Rock mass stiffness from large diameter plate load and CSW tests

Rigidité de la masse rocheuse provenant d'une charge d'une plaque de grand diamètre et des tests de CSW

Nico Vermeulen, Jacobus Breyl
Geotechnical Engineering, Jones & Wagener, South Africa, nico@jaws.co.za

Gerhard Heymann
Department of Civil Engineering, University of Pretoria, South Africa

ABSTRACT: Rock mass stiffness, as opposed to the stiffness of the intact rock, is one of the primary geotechnical parameters for the design of foundations on weathered rock. This paper presents a case study where rock mass stiffness of a typical sedimentary rock was measured in situ by various methods including continuous surface wave seismics and large diameter plate load testing. Observations from the test results are used to highlight a number of important aspects of soil and rock characteristics that need to be considered in the interpretation of field and laboratory tests. These aspects include the influence of shear strain level and of the rock mass structure on the stiffness measured by various test methods.

RÉSUMÉ : La rigidité d'une masse rocheuse mise en comparaison à la rigidité d'une roche intacte, est l'un des principaux paramètres géotechniques à prendre en compte pour la conception de fondations sur une roche résistante. Cette publication présente une étude de cas où la rigidité de la masse rocheuse d'une roche sédimentaire typique a été mesurée in situ par diverses méthodes incluant des ondes sismiques continues à la surface et de tests sur une charge d'une plaque de grand diamètre. Les observations des résultats du test ont été utilisées pour mettre en évidence un nombre important des aspects du sol et des caractéristiques de la roche qui ont besoin d'être pris en considération dans l'interprétation des tests sur le site et ceux du laboratoire. Ces aspects incluent l'influence du niveau de contrainte de cisaillement et de la structure de la masse rocheuse sur la rigidité mesurée par les diverses méthodes.

KEYWORDS: rock mass stiffness, CSW, plate load test.

1 INTRODUCTION.

The Mmamabula Energy Project is located on the Waterberg coal field near Mahalapye, Botswana. The Waterberg coal field comprises extensive coal deposits that lie east-west across the border between western Botswana and the north-eastern parts of South Africa, refer Figure 1.

During 2007 to 2009, extensive geotechnical investigations were undertaken for the proposed 2.5 GW coal fired Mmamabula Power Station. Of particular interest was the quantification of the rock mass stiffness, both under static loading and dynamic loading conditions associated with the heavy vibratory plant structures.

This paper presents the results and interpretation of plate load tests and continuous surface wave seismic tests to determine the in situ rock mass stiffness of the Jurassic sandstone and siltstone that underlie the plant site. The results illustrate the influence of strain level and rock jointing on the interpretation of the rock mass stiffness.

1 .1 *Geology*

The Mmamabula area is located directly below the Tropic of Capricorn in Botswana, between the A1 national road and the Tuli-farms on the border with South Africa.

The regional geology is characterized by a blanket of windblown Quaternary and Tertiary Kalahari sands, underlain by Jurassic sandstone, siltstone and mudstone of the Ecca Group, Karoo Supergroup. The base of the Karoo comprises glacial deposits (tillite) of the Dwyka Group. The pre-Karoo Proterozoic basement is an irregular buried glaciated land surface composed of folded Waterberg Group quartzite and Palaeo-proterozoic granite/gneiss. Economically viable coal seams are found within the Karoo sediments.

The transition from the transported Kalahari sands to the poorly represented residual soils and the rock head is often characterised by a well developed pedogenic horizon comprising ferricrete or calcrete.

The simplified Mmamabula site profile can be described as 2 m of windblown Kalahari sand, overlying 1 m of nodular to hardpan ferricrete, overlying about 0.5 m of poorly represented residual soils derived from sandstone or siltstone. The rock head is generally represented by soft rock sandstone, often intercalated with very soft rock to soft rock siltstone and weathered joints.

Exploration drilling in the area suggest that the permanent water table is located at depths of 33 m to in excess of 70 m.

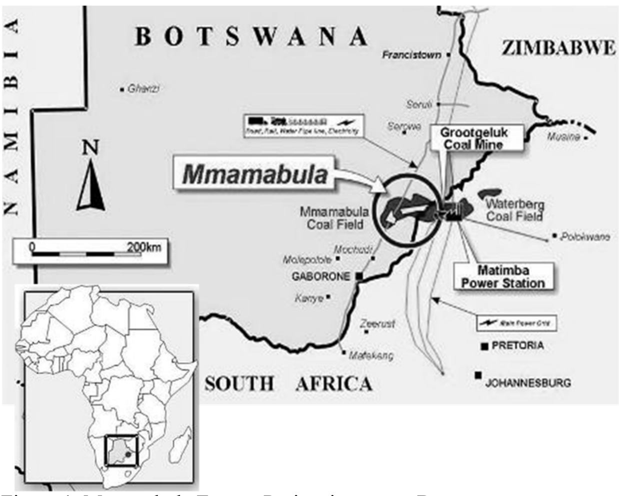

Figure 1. Mmamabula Energy Project in eastern Botswana.

2 SITE INVESTIGATIONS

Comprehensive investigations were carried out on the 100 ha power plant site in order to characterise the geotechnical properties of the foundation soils and rock. These included test pitting, core drilling, electrical resistivity and conductivity geophysics, penetrometer testing, plate load testing, continuous surface wave seismic tests and laboratory testing on representative soil and rock samples.

This paper focusses on the results of the large diameter plate load tests (LPLT) and the continuous surface wave tests (CSW) to determine the in situ rock mass stiffness.

2 .1 *Test sites*

Once the test pitting and drilling investigations were well advanced, the information from these investigations were used to select two test sites representative of a deeper rock head (Site 1) and shallower rock head (Site 2). These sites were selected for more detailed and multi-level plate load and CSW testing in an area where the heavy and sensitive power block structures are to be constructed. At Site 2, soft rock sandstone was present at 2.6 m below ground level (test level A) and soft rock siltstone at 4.8 m below ground level (test level B). The test results at Site 2 are discussed in detail below.

2 .2 *Test procedure*

The LPLT and CSW tests were carried out in the following sequence:

- At the center of the test site a CSW test was carried out at ground surface (test level i), i.e. on top of the Kalahari sands. A shaker and geophones were placed at the test level and the Rayleigh wave velocity measured at a range of frequencies.
- The Kalahari sands were stripped and a second CSW test conducted on top of the exposed ferricrete (test level ii).
- The first LPLT test platform for Test A was created at a depth where bedrock was first encountered and a third CSW test was carried out at this level.
- Four three strand cable rock anchors were installed to a depth of 16m to provide reaction for the plate jack, two on each side of the plate located at least 2 m from the plate edge.
- The test plate of 1.128 m diameter (1 m^2 area) was embedded onto the test surface using quick-setting cement to limit seating and bedding errors.
- The reaction beam was assembled and the anchors stressed to 400 kN each to prevent the beam from lifting-off under the full plate load of approximately 100 t. The reaction beam rested on concrete supports for the safety of personnel and equipment.
- The hydraulic jack, spacers, load cell, vibrating-wire displacement transducers (3no at 120°) and mechanical dial gauges (3no at 120°) were installed to record the load on the plate and the plate settlement. The settlement gauges were mounted on independent reference beams supported at least 1.5 m from the plate edge.
- A seating load of approximately 50 kN was applied and all settlement gauges zeroed. The test load was applied in increments to a maximum of 1000 kN (100 t) with multiple unload-reload cycles. During each load increment, the jack force and dial gauge settlement were recorded with time until the settlement of the plate stabilized under a constant load.
- On completion of the tests at the rock head (Test A), the central area of the test site was deepened by approximately 2 m using an excavator equipped with a hydraulic hammer. A second set of CSW tests and LPLTs (Test B) were carried out at the base of the core excavation using the same procedure as with Test A, see Figure 2.

Figure 2. LPLT arrangement at final depth, Test B, showing reaction beam, load cell, hydraulic jack, plate and reference beam.

2 .3 *CSW test results*

The seismic source for the CSW tests were two mechanical shakers with balanced counter rotating eccentric masses. Details of the shakers are given in Heymann (2013). Rayleigh wave phase velocities were measured under monotonic conditions at frequencies ranging between 8 Hz and 90 Hz. Inversion of the dispersion data was conducted using the neighborhood algorithm suggested by Wathelet (2008).

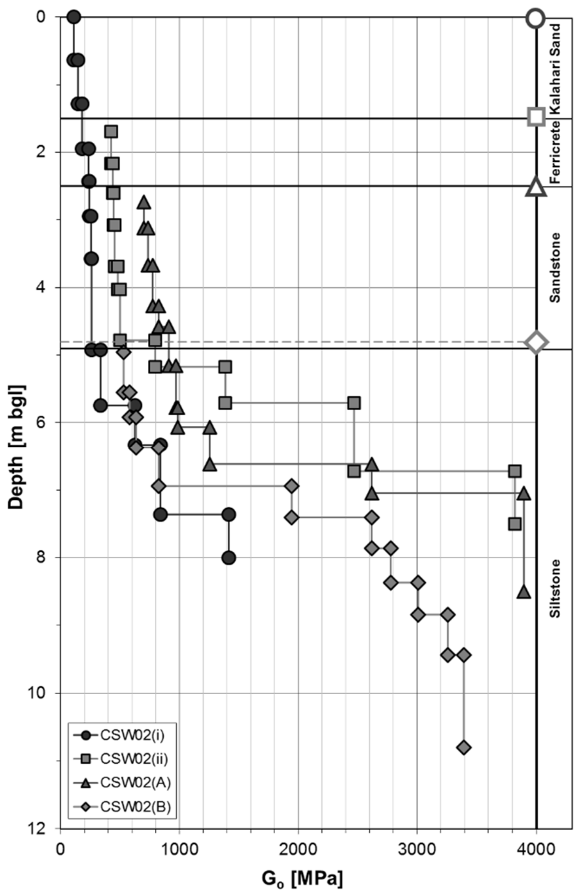

Figure 3. CSW test results at Site 2 showing inferred G_0 values as measured from four progressively deeper test depths.

The CSW test results at Site 2 are presented in Figure 3, where G_0 is the small strain shear modulus calculated from the shear wave velocities. The depth of measurement depends on the wave length of the Rayleigh wave. High frequencies induce short wave lengths which penetrate to shallow depth, whereas long wavelengths at low frequencies penetrate deeper. Low frequency phase velocity measurements are therefore affected by the shallow near surface material. In theory, the inversion analysis, where the inversion curve of a theoretical profile is matched to the measured dispersion data, should eliminate the masking effect that the shallow material has on deep measurements. Figure 3 shows that for the ferricrete and sandstone, as the CSW tests were progressively conducted at greater depth, the stiffness at the same level increased. This indicates that the inversion analysis did not fully account for the masking effect from the softer near surface layers.

2 .4 *LPLT results*

The LPLTs were carried out based on the guidelines of ASTM D1195-92 and ASTM D4394-84.

The load displacement results of the large plate load tests at Site 2 are presented in Figure 4: LPLT02(A) at the level of the rock head, and Figure 5: LPLT02(B) at 2 m below the rock head.

Both figures show relatively linear elastic behavior for loads up to 1 MN, equivalent to 1 MPa bearing stress. The net plastic deformation after unloading including seating, accounts for less than 20% of the maximum total deformation in both tests.

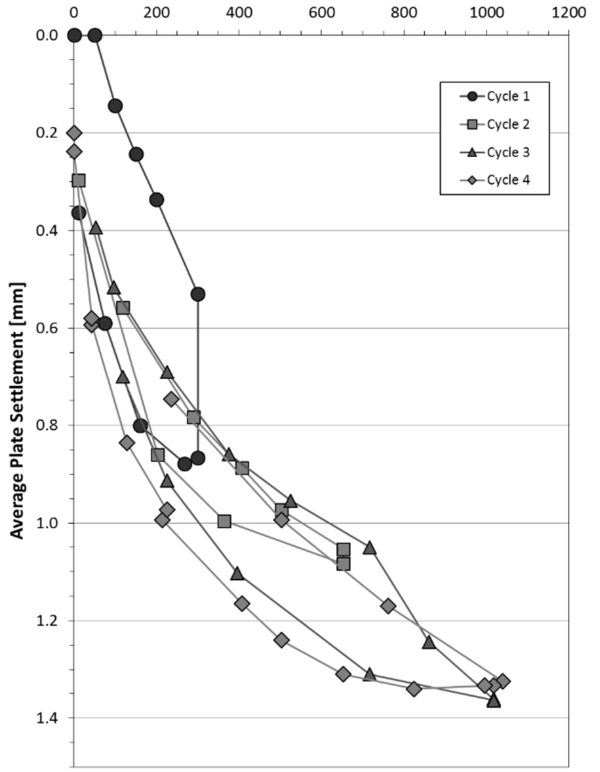

Figure 4. LPLT02(A) test results at 3.3 m bgl.

2 .5 *Rock mass description*

A geotechnical borehole, PP-BH108, was drilled at Site 2 prior to the CSW and plate load tests. The rock mass from 3.3 m to 4.9 m, relevant to Test A, is described as highly weathered, medium to closely jointed, coarse and medium grained, soft rock, sandstone with one rough undulating joint set at 5° to 10°. From 4.8 m to 10.3 m, relevant to Test B, the rock mass is described as medium weathered, closely jointed, thinly bedded, soft rock, siltstone with one smooth planar joint set at 5° to 10° and a smooth undulating horizontal bedding plane.

Table 1 presents point load index strength test results of intact rock core samples from within the LPLTs' influence depth.

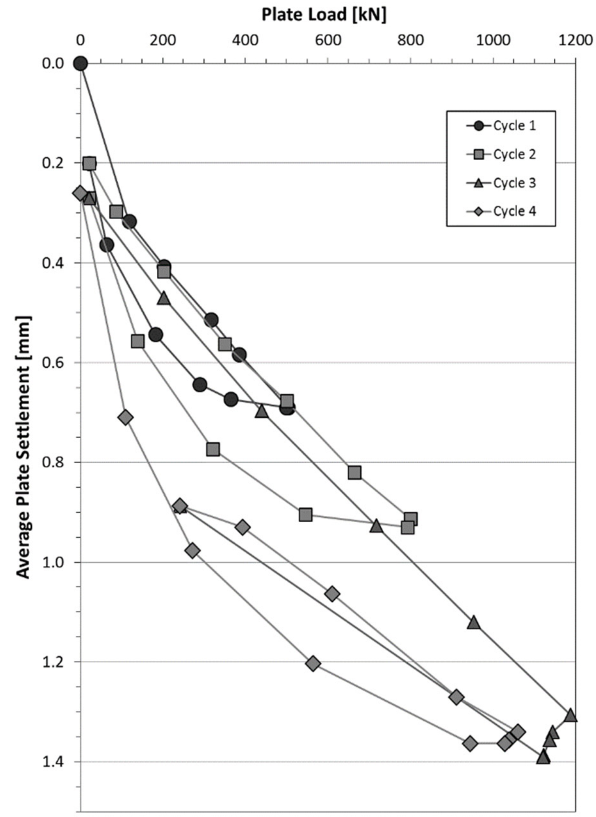

Figure 5. LPLT2(B) test results at 4.8 m bgl

Table 1. Point load test results from borehole PP-BH108 at Site 2.

Depth (m bgl)	I_{s50} (MPa)	UCS (MPa)	E (GPa)
3.0	0.11	4	1
5.2	1.11	47	8
7.4	2.43	102	17
6.6	1.02	43	7
8.5	1.90	80	14

The results in Table 1 for Site 2 are similar to the average obtained from 300 UCS (Unconfined Compressive Strength) and UCM (UCS with local axial strain measurement of stiffness) tests and almost 600 point load tests carried out on borehole core from the greater power station site. The results of the UCS and UCM tests were used to develop local site calibrations for deriving both UCS and stiffness values from the indirect point load index strength results for both the sandstone and siltstone.

Over the upper 8 m of the profile, the average UCS is 37 MPa and the average intact Young's modulus, E, is 8 GPa.

3 DISCUSSION

3.1 *Mmamabula case study*

If it is assumed that the influence depth of the plate load tests is approximately twice the plate diameter, i.e. elastic response, then it is possible to estimate the associated level of shear strain imposed during the test as follows:

$$\Delta\epsilon_v = \frac{\Delta d}{2D} \tag{1}$$

$$\Delta\gamma = 2\Delta\epsilon_s = 2\left[\frac{2}{3}\Delta\epsilon_v(1+\nu)\right] \tag{2}$$

Where γ = engineer's shear strain, ε_s = pure shear strain, ε_v = vertical strain, ν = Poisson's ratio (0.25 based on the laboratory UCM tests), d = average plate settlement and D = plate diameter.

Using Eq. 1 and Eq. 2, the tangent Young's modulus of the LPLT results can be expressed as a function of shear strain as shown by the symbol plots in Figure 6. The shallower Test A at the rock head shows a softer initial response before stabilizing at a stiffness of about 1 GPa. This behavior can be explained with reference to Table 1 which shows a lower intact rock strength and stiffness at a depth of 3 m where the rock is more weathered or decomposed. In addition, the plate load tests would also initially be affected by seating or bedding errors (despite the use of cement bedding) and thereafter by some closing-up and compression of the near horizontal joint and bedding planes.

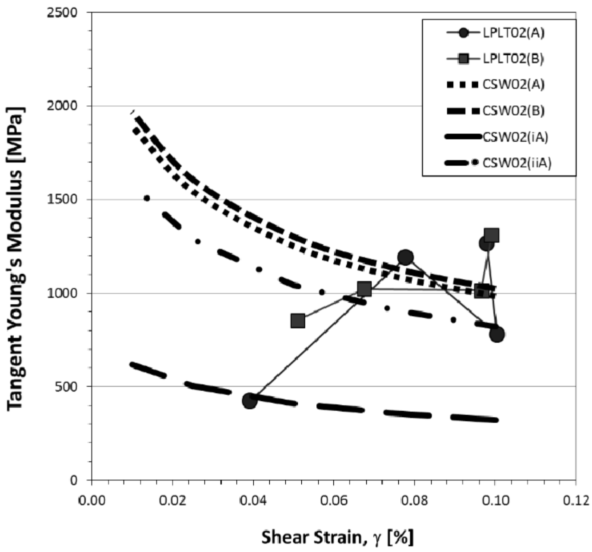

Figure 6. Inferred Young's modulus from LPLT & CSW tests at Site 2.

Also shown on Figure 6 are the traces of the Young's modulus derived from the CSW small strain shear stiffness measurements. The Young's modulus at small strain, E_0, was first calculated using Eq. 3. The modulus at strain levels comparable to the plate load tests were then calculated using the stiffness degradation model (average) proposed by Clayton and Heymann (2001).

$$E_0 = 2(1+\nu)G_0 \tag{3}$$

Where E_0 = small strain Young's modulus.

It was assumed that Young's modulus degrades at the same rate as shear modulus as a function of shear strain.

$$\frac{E}{E_0}(\gamma) \equiv \frac{G}{G_0}(\gamma) \tag{4}$$

Whereas the plate load rock mass stiffness increases with increasing strain for reasons discussed above, the CSW derived rock mass stiffness decreases with increasing strain from the small strain stiffness. The two methods approach similar stiffness values at about 0.06% to 0.1% shear strain for this case study.

Part of the reason why the CSW and LPLT stiffness estimates are so close to one another is the fact that both tests were conducted at the same level and on the same material. The CSW results were, therefore, unaffected by the potentially masking effects of the comparatively softer overburden sands. Figure 6 also shows the predicted stiffness trends for the rock mass at the rock head (test level A) based on CSW tests that were conducted at ground surface CSW02(iA), and at the pedogenic transition, CSW02(iiA). The results clearly show how the loose windblown sands mask the true stiffness of the rock mass. The dense to very dense pedogenic horizon (ferricrete) has a less marked masking effect on the rock mass stiffness and predicts values close to that measured on the rock itself.

3.2 *General interpretation*

The measurement of rock mass stiffness by in situ CSW and large diameter plate load tests compared well in this case study.

However, in order to extend the interpretation of these and similar geotechnical tests, the strain level at which a test is conducted and the structure and fabric of the material require consideration. To illustrate we consider three hypothetical cases and compare the expected stiffness values measured by laboratory tests with in situ CSW and PLTs respectively:

a) *Intact Rock Mass* - no jointing or discontinuities. In this case, all three test methods should theoretically measure the same stiffness at comparable strain levels.
b) *Horizontally Jointed Rock Mass* - with weathered joint fill. Laboratory tests on core samples are no longer representative of the rock mass conditions. The CSW will record the stiffness of the joint fill. If strain levels during the PLT are small enough to prevent closure of the joints and interaction between the intact rock units, then the CSW and PLT should measure similar stiffness values at comparable strain levels, but significantly lower than the laboratory test stiffness. Else, if closure of the joints occurs during the plate test, it will record higher stiffness values than the CSW. At Site 2, the average small strain rock stiffness, E_0, measured to a depth of 8 m with the CSW was less than half of the laboratory intact stiffness.
c) *Vertically Jointed Rock Mass* - with weathered joint fill. The CSW stiffness values are still likely to be dominated by the softer joint fill and should underestimate the vertical rock mass stiffness. The PLT stiffness will be more representative of the vertical rock mass stiffness which should be similar, if slightly lower, than the laboratory intact rock stiffness.

The above suggests that strain level, structure and load/response orientation should be considered when designing foundations based on the data from in situ and laboratory tests.

4 CONCLUSION

The Mmamabula power project afforded an unique opportunity to compare rock mass stiffness values as determined by CSW seismic geophysics and by physical large plate load tests.

The results illustrate that both methods independently predict similar rock mass stiffness at comparable shear strains for a horizontally jointed and bedded sedimentary rock mass.

5 REFERENCES

ASTM D1195-93. Standard test method for repetitive static plate load tests of soils and flexible pavement components, for use in Evaluation and design of airport and highway pavements.

ASTM D4394-84 (Reapproved 1998). Standard test method for determining in situ modulus of deformation of rock mass using rigid plate loading method.

Clayton C. R. I. and Heymann G. 2001. The stiffness of geomaterials at very small strains. *Géotechnique*, 51(3), 245-256.

Heymann, G (2013). Vibratory sources for continuous surface wave testing. *Geotechnical and Geophysical Site Characterization (ISC'4)*, Coutinho & Mayne (Eds.). Taylor and Francis Group, London, pp.1381-1386.

Wathelet M. (2008). An improved neighborhood algorithm: parameter conditions and dynamic scaling. *Geophysical Research Letters*, Vol. 35, No. 9, DOI:10.1029/2008GL033256, 2008.