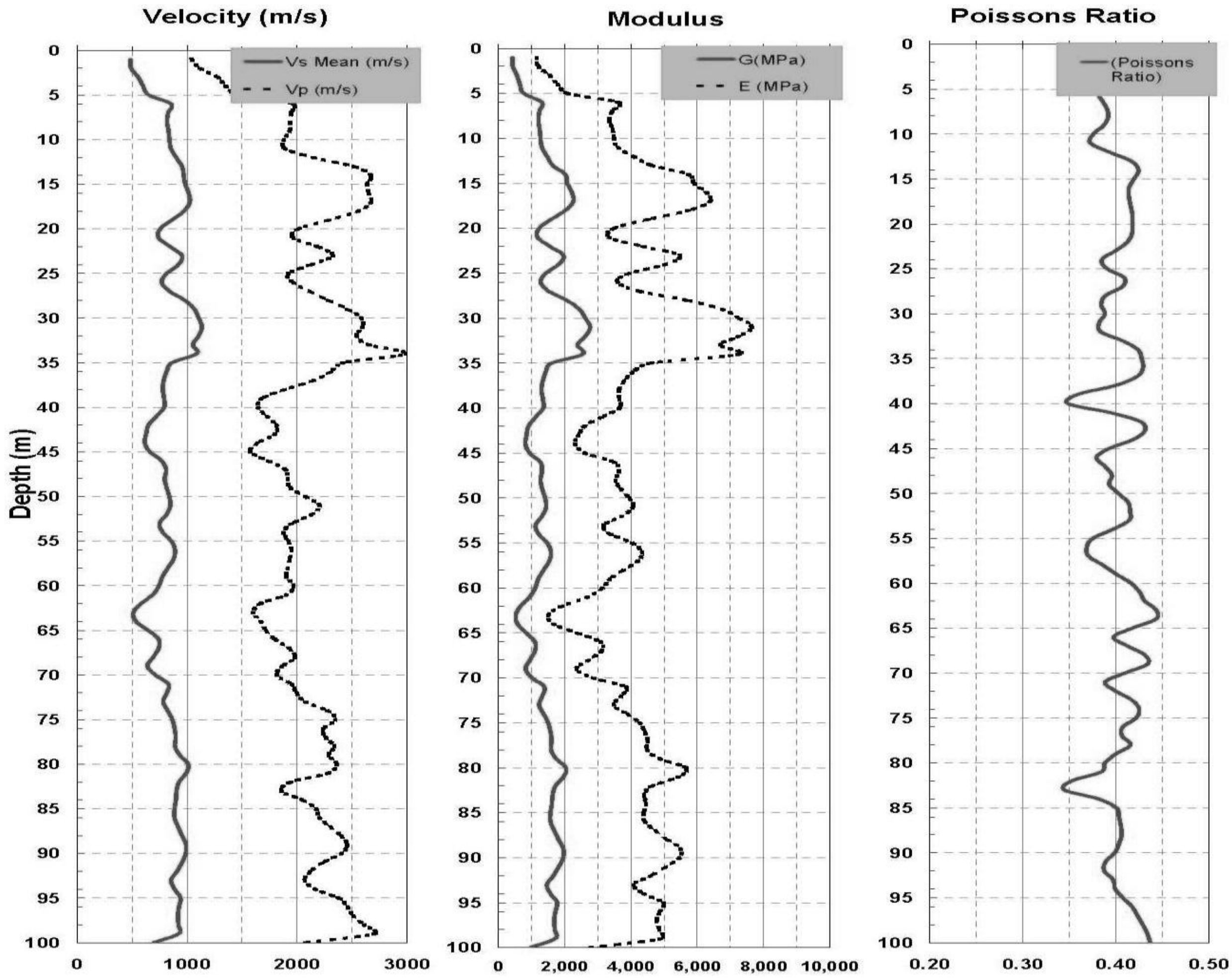

Figure 4: Results of cross-hole test

LABORATORY TESTING PROGRAM

Compressive Strength Testing

Uniaxial compression and point load strength tests were carried out on intact rock specimens retrieved from the drilled boreholes. Figure 5(a) shows the compressive strength results from uniaxial compression and point load tests. It is also possible to obtain an estimate of the deformation modulus of a jointed rock mass from empirical relationships with the uniaxial compressive strength of the intact rock. Hoek and Brown (1997) proposed a relationship between the in-situ modulus of deformation (E_m) and uniaxial compressive strength (q_c) employing Geological Strength Index (GSI) classification system as follows:

$$E_m(GPa) = \sqrt{\frac{q_c(MPa)}{100}}10^{\left(\frac{GSI-10}{40}\right)} \tag{4}$$

BS 8004 proposed another empirical relationship to obtain deformation modulus of jointed rock mass from uniaxial compressive strength of intact rocks as follow:

$$E_m = j \times M_r \times q_c \tag{5}$$

where j is the rock mass factor and M_r is the modulus ratio.

Figure 5(b) presents the in-situ deformation modulus versus depth from the two empirical methods. The following parameters were used in equations (4) and (5) based on the rock type and structure encountered at the site: GSI ~ 40-50; j ~ 0.2-0.5; and M_r ~ 300-400. The figure shows that the modulus values obtained when using BS 8004 are markedly lower than those obtained when using the Hoek and Brown correlation.

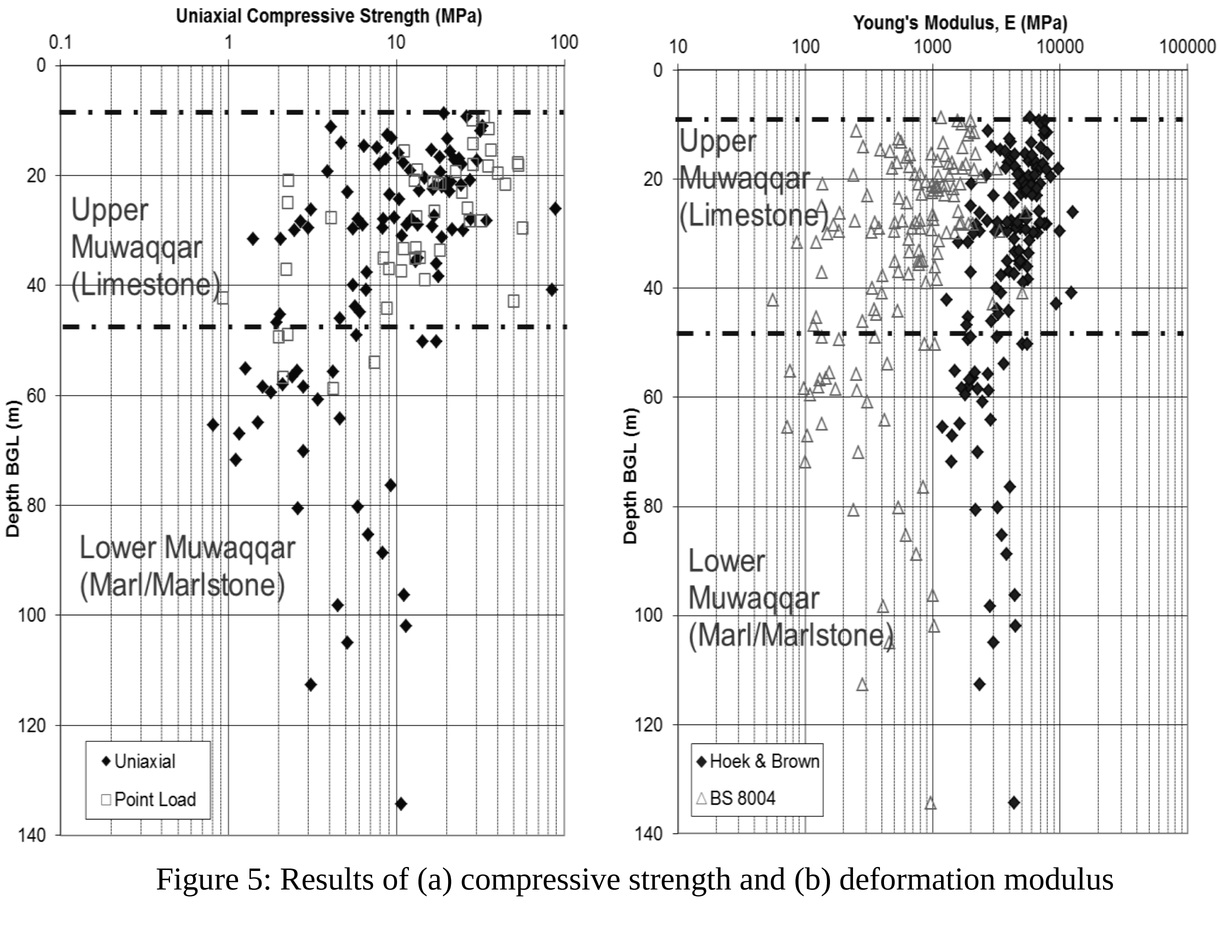

Figure 5: Results of (a) compressive strength and (b) deformation modulus

Resonant Column Testing

Resonant column tests were performed on intact rock specimens using fixed-free Stokoe-type apparatus to determine the dynamic soil properties at small strains (e.g., shear wave velocity, shear & Young's moduli, Poisson's ratio, damping ratio). These tests were performed by an external agency outside Jordan. The basic operational principle is to vibrate the cylindrical specimen in first-mode torsional motion. Harmonic torsional excitation is applied (by running current through the electromagnet drive head) to the top of the specimen over a range in frequencies, and the variation of the acceleration amplitude of the specimen with frequency is obtained (measured by an accelerometer mounted on the drive head). Once first-mode resonance is established, measurements of the resonant frequency and amplitude of vibration are made. These measurements are then combined with equipment characteristics and specimen size to calculate shear wave velocity and shear modulus based on elastic wave propagation.

The tests were carried out in a confining system (employing stainless steel chamber) where the tests results were obtained at different confinement pressures (confinement pressures were selected based on the in situ mean effective stresses). The rock materials were trimmed to prepare the test specimens. Figure 6 presents the non-linear deformation characteristics (shear modulus reduction curves, i.e., G – log shear strain [γ] curves) from resonant column tests for the limestone, marl/marlstone and bituminous limestone and marlstone materials.

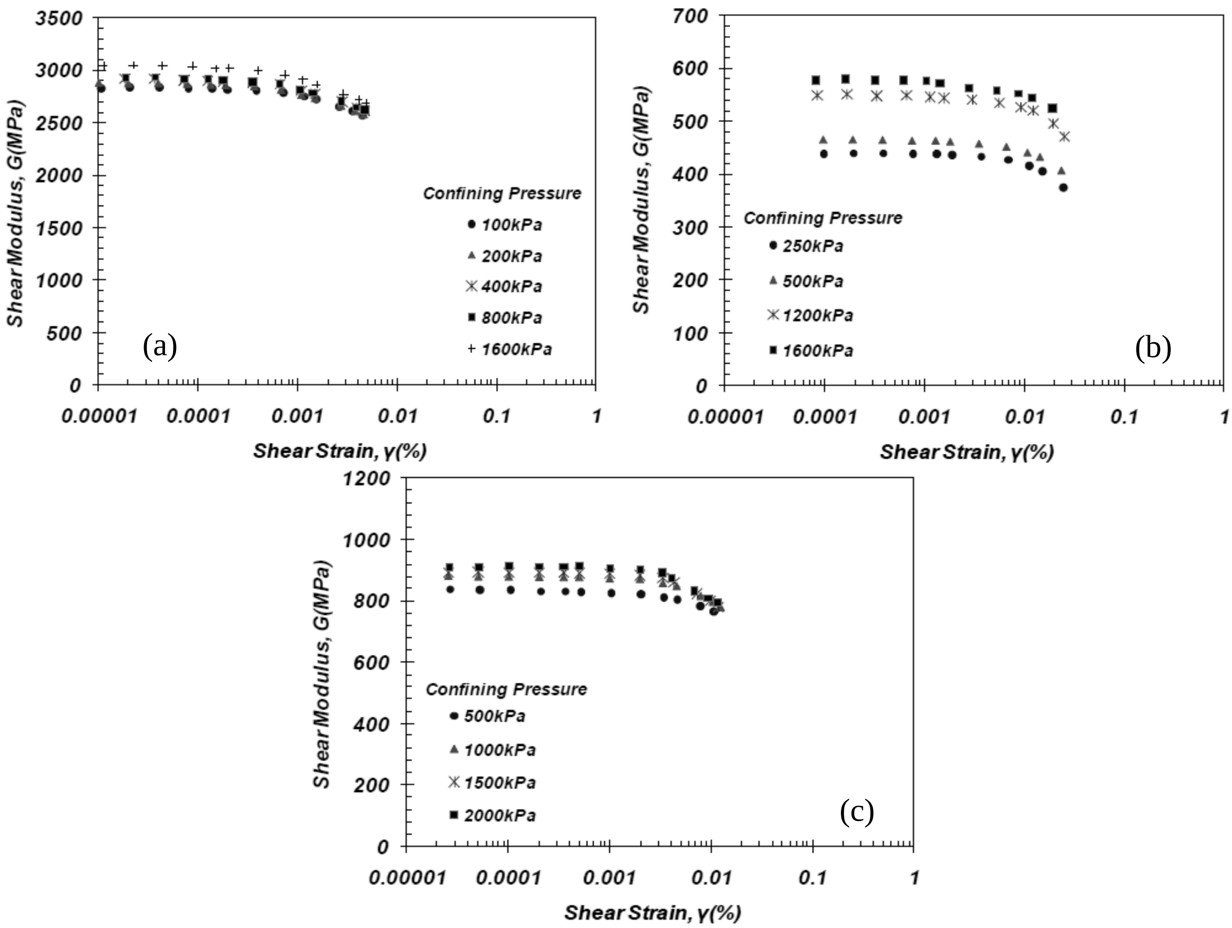

Figure 6: Results of resonant column test for (a) limestone (b) marlstone (c) bituminous material

Torsional Shear Testing

The torsional shear test is another method for determining the shear modulus along with other dynamic properties using the same fixed-free Stokoe-type apparatus but operating it in a different manner. A cyclic torsional force with a given frequency, generally below 10Hz, is applied at the top of the specimen. Instead of determining the resonant frequency, the stress-strain hysteresis loop is determined from measuring the torque-twist response of the specimen. Proximitors are used to measure the angle of twist while the voltage applied to the coil is calibrated to yield torque. Shear modulus is calculated from the slope of a line through the end points of the hysteresis loop. These tests were also performed by an external agency outside Jordan.

Like the resonant column test, the torsional shear tests were carried out in a confining system where the tests results were obtained at different confinement pressures. Figure 7 presents the non-linear deformation characteristics (shear modulus reduction curves) from torsional shear tests for the limestone, marl/marlstone and bituminous limestone and marlstone materials.

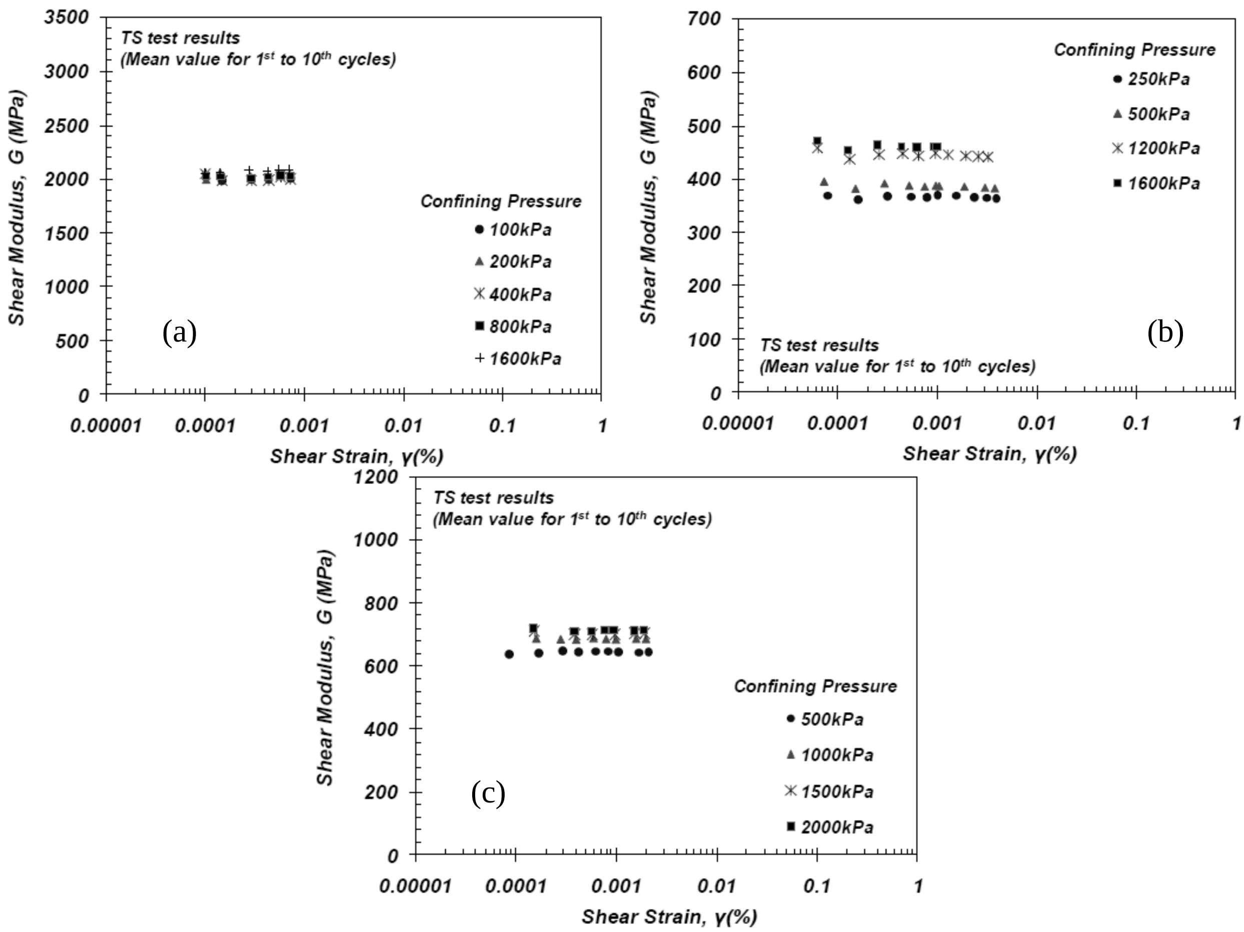

Figure 7: Results of torsional shear test for (a) limestone (b) marlstone (c) bituminous material

Ultrasonic Pulse Testing

Ultrasonic pulse velocity tests were carried out on marlstone and limestone rock core specimens using PROCEQ ultrasonic velocity equipment. Similar method to those described in ASTM C 597 and BS 1881-203 for measurement of ultrasonic pulse velocity in concrete were adopted. These tests were conducted to determine the pulse compressional wave velocity and subsequently the elastic modulus of the material at zero confining pressures. The rock specimens were mounted between the transmitter and receiver transducer holders. The velocity of ultrasonic waves covering the length of the specimens can be calculated by measuring the time between sending and receiving waves. The dynamic Young's modulus can be determined from the ultrasonic velocity (v) using the following relationship

$$E_d = \rho v^2 \frac{(1+\nu)(1-2\nu)}{(1-\nu)} \tag{6}$$

where the density (ρ) and Poisson's ratio (υ) were based on laboratory and geophysical test results.

The rock specimen details and the calculated ultrasonic pulse velocity and dynamic Young's modulus values are presented in Table 1.

Table 1: Results of ultrasonic pulse tests

Material	Depth (m)	Specimen Length, L (cm)	Sonic Velocity, v (km/s)	Dynamic Modulus, E_d (MPa)
Limestone	22	8.0	2.79	3734
Limestone	24	10.5	2.86	3833
Limestone	26	10.2	3.24	4345
Limestone	34	10.3	3.52	4732
Limestone	36	10.5	2.41	3233
Marlstone	45	10.9	1.22	1109
Marlstone	32	7.9	1.51	1364
Marlstone	56	6.3	1.18	1074
Marlstone	59	8.2	1.62	1477

COMPARISON OF FIELD AND LABORATORY MEASUREMENTS

Figure 8 presents and compares the Young's modulus from field and laboratory measurements. The modulus derived from the uniaxial compressive strength results was estimated using the Hoek and Brown (1997) empirical relationship and represents the in-situ rock mass stiffness at relatively large strain levels (~ 1%). The dynamic Young's modulus values from the resonant column and torsional shear tests were calculated from the shear modulus using the Poisson's ratio obtained from the geophysical survey. In addition, the results represent the very small strain stiffness (~ 0.001%) at the different confining pressures employed during the test in the laboratory. The intact small strain dynamic Young's modulus at zero confinement was attained from the ultrasonic pulse velocity tests. The pressuremeter modulus in Figure 8 represents the average values from the two unload/reload cycles carried out during the test. The seismic geophysical down-hole and cross-hole surveys were carried out to obtain the dynamic modulus of the in-situ rock mass.

In general, the stiffness results from the laboratory and field measurements compare relatively well. The modulus from the geophysical down-hole and cross-hole tests, which were carried out in the same borehole, are in good agreement. In addition, the pressuremeter stiffness values also agree with the geophysical results with the exception of one test. The Hoek and Brown empirical relationship proved to yield rock mass stiffness values remarkably similar to the field tests. The 'simple' ultrasonic velocity test results were close to the general modulus trend with depth whereas the dynamic resonant column and torsional shear tests underestimated the stiffness of the marl/marlstone materials but yielded reasonable results in limestone.

As mentioned earlier in the paper, the dynamic small strain stiffness is expected to be higher than those obtained from the static tests (Figure 1). In addition, the in-situ rock mass stiffness should be lower than intact values due to the presence of joints and other defects in the overall rock mass. However, the general agreement between the different methods could be due to compensating effects arising from disturbance during sampling, strain level, overburden pressure and joint structure among others. In addition, the agreement between the static and dynamic tests as well as the field and laboratory tests depends on the material type and complexity of the geological structure.

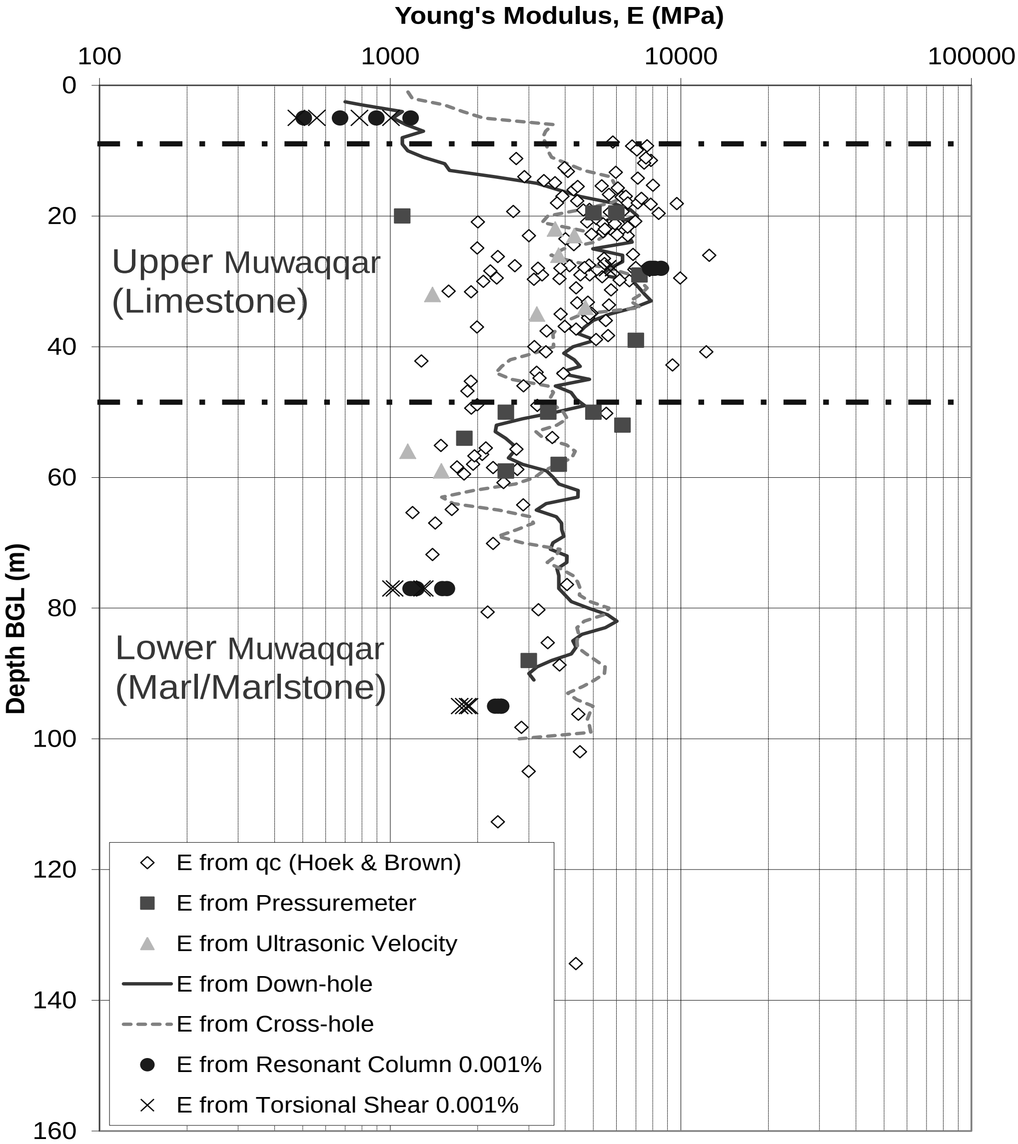

Figure 8: Comparison of Young's modulus values from field and laboratory measurments

CONCLUSIONS

The following conclusions may be drawn on the basis of the content of the present paper:

- There is reasonably good agreement between the static and dynamic stiffness results from the laboratory and field measurements.
- The stiffness values from the geophysical seismic cross-hole test compare fairly well with the down-hole test results.
- The rock mass stiffness estimated from the uniaxial compressive strength using the empirical relationship from BS 8004 yields results significantly lower than those obtained from other laboratory and field tests. On the other hand, the stiffness derived from the Hoek and Brown (1997) correlation agrees quite well with the other results.
- The 'simple' ultrasonic pulse velocity tests generated reasonable stiffness results when compared to other more elaborate methods.
- The results from the laboratory resonant column and torsional shear tests were slightly lower than the other methods in marl/marlstone but in good agreement in limestone. This is believed to be due to material disturbance during sampling.

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