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Why stabilise? Using triaxial tests for determining pavement stiffness and shear strength parameters of mechanically modified layers

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Triaxial testing of naturally occurring, slightly silty, medium-graded, coarse sand, derived from completely weathered granite (with some gypsum), compacted to 95 % of Mod AASHTO density generated unload/reload Young's modulus E-values of about 300 MPa at a representative confining stress of 90 kPa. This is some 80 % higher than what would usually be expected for this type of G5 material.

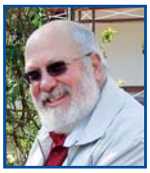
Mohr-Coulomb shear strength parameters of c = 15,9 kPa and φ = 51,4° were obtained from the high-quality triaxial tests.

Taking into account the variability of the materials, it is suggested that these be downgraded to c = 12,7 kPa and φ = 48,8° for use as design parameters.

It is common practice to stabilise natural gravel materials to generate weakly cemented sub-base layers. However, the presence of naturally occurring gypsum within the in-situ granite generated concern as negative effects were observed on elements of past construction.

The use of cement as a stabilising agent was eliminated by generating a nearly equally strong layer by mixing the naturally occurring gravels with varying quantities of crushed stone, crusher waste and dune sand, the latter to combat a high plasticity index. The best result was obtained by using a blend of 50 % natural gravel, 30 % crusher waste and 20 % dune sand.

These blended materials, generated unload/reload Young's modulus *E*-values of some 560 MPa at 95 % Mod AASHTO compaction and a confining stress of 247 kPa. This *E*-value is very similar to what is thought would be attained for a cracked and hydrated cement-stabilised layer but without the disruptive effects of a lowered confining stress that would be the case when a stabilised layer shrunk and cracked on hydration of the cement stabilising agent.



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Triaxial testing, unload/reload Young's modulus, confining stress, shear strength, airport pavement, mechanical modification

INTRODUCTION

Extensions to Walvis Bay Airport are at present taking place under the auspices of the Namibian Ministry of Works Transport and Communication. For works of this nature, laboratory testing is normally confined to gradings, Atterberg limits, maximum dry density (MDD)/optimum moisture content (OMC) and California bearing ratio (CBR) tests. However, on this site it was required that the modulus values of the various pavement layers be validated during construction to satisfy design requirements. Namibian Technical Services (NTS), under instruction of the Namibia-based civil engineering consultancy Windhoek Consulting Engineers (WCE), established an electronically controlled triaxial testing system to undertake this task. ARQ Consulting Engineers (Pty) Ltd of Pretoria was retained by NTS and WCE as a specialist advisor.

This paper details the triaxial testing system established for generating representative E (Young's modulus) and Mohr-Coulomb (c and ϕ) shear strength parameters for the pavement layers, comprising decomposed granite available from gravel pits located nearby as well as layers generated by mixing these naturally occurring materials with various proportions of crushed rock, crusher waste and dune sand.

TRIAXIAL TESTING

Historical perspective

Triaxial testing of materials used in pavement layers for roads (Maree 1979 and 1982), airfields, and as ballast and sub-ballast for

***Table 1** Types of triaxial test*

Conditions	Type of triaxial test		
	Nuu	Scu	Scd
Moisture conditions	Natural	Saturated	Saturated
Consolidation conditions	Unconsolidated	Consolidated	Consolidated
Drainage conditions	Undrained	Undrained	Drained
Typical scenario	End of construction	Rapid draw-down in an embankment dam	Long-term steady state or drained conditions in an embankment dam
Loading	Fast	Fast	Slow
Development of excess pore pressures	Not developed	Developed	Not developed

railways (Wolff 1985; Hugo & Engelbrecht 1982) is well documented. Gräbe and Clayton (2005) argue that triaxial testing is not suitable for interpreting pavement stresses, owing to principal stress rotation during application of load. However, the specialised nature of the hollow cylinder apparatus testing used by them is not widely available, except to extensively funded research organisations. Although there may be some reservations about the use of triaxial testing to predict stiffness and shear strength parameters for pavement layers, this method is more accurate than the empirical correlations established for relating stiffness to for example, the California bearing ratio (CBR) value.

Type of triaxial test

Triaxial testing (Lambe & Whitman 1969) is normally carried out under three main conditions. Table 1 details these.

The shear strength parameters derived from these tests may be expressed in terms of total or effective stress depending on the pore pressures which are developed.

Road and/or airfield pavements generally comprise granular materials which are usually not fully saturated and are subject to high rates of loading. The Scu test is not appropriate, as this models saturated conditions. The Scd test is also not appropriate, as this is meant for very slow rates of loading, also under saturated conditions.

Haupt (1980), Emery (1985) and Wolff (1992) established that most road pavements exhibit moisture contents, under service life, which are near to optimum moisture content (OMC). More cohesive materials tend to be above OMC while more granular materials are usually below OMC. Although repeated loading does cause some compaction of materials, the moisture contents are usually not sufficiently high to cause saturation. Ideally when triaxially testing road construction materials, as used in pavement layers, one would require that the moisture content is close to field conditions and that the rate of loading and unloading is also similar. Under these conditions it is highly unlikely that pore pressures will develop (as the moisture content is far from saturation) and also highly unlikely that drainage of the sample will occur (as rates of loading are too fast and the pavement layer is prevented from draining by the surfacing above and the subgrade below).

Test method D2850–03a of ASTM (2003) (referred to as D2850 below) describes the protocols for undertaking unconsolidated undrained (uu) testing on cohesive materials. It is not ideal for modelling the testing of pavement layer materials which are granular in nature (with a significant frictional or ϕ-component, although due to particle interlock, they do exhibit a noteworthy cohesion or c-intercept). However, the test is fast, modelling to some degree the rate of loading on a pavement, performed at in-situ or near OMC conditions (modelling fairly well the equilibrium moisture content (EMC) (Emery 1985), conditions which occur in pavement layers during service, and undrained (modelling the limited flow of water in pavement layers when subject to traffic loading).

Although not ideal, D2850 probably represents field conditions for granular pavement layers fairly closely. The procedures stipulated in the test method were followed to generate total (as opposed to effective) shear strength parameters. Deviations from the standard test method are listed below.

Largest particle size

The test mould used on the project was 100 mm in diameter. Clause 6.1 of D2850 indicates that the largest particle size shall be smaller than one sixth the specimen diameter. This criterion would generate a maximum particle size of some 17 mm. The standard sieve size closest to this value as utilised in South and Southern Africa is 19 mm. A 19 mm maximum size aggregate was used.

Sample preparation

Sample preparation as per TMH1 (1979) for the determination of maximum dry density (MDD), optimum moisture content (OMC) and California bearing ratio (CBR) is achieved by breaking down material larger than 19 mm in size and adding the broken fragments back into the sample. This has the effect of masking the actual properties of the material by generating a larger portion of high-quality materials.

It was reasoned that if a representative sample for the determination of the critical Young's modulus stiffness parameter, E, was to be obtained, this practice should be abolished for the triaxial testing. Accordingly in the preparation of the sample, those fragments larger than 19 mm in size were extracted from the sample and discarded.

Tamper size

The tamper size in D2850 is specified to be less than or equal to one half the mould diameter, while the actual tamper being used on this project was some 95 mm diameter. This criterion used in D2850 is probably to simulate the kneading effect that pad foot rollers (traditionally used to compact cohesive fills) have on the material properties. These compactions techniques tend to produce a dispersed or laminar and supple structure when compacted slightly wet of OMC which is usually the moisture content specified for most cohesive soils.

In contrast, the materials being utilised on site were very much less cohesive than the clay materials for which this test was specifically designed. The actual materials being used were likely to have a soil particle

Figure 1 *Mould used for sample preparation*

Figure 2 *Compaction of sample*

Figure 3 *Material slightly proud of mould*

structure midway between flocculated and dispersed when compacted at optimum moisture content in the field with traditional compaction equipment.

The size of tamper used tends to generate this structure, which is commensurate with the same field effects; thus the reason for deviating from the standard.

Number of layers

The standard specifies that the material be compacted in a least six layers. For this particular project, eight layers of 25 mm each are used in the preparation of the sample.

Samples were compacted at optimum moisture content to the densities specified for the pavement layer in question. Manually operated drop hammers were used with the

Figure 4 *Sample on triaxial base*

Figure 5 *Installing rubber membrane and O-ring seals*

Figure 6 *Details of test set-up prior to start*

surface of each layer being roughened before placement of the subsequent one.

The last layer was usually slightly proud of the surface and levelling was achieved by hammering flat with a straight edge.

Figures 1–6 detail the process followed in the preparation of the sample and insertion onto the base plate prior to triaxial shearing.

Rate of testing

The rate of strain specified in the standard test is 0,3 %/minute for brittle materials which achieve maximum deviator stress at 3 % to 6 % strain.

The granular materials being tested on site typically fail at about 1 % axial strain. Although a testing rate of half this value (that is, 0,15 %/minute) was tried and found to be adequate, there was no good reason why a rate faster than that specified in the standard should be used other than that it approximates better the rate of field loading.

A rate of 0,3 %/minute for a 200 mm long sample corresponds to a rate of movement of 0,6 mm/minute. The latter rate is very much slower, however, than actual load conditions where a velocity of 10 km/hour (typically exhibited by an aircraft when taxiing) would correspond to a movement in the order of 150 m/minute. Even taking inertial effects into account, the test represents a condition that is very much slower than the loadings in practice. Slower testing usually tends to produce lower bound values. In this light it was reasoned that the test would tend to produce conservative results.

The testing rate of 0,3 %/minute was supported for both initial loading and unload/reload cycles.

Application of confining stress

The D2850 standard specifies that the confining stress should be applied to the sample for approximately ten minutes to 'allow the specimen to stabilise under the chamber pressure ...'. To speed up the process, certain tests were performed on site during a preliminary programme by ignoring this protocol, since no stabilisation of the sample was observed to occur in the electronically monitored condition. However, this condition was followed in all future triaxial tests performed to monitor stiffness with the pneumatically applied confining stress, σ_3, applied to the specimen for ten minutes prior to shear loading.

MATERIALS

An analysis of 144 tests on materials envisaged for the pavement layers generated, in statistical format, the parameters as detailed in table 2.

The above would suggest that, unmodified, the above materials would be suitable for selected sub-grade layers and possibly, after modification, as sub-base.

The plasticity index PI = 13 is too high for sub-base as TRH 14 (1985) specifies a value of PI < 10 for G5 materials. Of significance, though, is the fact that some gypsum was observed to occur in most samples tested. Gypsum $CaSO_4.2H_2O$, when heated in air, is slowly converted to hemi-hydrate $CaSO_4.1/2\ H_2O$ at about 70 °C or below and rapidly at 90 °C and above. Heating gypsum at higher temperatures produces anhydrite $CaSO_4$. (Deer *et al* 1966). TMH1 (1979) requires moisture content determination by drying to constant mass at 105 °C. This would in all

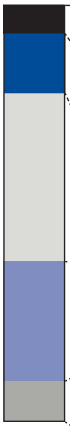
Table 2 *Statistical summary of tests on 144 samples*

	MDD	OMC	100	98	95	93	90	GM	LL	PI	LS
Mean	1 839	13	90	70	48	36	24	1,7	37	13	6
Standard deviation	177	3	26	19	14	11	8	0,4	9	5	2
Co-efficient of variation (%)	10	22	29	28	29	31	35	25,1	23	42	30

MDD = Maximum dry density under the Mod AASHTO compactive effort (kg/m^3)
OMC = Optimum moisture content under the Mod AASHTO compactive effort (%)
100 = California bearing ratio at 100 % of Mod AASHTO density
98 = California bearing ratio at 98 % of Mod AASHTO density
95 = California bearing ratio at 95 % of Mod AASHTO density
93 = California bearing ratio at 95 % of Mod AASHTO density
90 = California bearing ratio at 95 % of Mod AASHTO density
GM = Grading modulus
LL = Liquid limit
PI = Plasticity index
LS = Linear shrinkage

Results of layered elastic analysis with zero variation assumed
(Using SAMDM for Category A roads) (29 April 2005)

Project: Walvisbay Airport — Analysis: Evaluation of new design
Designer: H C de Wit — Boeing 747 main wheel with evaluation at X = 0 and Y = 0



Layer description and properties	Evaluation parameters and expected life
70 mm asphalt surfacing (0 % COV) Stiffness = 3 000 MPa (0 % COV) Poisson's ratio = 0,44	Maximum horizontal tensile strain Asphalt fatigue criterion for contin. graded 336,7 Microstrain expected life ≤ 0,1 MESA
150 mm granular base (0 % COV) Stiffness = 500 MPa (0 % COV) Poisson's ratio = 0,35	Safety factor stress Granular material safety factor (G1 Dry) 2 (SF) Expected life ≥ 99 MESA
425 mm cemented sub-base (0 % COV) Stiffness = 850 MPa (0 % COV) Poisson's ratio = 0,35	Maximum horizontal tensile strain Cemented material fatigue criterion (C4) 282 Microstrain expected life = 0,8 MESA
300 mm selected layer (0 % COV) Stiffness = 180 MPa (0 % COV) Poisson's ratio = 0,35	Safety factor stress Granular material safety factor (G6 Dry) 0,87 (SF) expected life = 0,6 MESA
Semi-inf sandy subgrade Stiffness = 150 MPa (0 % COV) Poisson's ratio = 0,35	Vertical compressive strain Sub-grade rutting criterion for 10 mm rut 582,8 microstrain expected life = 0,4 MESA

COV of tyre load = 0% — COV of contact pressure = 0 % — Lateral wander = 0 mm

Rubicon1 Ver 2.2.80 (1358-6515-E713-E719)

Figure 7 *Pavement design proposed for major portions of the project*

likelihood to some extent mask liquid limit and plasticity index evaluation.

With a CBR value indicating a G5 material (TRH 14 1985) when compacted to 95 % of Mod AASHTO density, the expectation would be, following the correlations established by Emery (1985), that an *E*-value of about 48 x 3,7 = 178 MPa would be attained for this material. However, the fairly high *PI* = 13 value may have the effect of decreasing the effective *E*-value.

REPRESENTATIVE CONFINING-STRESS VALUES

In order to arrive at representative E-values, to adjudicate whether the materials used on site comply with the requirements for which they were designed it was necessary to know what type of aircraft the design caters for and which pavement layer is being adjudged. Whereas truck traffic on a road in South Africa may typically impose a contact stress of some 800 kPa over an area of some 0,05 m^2 (De Beer *et al* 2004), the landing gear of a Boeing 747 aircraft imposes a contact stress of some 1 500 kPa over a much larger area (Packard 1967).

The above stresses are vertical contact stresses, but in order to arrive at representative horizontal σ_3-stresses, it was necessary to predict the effect that design traffic would have on the pavement. The pavement proposed for this site is given in figure 7.

An axi-symmetric finite element analysis using the Phase II software was performed on the design pavement under an idealised load imposed by a Boeing 747. The σ_3-values as depicted in table 3 were obtained for the midpoint of the pavement layer indicated.

It should be noted that the above values represent single-valued, best estimates of the actual σ_3-values that are likely to occur in practice. More detailed analysis – possibly using 3-d finite element models – may yield more accurate results, but are not expected to be materially different from those given above.

Also, as will be shown later, prediction models exist for conversion of the E-value conducted at a given confining stress to that which may exist under a different one.

Table 3 *Representative horizontal confining stresses*

Pavement layer	Horizontal stress (kPa)
Granular base	814
Cemented sub-base	247
Selected sub-grade	90
Sub-grade	70

REPRESENTATIVE YOUNG'S MODULUS E-VALUE

A typical triaxial test result with an unload-reload cycle is represented in figure 8.

Stress/strain curves for all materials tested triaxially generate an infinite number of *E*-values. The initial portion of the curve is sometimes concave which represents 'bedding-in' of the material prior to shear, thereafter the initial phase of the test, the unload/reload cycle and finally the failure portion.

For each test, conducted under a specific σ_3 or confining stress, the *E*-value varies from the initial value, E_i, which is the slope of the stress-strain curve near the axial strain $\varepsilon = 0$, to $E = 0$, where the sample fails. Therefore, not only is the *E*-value dependent on the confining stress, but it also depends on the strain or stress level within the sample and whether the sample is subject to the initial loading condition or under the action of a stress-reversal which would generate an unload/reload modulus value.

Since the loads imposed on a pavement structure are repetitive in nature, it is argued that the *E*-value most representative of actual loading conditions is the unload reload modulus (that is, E_{ur}) or that represented by the dark line approximately midway between the unload and reload cycles in the test.

This is the value which was evaluated for the granular pavement layers at the representative confining stress as given in table 3. As the strain at failure for most samples tested was about 1 %, the unload/reload cycle was performed at about half this value, that is, at $\varepsilon = 0,5$ %.

Figure 8 *Typical stress strain curve for subgrade material*

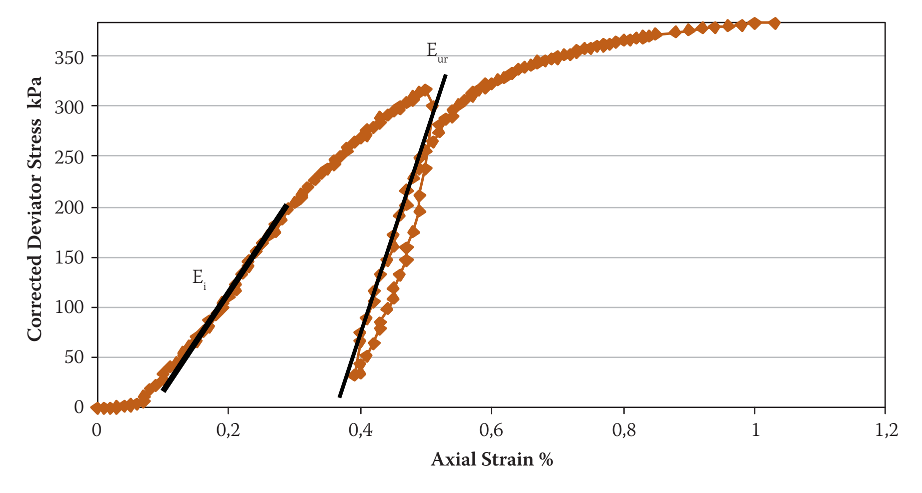


Table 4 *Typical material parameters*

Compaction standard	Optimum moisture content (OMC) (%)	Maximum dry density (MDD) (kN/m^3)	Void ratio	Degree of saturation (%)
Proctor 90%	11,3	17,1	0,52	57
NRB 95%	11,0	18,1	0,44	67
Mod 100%	10,8	19,1	0,36	80

FACTOR AFFECTING STIFFNESS PARAMETERS

Moisture and compaction conditions

As detailed in figure 8, the representative *E*-value determined is that generated by the mid-way point of the unload/reload cycle on a triaxial test performed on a material compacted to the specified field density at the optimum moisture content determined for the field compaction specified.

This latter statement is very important as the optimum moisture content (OMC) is dependent on the compactive-effort specified. The Mod AASHTO optimum moisture content, which is that used for materials that are to be compacted to 100 % of Mod AASHTO, is lower than that for the NRB effort, which usually generates about 95 % of the Mod AASHTO effort, and lower again than the Proctor effort, which in turn usually generates about 90 % of the Mod AASHTO effort.

For materials whose field density was specified at 95 % of Mod AASHTO, the sample was prepared using the NRB compactive effort and the optimum moisture content obtained was used for compaction of the samples triaxially tested. Similarly, if 90 % of the Mod AASHTO value was specified as the field density, the Proctor standard was used to determine the OMC value.

Table 4 details typical values for the granular materials encountered on site. It should be noted that the higher the compactive effort, the lower the OMC.

High confining stresses

The confining stresses specified in table 3 for all layers except the base course are less than 300 kPa. During testing on site it was found that at confining stresses below 300-400 kPa, the O-rings and seals on the equipment were quite capable of maintaining the pressure without leakage.

A test was attempted at a confining stress of 600 kPa, but leakage occurred. For the base course test, table 3 specifies a confining stress value of some 800 kPa. It was very difficult to conduct this test at such a high confining stress without serious damage to the equipment. The following method was recommended for the determination of the representative *E*-value at high confining stresses.

Hyperbolic parameters

Originally Duncan and Chang (1970), and later Duncan *et al* (1980), recommended a method for evaluating the results of triaxial data for calculating the hyperbolic parameters R_f, K and n, which may be used in the prediction of E-values under various conditions. Appendix A details the formulations, but in summary:

R_f This coefficient represents the ratio between the failure stress actually obtained in the triaxial test and the predicted ultimate strength of the material. It has a range of zero to unity

K This modulus number represents the strength or stiffness of the material. Typical ranges are 100 for a soft clay, to 3 000 to 4 000 for a very soft rock

n The so-called modulus exponent which dictates the rate a change of strength with confining stress

p_a atmospheric pressure

It was recommended that the following procedure be adopted for determining the representative *E*-values for the base course, or other portions of the pavement, where the representative confining stress exceeded 300 kPa:

- Perform three tests at confining stresses of 100, 200 and 400 kPa
- Perform two unload/reload cycles per test and determine a representative E_{ur} for each test, either by taking an average value or else deciding by visual inspection which cycle is most representative
- Conduct the Duncan *et al* (1980) analysis to determine the hyperbolic exponent n
- Calculate the E_{ur}-modulus for the σ_3 = 814 kPa condition by applying the fact that the *E*-value varies as per equation 1 in the same way as the ratio of $(\sigma_3/p_a)^n$, where p_a = atmospheric pressure and n is the hyperbolic exponent.

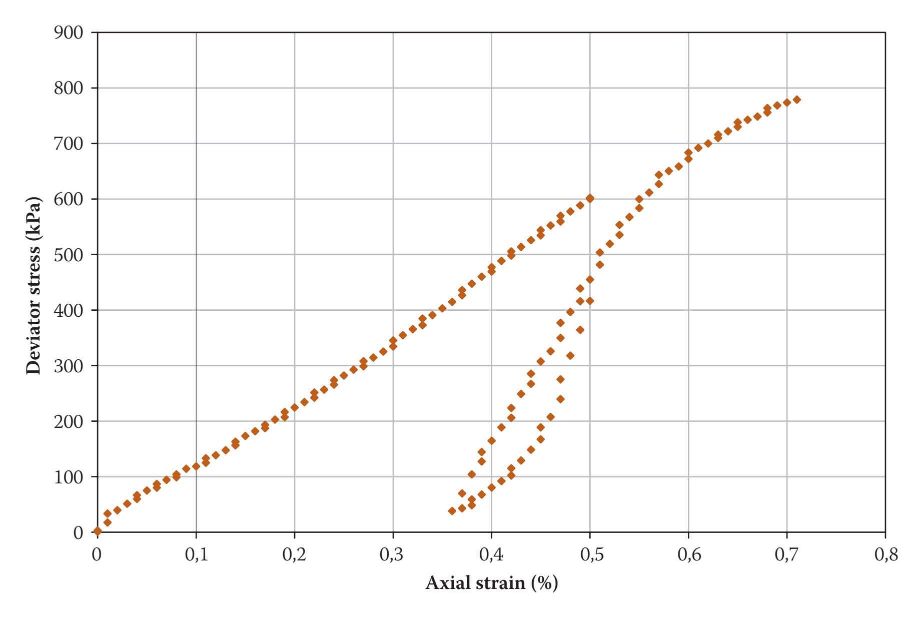
$$E_{i/ur} = K_{i/ur}\, p_a \left(\frac{\sigma_3}{p_a}\right)^n \qquad (1)$$

QUALITY OF TEST

In order to ensure only high-quality tests were used for confirmation or rejection of materials, it was suggested that the following simple checks be carried out:

- All tests on a material sample should be carried out using a minimum of three confining stresses; one stress at the precise value as detailed in table 2, one below it and one above it
- These three tests should be used to generate R_f, K and n parameters as detailed above
- In the plotting of the log-log graph to determine K and n, the regression coefficient r between the values of E_i/p_a and σ_3/p_a should exceed 0,95
- In addition, the regression coefficient between the average normal stress $p = (\sigma_1+\sigma_3)/2$ plotted as abscissa and the shear strength $q = (\sigma_1-\sigma_3)/2$ plotted as the ordinate, used in the derivation of the Mohr-Coulomb shear parameters a and α, should have a regression coefficient $r > 0{,}99$. The above definitions of p and q are taken from Lambe and Whitman (1969). If these conditions were not met, it

***Figure 9** Deviator stress vs axial strain triaxial test result for sample 189*



was usually a simple matter to determine which test was in error. The relevant test was repeated until the above conditions were met.

STABILISED MATERIALS

Materials proposed for use as sub-base are usually stabilised using either cement or lime to effectively generate a low-strength 'concrete'. However, the material in the pavement usually cracks due to shrinkage induced by hydration of the cementing agent. The elastic modulus or *E*-value proposed as per figure 7 for this 'shrunk and cracked' pavement layer was probably somewhat high at 850 MPa; 600 MPa was deemed more likely.

To simulate this state-of-the-material in a triaxial cell of 100 mm diameter and 200 mm length was very difficult, if not impossible. It was postulated that the following procedure be considered for ensuring that the testing regime modeled the as-constructed regime with some degree of similarity.

- Prepare the sample to the density, moisture and stabilising agent content as specified
- Allow to cure in a moist environment for the same number of hours that would be used during construction
- Test at the required confining stress and derive the E_{ur}-value
- Modify the curing period until an *E*-value of 600 MPa is consistently obtained and use this value as the base value for comparing samples obtained during field sampling

Subsequent to the above proposal it was found when analysing the pavement structure for fatigue and prediction of service life, as per the recommendations contained in Jooste (2004), that cracking of the stabilised layer in most cases represented a critical limiting criterion for long life. It was consequently decided that this layer should not be stabilised using chemical modification but constructed using mechanical modification of the natural gravel by adding various proportions of crushed stone, crusher waste and/or dune sand. As the representative confining stress for this layer was less than 300 kPa, no further testing problems were predicted and actual results obtained are detailed later.

HOT MIX ASPHALT SURFACING

Various methods may be used to determine the resilient modulus of asphalt specimens, but as the triaxial test equipment was available on site, the following procedure was suggested.

The surfacing materials are prepared in the same manner as the granular materials, except that compaction should take place at a sample temperature which matches the pavement laying temperatures and that bitumen or any modified form of binder forms the compaction fluid. Typical compaction temperatures are given in Taute *et al* (2001) or COLTO (1988).

Before testing the sample should be soaked and then allowed to drip dry.

The sample should be tested at temperature which matches the average hottest condition of the pavement during trafficking, at confining stresses of 100, 200 and 400 kPa. The same procedure should be adopted for the base course in predicting the E_{ur}-modulus at a confining stress of some 1 000 kPa, which is thought to closely model the field situation.

RESULTS OBTAINED FROM INITIAL TESTING

Initially some 36 tests were conducted on materials which were used for construction of the selected sub-grade layer. Figure 9 represents a stress strain curve for sample 189, a representative material at a density of 95 % of the Mod AAHSTO standard (18,1 kN/m^3) at OMC (11,1 %) and 90 kPa confining stress. The void ratio for this material, as per table 4, was e = 0,44, while the degree of saturation at this state of compaction was 67 %.

Hyperbolic parameters

Three samples, numbers 188, 189 (the stress strain curve which is depicted in figure 9, and volumetric parameters in table 4) and 190, were subjected to analysis as recommended by Duncan *et al* (1980). Representative parameters calculated as per the recommendations advocated in this paper were:

R_f = 0,57
K = 2 230
n = 1,20

The regression coefficient, r, generated in the derivation of the K and n parameters was r = 0,9999.

Shear strength parameters

The corresponding shear strength parameters derived from the three tests conducted at confining stresses of 50, 90 and 150 kPa were:

c = 15,9 kPa
ϕ = 51,4°

The regression coefficient generated in the p/q derivation of the above parameters was r = 0,9990.

Both the above r values satisfy the conditions suggested in the 'Quality of test' section.

Young's modulus E-value

The stiffness values derived from the unload/reload cycles of the three tests were 218, 296 and 402 MPa respectively.

Using the values of K = 2 230 and n = 1,20 derived above, an initial stiffness of

E_i = 2 230 x 101,6 x (90/101,6)^{1,2} or 196 MPa would be predicted using Duncan *et al* (1980) at a confining stress of 90 kPa.

The actual value measured in the unload/reload cycle was 296 MPa or some 1,5 times higher than the initial value.

Comparisons with other data

A dry density of 18,1 kN/m^3 (the 95 % Mod AASHTO optimum to which the sample

Figure 10 *Conversion graph from E_i to E_{ur}*

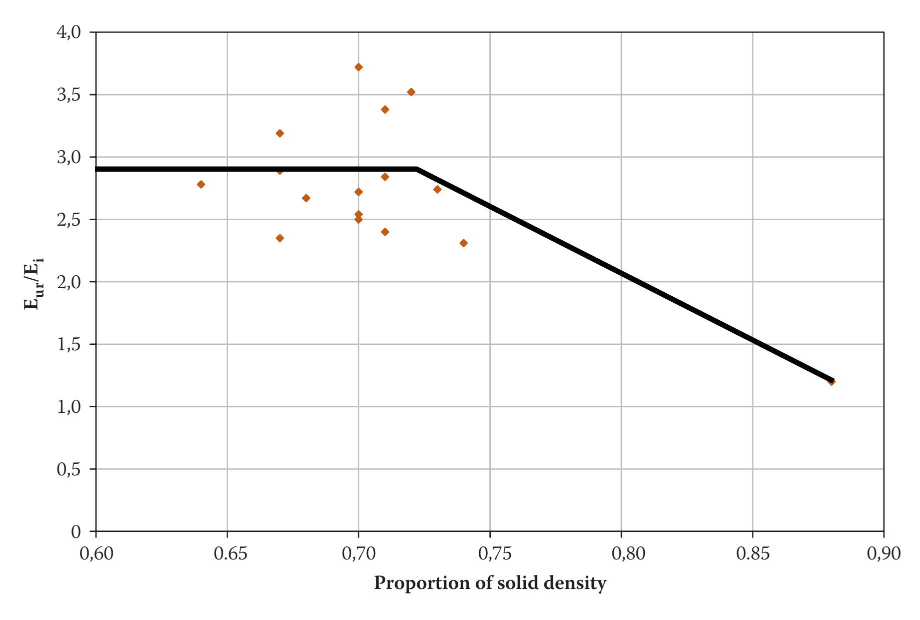


Table 5 *Statistical summary of initial testing performed on site*

	Density (% of MDD)	E_{ur} modulus at confining stress (kPa)					
	Confining stress (kPa)	10	20	30	50	100	200
Mean	90		34,0		55,9	100,9	123,5
Std dev			20,4		18,7	15,7	19,3
Mean	95		94,9		83,1	121,6	172,1
Std dev			71,7		16,8	39,0	55,7
Mean	98		161,0		208,4	248,0	271,9
Std dev			35,4		31,4	16,4	25,9
Mean	100	25,0	78,8	38,8	95,9	134,7	244,6
Std dev		26,9	51,5	28,2	51,6	68,5	194,8

Std dev = Standard deviation

was compacted prior to the test) represents a density of ≈70 % of solid for a material with an SG = 2,6. Duncan *et al* (1980), after analysing many different materials from good quality gravels classified as GW to high plasticity CH-clays, are of the opinion that for loose/soft materials – that is, those at a density of ≈70 % of solid – the ratio between the unload/reload modulus (E_{ur}) and the initial modulus (E_i), is in the range 1,5–3,0. For well-compacted, well-graded, low void-content materials (that is, those where the density was ≈88 % of solid), this ratio falls to about 1,2.

Data from a variety of tests performed on site were evaluated and the results are depicted in figure 10, which generally supports the findings of Duncan *et al* (1980) for low densities.

Further testing

Figures 11–14 illustrate the modulus values obtained over some 144 tests conducted during the initial phases of the project when lower quality materials were being encountered in borrow areas, while table 5 presents a statistical summary of the data.

The following is evident from figures 11–14 and table 5:

- Generally modulus values increase with increasing confining stress
- Generally modulus values increase with increasing compaction, but in the transition from 98 % to 100 % of Mod AASHTO it would appear that the reverse is the case. This is probably due to breakdown of the material under the high compactive effort

Tests on mechanically modified samples

Table 6 provides a summary of the results of tests performed using various combinations of the naturally occurring material from the borrow pit (BP), crusher stone (CS), crusher waste (CW) and dune sand (DS). Initially a 50:50 mixture of BP and CS produced average results with E-values varying between 260 MPa and 400 MPa for materials where the plasticity index was as high as PI = 14.

Various other combinations were tried, but that which appeared to generate the best results was with 50 % BP, 30 % CW and 20 % DS. Here an E-value of some 560 MPa was generated at 95 % Mod AASHTO compaction and 247 kPa confining stress. The addition of the dune sand had also lowered the PI to 9, while the CBR at 95 % Mod AASHTO density was a high 66. This PI-value may not be critical, as some researchers have indicated that bar linear shrinkage correlates better with CBR than PI (Parrock 2007).

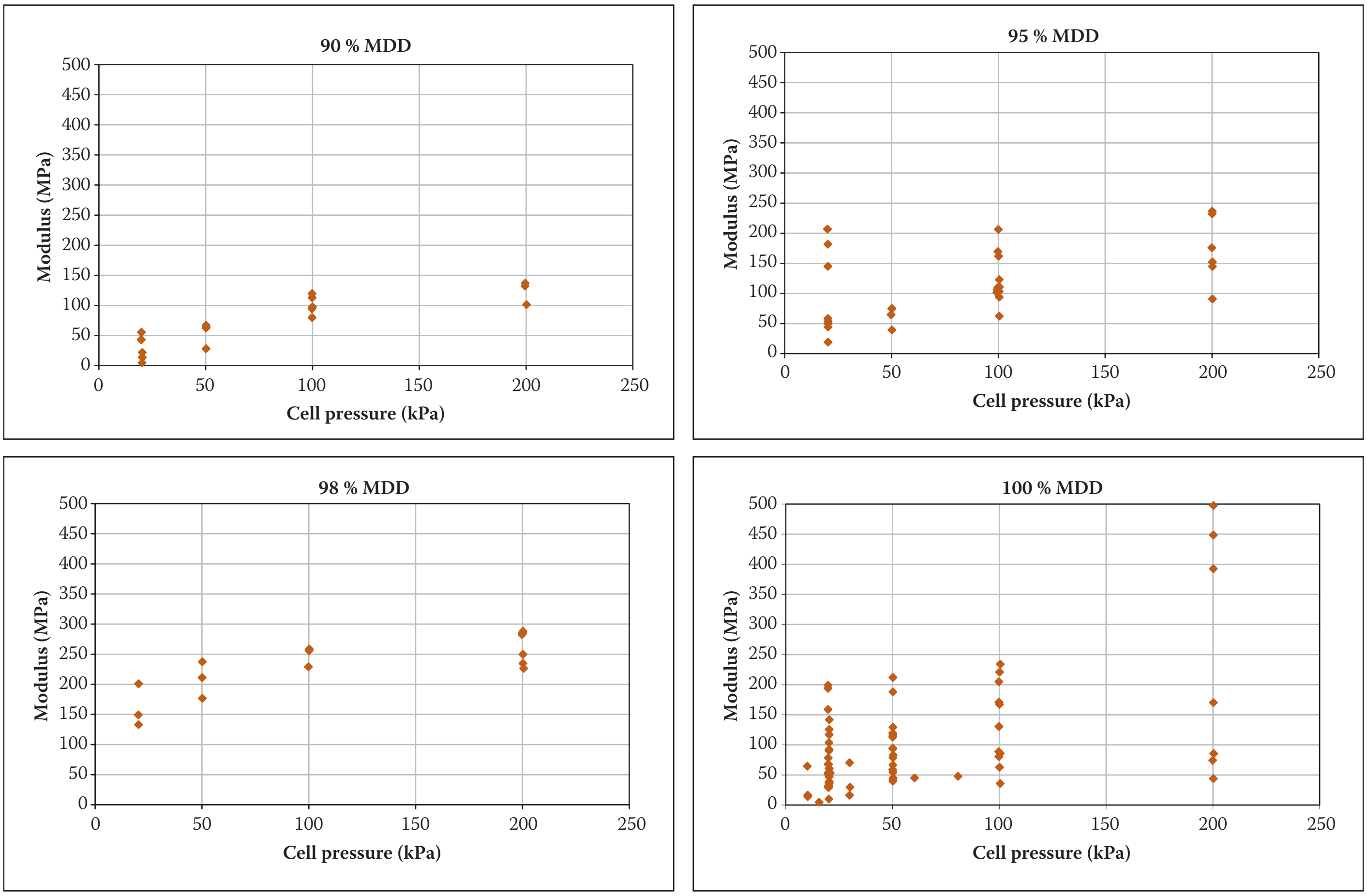
A very attractive alternative to a cement stabilised sub-base was accordingly generated which basically used low-cost locally available materials, blended to give a high-modulus, low-PI material with a good CBR value.

An added benefit using this approach was that the sub-base would comprise a homogeneous layer free of cracking in which the beneficial confining stresses generated during compaction would be locked into the layer. This is in contrast to a stabilised layer, which although initially exhibiting probably higher E-values, shrinks and cracks on hydration of the cement, effectively lowering the E value but significantly discarding the locked-in beneficial, compaction-induced horizontal stress. This is deemed a major disadvantage of stabilising.

CONCLUSIONS

- The in-situ materials being used from borrow pits on site would generally classify as G5 as per TRH14, (45 < CBR < 80) at 95 % Mod AASHTO density
- Utilising the correlations given in Emery (1985), it would be expected that a representative Young's modulus of E ≈ 178 MPa would be attained for these gravels
- The actual E_{ur}-value derived for this material was nearly 300 MPa or some 70 % higher than usually expected
- The plasticity index (PI) of the material (average PI = 13) would tend to suggest that the material may not be suitable as selected subgrade or sub-base pavement layers. This may be due to the fact that the presence of gypsum in the material was masking the free water availability
- Mohr-Coulomb shear strength parameters of
  c = 15,9 kPa
  ϕ = 51,4°
  were derived for the material when compacted to 95 % of the Mod AASHTO density standard. If, as detailed by Harr

***Figures 11–14** Representative unload/reload modulus values obtained during initial testing of borrow areas at 90, 95, 98 and 100% of the Mod AASHTO density standard*



(1987), a coefficient of variation of c is assumed at 40 % and that for ϕ of 10 %, and a cautious estimate of the design value, as advocated by Simpson (1995), Schneider (1997) and Simpson (1997) was calculated at the (mean – 0,5 times the standard deviation), then design values for the G5 borrow materials of:

c = 12,7 kPa

ϕ = 48,8°

would be relevant

- The above formulations may be used in the equations contained in Jooste (2004) to predict fatigue life of the actual pavement layer
- The natural gravel materials may be modified to enhance quality. Chemically they may be stabilised by the addition of lime, cement or cement blends. On the positive side, this would definitely lower the PI and cement particles together, but past experience on site indicated negative results due to the gypsum contained in the materials. Also, hydration would cause shrinkage and cracking of the layer with a loss of the beneficial compaction induced stress. The high cost of the appropriate stabilising agent in this area is also a negative factor
- Mechanical modification of the materials has been shown to be highly beneficial if the correct proportions are used. The testing on site has indicated that 50 % borrow material blended with 30 % crusher waste and 20 % dune sand generated very good results with a representative Young's modulus of E = 560 MPa and a plasticity index of PI = 9 being attained
- This latter combination of materials was selected for sub-base construction

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APPENDIX: THE EVALUATION OF NON-LINEAR STRESS-STRAIN HYPERBOLIC PARAMETERS OF SOILS

Introduction

The majority of soils (both sands and clays) exhibit behaviour over a wide stress range that is non-linear, inelastic, and dependent on the magnitude of the confining stress and has a variable Poisson's ratio. This behaviour may be modelled mathematically using the formulations listed below.

Stress-strain behaviour of soils

The following explanation has been taken from Duncan and Chang (1970) and Duncan *et al* (1980).

Non-linearity

Figure 1 illustrates the results of a typical triaxial test conducted on a soil at confining stresses of σ_3 = 50, 100 and 200 kPa.

Kondner (1963) and Kondner and fellow workers (1963a, 1963b, 1965) established that the stress-strain curves may be approximated by the following hyperbolic equation:

$$(\sigma_1 - \sigma_3) = \frac{\varepsilon}{a + b\varepsilon} \qquad (1)$$

in which $(\sigma_1\text{-}\sigma_3)$ is the deviator stress, ε the axial strain and a and b constants which may be derived by re-plotting the stress-strain curve on transformed axes as detailed in figure 2.

If equation 1 is rewritten in the form:

$$\frac{\varepsilon}{(\sigma_1 - \sigma_3)} = a + b\varepsilon \qquad (2)$$

then a will be the intercept and b the slope of the resulting straight line.

Now from equation 1:

$$\frac{lim(\sigma_1 - \sigma_3)}{\varepsilon \to \infty} = (\sigma_1 - \sigma_3)_{ult} = \frac{1}{b} \qquad (3)$$

Also if equation 1 is now differentiated with respect to, we have:

$$\frac{\partial(\sigma_1 - \sigma_3)}{\partial\varepsilon} = \frac{\partial u}{\partial v} = \frac{v\partial u - u\partial v}{v^2} \qquad (4)$$

$$\frac{(a+b\varepsilon)1 - \varepsilon b}{(a+b\varepsilon)^2} = \frac{a + b\varepsilon - b\varepsilon}{a^2 + 2ab\varepsilon - b^{\varepsilon}\varepsilon^2} \qquad (5)$$

Now for $\varepsilon = 0$

$$\frac{\partial(\sigma_1 - \sigma_3)}{\partial\varepsilon} = E_1 = \frac{1}{a} \qquad (6)$$

From figure 2 and equations 1-6 above it is seen that the constants a and b have actual physical meaning; a is the reciprocal of the initial tangent modulus E_i and b the reciprocal of the asymptotic value of stress difference which the stress-strain curve approaches at infinite strain. This asymptotic value of deviator stress is larger than the compressive strength or deviator stress at failure, by a small amount. If we call this failure ratio R_f we may express it as:

$$R_f = \frac{(\sigma_1 - \sigma_3)_f}{(\sigma_1 - \sigma_3)_{ult}} \qquad (7)$$

If we express a and b in terms of E_i and $(\sigma_1 - \sigma_3)_{ult}$ we may rewrite equation 1 as:

$$(\sigma_1 - \sigma_3) = \frac{\varepsilon}{\frac{1}{E_i} + \frac{\varepsilon R_f}{(\sigma_1 - \sigma_3)_f}} \qquad (8)$$

Stress-dependency

Except for unconsolidated-undrained tests on saturated soils, the tangent modulus values and compressive strength of geomechanical materials vary with confining stress. The relationship between initial tangent modulus and confining stress was determined experimentally by Janbu (1963) as:

$$E_i = K\, p_a \left(\frac{\sigma_3}{p_a}\right)^n \qquad (9)$$

Where:

E_i = initial tangent modulus
σ_3 = minor principal stress
p_a = atmospheric pressure expressed in the same units as E_i and $_3$
K = a modulus number
n = a modulus exponent

Mohr-Coulomb failure criterion

The relationship between compressive strength and confining stress may be derived from the Mohr-Coulomb failure condition as:

$$(\sigma_1 - \sigma_3)_f = \frac{2c\cos\phi + 2\sigma_3\sin\phi}{1 - \sin\phi} \qquad (10)$$

where c and ϕ are the Mohr-Coulomb strength parameters.

Tangent modulus

The tangent modulus corresponding to any point on the non-linear stress-strain curve may be expressed as:

$$E_t = \frac{\partial(\sigma_1 - \sigma_3)}{\partial\varepsilon} \qquad (11)$$

Performing the above differentiation on equation (8) one obtains the following expression for the tangent modulus:

$$E_t = \frac{\frac{1}{E_i}}{\left[\frac{1}{E_i} + \frac{\varepsilon R_f}{(\sigma_1 - \sigma_3)_f}\right]^2} \qquad (12)$$

The form of this equation is not very useful as the tangent modulus is related to both

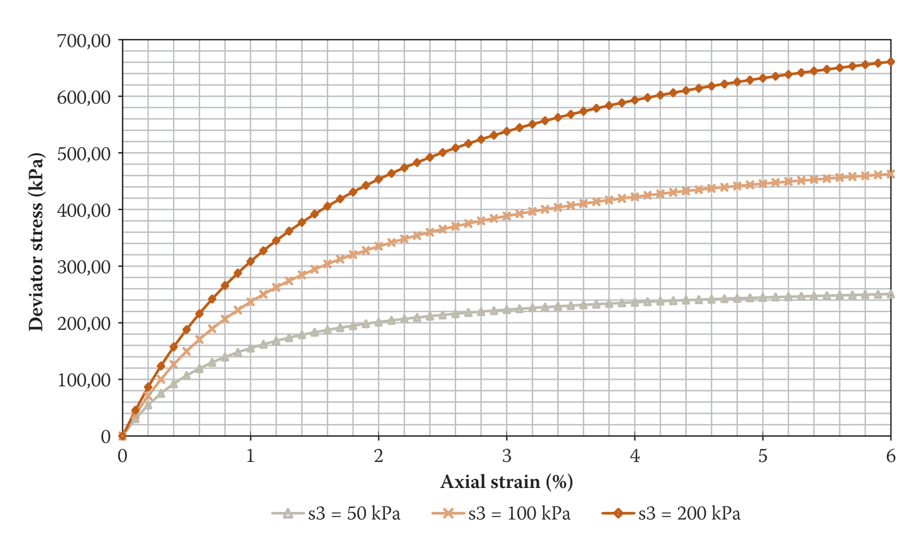

***Figure 1** Graphical output of a typical triaxial test*

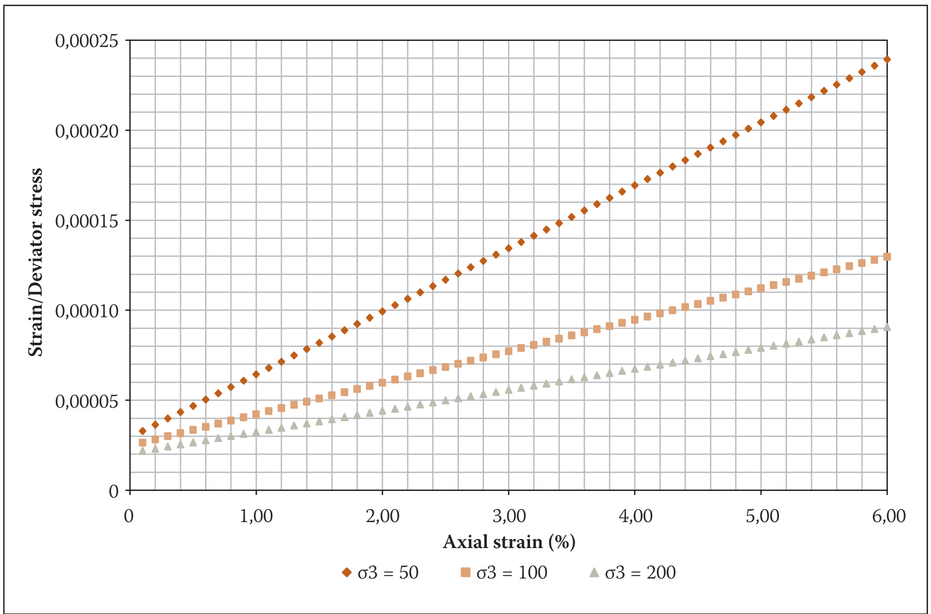

***Figure 2** Transformed axes plot of figure 1*

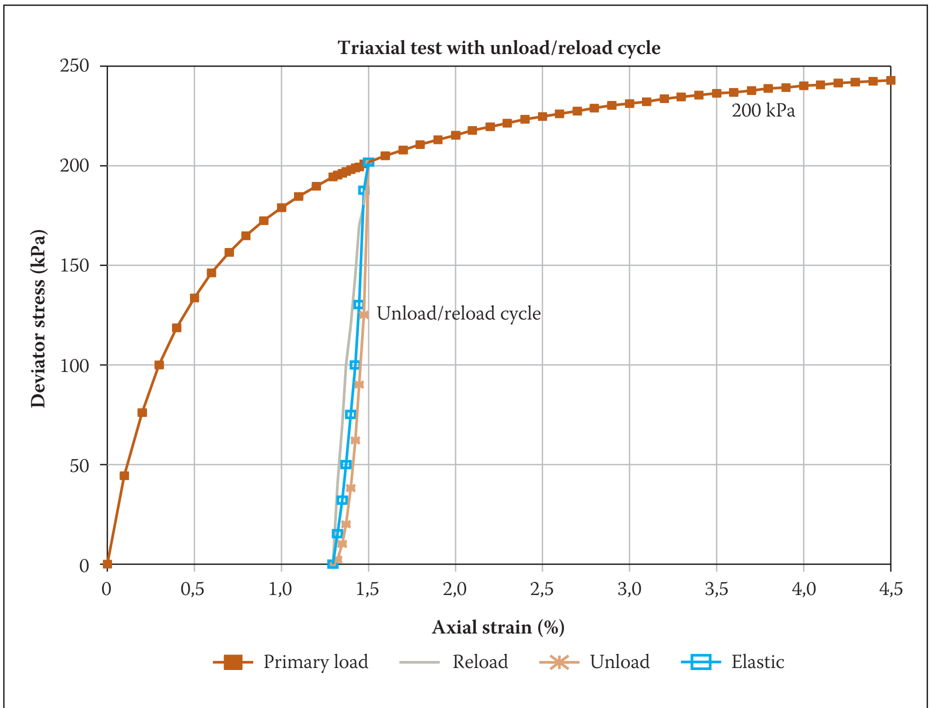

***Figure 3** Triaxial test with unload/reload cycle*

deviator stress and strain, which have different reference states. Consider for example the stress state in a soil mass before an external load is applied. The deviator stress may be uniquely defined at a given value, (depending for example on the overburden stress and the horizontal stress which is a function of K_0), but the state of strain may logically be referred to as zero. As the stress state is usually simple to define, we shall eliminate strain from the above equation and express the tangent modulus in terms of stress only. If equation 8 is rewritten as:

$$\varepsilon = \frac{\sigma_1 - \sigma_3}{E_i \left[1 - \frac{R_f(\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_f}\right]} \tag{13}$$

and the expression for strain substituted in equation 12, E_t may be expressed as:

$$E_t = (1 - R_f S)^2 E_i \tag{14}$$

Where

S = the stress level or the fraction of strength mobilised and is given by:

$$S = \frac{(\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_f} \tag{15}$$

If the expressions for E_i, $(\sigma_1 - \sigma_3)_f$ and S given by equations 9, 10 and 15 are substituted into equation 14 the tangent modulus for any stress condition is given by:

$$E_t = \left[1 - \frac{R_f(1 - \sin\phi)(\sigma_1 - \sigma_3)}{2c\cos\phi + 2\sigma_3\sin\phi}\right]^2 K p_a \left(\frac{\sigma_3}{p_a}\right)^n \tag{16}$$

Unload/reload cycles

If during a primary load cycle a soil is subject to an unload/reload cycle, the strains induced upon primary loading are only partially recovered on unloading. The soil behaves nearly elastically on reloading. If this unload-reload is repeated at different strains and stress levels the same modulus values are obtained. This is illustrated in figure 3.

This figure shows a typical unload/reload cycle illustrating the approximately linear behaviour. The unload/reload modulus thus obtained is independent of stress difference and is only dependent on the confining stress σ_3 and may be represented by:

$$E_{ur} = K_{ur} p_a \left(\frac{\sigma_3}{p_a}\right)^n \tag{17}$$

Where

E_{ur} = the unload/reload modulus

K_{ur} = the corresponding modulus number, and

n = the modulus exponent which for all practical purposes is the same for unload/reload as it is for primary loading

Poisson's ratio

Poisson's ratio may be calculated from:

$$\nu = \frac{\Delta\varepsilon_a - \Delta\varepsilon_v}{2\Delta\varepsilon_a} \tag{18}$$

Where:

ν = Poisson's ratio, and

ε_a, ε_v = axial and volumetric strains

The above may be use to evaluate Poisson's ratio at differing stress levels, or alternatively the constant bulk modulus approach as advocated by Duncan *et al.* (1980) may be incorporated. From the theory of elasticity the bulk modulus is defined as:

$$B = \frac{\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3}{3\Delta\varepsilon_v} \tag{19}$$

Where:

$\Delta\sigma_{1,\,2,\,3}$ = are the changes in the principal stresses

B = the bulk modulus and

$\Delta\varepsilon_v$ = the corresponding change in the volumetric strain

In a conventional triaxial test the minor principal stresses are held constant and the above equation reduces to:

$$B = \frac{(\sigma_1 - \sigma_3)}{3\Delta\varepsilon_v} \tag{20}$$

The parameter B may thus be evaluated by comparing corresponding points for deviator stress and volumetric strain.

As with Young's modulus, the bulk modulus B also varies with confining stress, the variation which may be approximated by:

$$B = K_b\, p_a \left(\frac{\sigma_3}{p_a}\right)^m \tag{21}$$

Where:

K_b = bulk modulus number and

m = bulk modulus exponent

The tangential Poisson's ratio (ν_t) is calculated from:

$$\nu_t = 0{,}5 - \frac{E_t}{6B} \tag{22}$$