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The performance of granitic, shale and limestone forest road aggregates under repeated loading

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ABSTRACT

This study compared the performance of three aggregate layerings, commonly used in the construction of unbound forest roads in Ireland, when they were subjected to repeated loading in a new large-scale test rig. These layerings comprised (i) a layer of uncrushed, granitic, sandy gravel - a good quality road aggregate (ii) a layer of shale - a poor quality aggregate, and (iii) a layer of crushed limestone – an excellent quality aggregate with a wet mix macadam (WMM) grading – on top of a poor quality shale sub-base layer . The repeated load testing rig was designed and constructed to test different surface or completion layering thicknesses of the aggregates over a common formation or subgrade

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material of silty sandy soil. This testing was achieved by surface loading the aggregates through a 200 mm-diameter rubber pad - attached to a hydraulic actuator on the test rig - for up to 150,000 load applications. The subgrade pressures and surface deflections were measured at applied stresses of 500 kPa, 750 kPa and 1000 kPa. The good quality granitic aggregate performed much better than the poor quality shale aggregate under the repeated loading and is suitable as a completion material for use in unbound forest roads. The shale aggregate can be used in unbound forest roads as a sub base material.

CE Database subject headings: Access roads; aggregates; load bearing capacity; load tests; loads; forests.

INTRODUCTION

The Irish forestry company, Coillte Teoranta, is the biggest constructor of unbound roads in Ireland and builds approximately 300 km of new and upgraded roads per year. Forest roads should ideally be constructed using high quality aggregates. These roads should be built as economically as possible, while achieving a standard of road that is structurally capable of doing its job. In many areas in Ireland these quality aggregates are not available locally. In some cases a decision has to be made between importing higher quality material at high prices, or using greater thicknesses of poorer local material in the road construction.

The pavement layers in unbound forest roads are normally defined as follows:

(1) the formation layer is the underlying prepared *in-situ* soil under the road, more commonly known as the subgrade.

(2) the completion layer is the top layer of the road. In forest road construction, the completion layer can be constructed from suitable local aggregates. If it's necessary to place a layer of imported aggregate on top of a layer of local aggregate, then the local aggregate layer is commonly known as the sub base.

Well graded aggregates of high strength and durability, and compacted at their optimum water content (OWC), can form a strong road pavement completion layer that reduces the transfer of excessive applied stresses from moving vehicular wheels to lower strength sub base and subgrade layers. Also, the sub base and subgrade layers must be of adequate thickness, compaction and strength to accommodate, at acceptable deformations, the stresses transferred through the completion layer.

Kennedy (1985) and Dawson et al. (1993) stated that suitable materials for granular layers in a road pavement should have a high stiffness to give good load spreading properties, and high shear strength to reduce rutting under construction traffic. They should also have a high permeability to allow surface water to drain freely and quickly, have non-plastic fines to maintain strength under wet conditions and not be susceptible to frost damage (Kennedy 1985). Unbound roads with a high proportion of unsuitable fine-grained completion material may be subject to surface disintegration due to its low shear strength (Simonsen and Isacsson 1999; Lekarp et al. 2000). As the contact pressure from a tire is mainly supported by the completion layer, the load from the tire can increase the pore water pressure in the road material when drainage is restricted. This pore water pressure increase can make unsuitable completion material unstable and may result in permanent deformation of the road surface (Simonsen and Isacsson 1999).

Indentations, called ruts, can develop at the surface of the completion layer over time. The rut depth, s (mm), may be calculated for geosynthetic, reinforced pavements from Giroud and Han (2004a):

$$s = \frac{\frac{P}{\pi r^2} f_s}{\left[\dfrac{h + 0.204h(R_E - 1)}{\left(0.868r + r\left(0.661 - 1.006J^2\right)\left(\frac{r}{h}\right)^{1.5}\log N\right)} + 1\right]^2 \left[\left[1 - 0.9\exp\left[-\left(\frac{r}{h}\right)^2\right]\right] N_C c_u\right]} \qquad [1]$$

where J is the aperture stability modulus of the geogrid (m N/^{o}); r, the radius of the equivalent tire contact area (m); h, the depth of the completion layer (m); N, the number of loading cycles; P, the wheel load (kN), N_c, the bearing capacity factor; f_s, the maximum allowable rut depth (75mm); and c_u, the undrained cohesion of the formation layer (kPa). The limited modulus ratio, R_E, can be calculated from Giroud and Han (2004a):

$$R_E = \min\left(\frac{E_{cl}}{E_{fl}}, 5.0\right) = \min\left(\frac{3.48CBR_{cl}^{0.3}}{CBR_{fl}}, 5.0\right) \qquad [2]$$

where E_{cl} and E_{fl} are the completion layer and formation layer resilient moduli, respectively (MPa), and CBR_{cl} and CBR_{fl} are the California Bearing Ratios (%) of the completion layer and formation layer aggregates, respectively. The design method of Giroud and Han (2004a) is unique insofar as it is theoretically based and experimentally calibrated, and the inter-relationships between various parameters (stress distribution, traffic volume, rut depth, etc.) are contained within a single equation, whereas more than one equation was needed with earlier methods (Giroud and Noiray, 1981; Giroud et al. 1985).

The study objectives were:

1. To design and build a repeated load testing machine that establishes the efficacy of using locally available aggregates in unbound forest road construction.
2. To collect, classify and perform repeated load tests on three aggregate materials that are currently used, singly or in combination, by Coillte Teoranta in Ireland for forest road construction. These materials were a good quality granite aggregate, a poor quality shale aggregate and a crushed limestone with a wet mix macadam grading (WMM).
3. To model the performance of the unbound aggregates using the finite element program, SIGMA/W, and Equations 1 and 2.

MATERIALS AND METHODS

Aggregate testing

The formation material consisted of a silty, sandy soil with small amounts of clay from Castledaly, County Galway. This soil was cohesive and was representative of subgrade soils found in Ireland. The three completion materials examined were a good quality granite aggregate from the Wicklow/Wexford region, a poor quality shale aggregate from the Leitrim region and a crushed limestone from a Galway quarry. The Wicklow/Wexford aggregate was an uncrushed, granitic, sandy gravel. It was chosen as the main sample for testing because its grading curve was almost entirely within the grading envelope for a wet mix macadam (WWM, Clause 810). The second aggregate was a shale and was extracted from a pit situated in the Arigna mountains near the village of Drumkeeran, Co. Leitrim. This aggregate is a mud shale, is soft and fissile, and disintegrates rapidly under loading and weathering. The crushed limestone, which is often used as completion material on top of poor local aggregates by Coillte Teoranta, was obtained from a quarry outside Galway City and graded to the specification of a WWM.

Classification tests, including tests for natural water content, Atterberg limits, specific gravity and particle size distribution, were carried out on both the formation and the completion materials in accordance with BS 1377 (1990).

The completion materials were also tested for durability - a measure of an aggregate's resistance to environmental influences like wetting, thermal expansion/contraction and freeze/thaw effects. Durability was tested using the magnesium sulphate soundness value (MSSV) test and the water absorption value (WAV) test. In bound roads, an MSSV $> 75\%$ is required for all road base and sub base aggregates, and a WAV $< 2\%$ is required for most road aggregates (BSI 812, 1990). The strength of the completion layer aggregates was also tested using the Aggregate Crushing Value (ACV) (BS 812 1990), the 10% Fines Value (TFV) (BS 812 1990), the Aggregate Impact Value (AIV) (BS 812 1990), the Aggregate Abrasion Value (AAV) (BS 812, 1990), and the California Bearing Ratio (CBR) (BS 1377 1990). In the field, the CBR of the completion material is dependent on the CBR of the formation (Giroud and Han 2004b). The cohesive strength of the formation material, c_u, was determined from the direct shear test (BS 1377 1990).

Placement of materials and instrumentation

The completion layer aggregates were compacted in a bin, on top of the 1000 mm-thick formation material, and tested at different thicknesses. The edges of the bin were sealed with silicone mastic and the bin was lined with a double layer of polythene. This was to ensure equilibrium of soil-water in the formation layer and to reduce friction along the bin sides. The formation material was compacted in the bin close to its maximum dry density as determined by the Proctor test (BS 1377 1990). Approximately 2,700 kg of soil was dried below the OWC, using an industrial gas heater, and placed in plastic bags, after removing particle sizes greater than 20 mm. The mass and water content of each bag of soil was calculated and recorded. The appropriate mass of water was added and mixed to each sample to increase the water content to its optimum value. The bags were then sealed to allow the soil-water to equilibrate. The soil was compacted in 50 mm layers in the bin to a height of 1000 mm, using a vibrating hammer with a 150 mm x 150 mm plate and an applied force of approximately 400 N. Water contents were taken at each layer and an average water content was calculated. The dry density and water content of the soil was also monitored using a nuclear density probe, at different heights, as the soil was placed. The preparation of the formation layer took approximately 4 - 5 weeks.

The completion materials were compacted, in 50 mm layers, close to their maximum dry density as determined by the vibrating hammer test. All particles significantly greater than 50 mm were removed from the completion materials to aid compaction. This removal had no significant change on the particle size distribution of the granite, but made the particle size distribution of the shale finer. The dry density and water content of the compacted materials were monitored using a nuclear density probe.

The resilient and permanent deflections, and resilient pressures in the soil, which occurred on load application, were measured using displacement linear strain conversion transducers (lscts) (MPE Transducers Ltd., UK), and hydraulic pressure cells, all with excitation voltages of 10 volts d.c. and an output range of 0-200 millivolts d.c. (resilient behaviour is also referred to as recoverable or elastic behaviour). The lscts were calibrated using a micrometer block and a computer program (LabVIEW™, National Instruments Ltd., Austin, USA) before use. The spindle axis of each soil surface lsct, as well as of each of two standard dial gauges, were positioned along a vertical plane that coincided with the central vertical axis of the loading pad, as shown in Figure 1. The maximum spindle travel distances of the lscts used in this study varied between 15 mm and 25 mm, the larger being positioned nearer the central vertical axis of the pad.

As the formation soil was being placed and compacted in the soil bin of the test rig, four 100 mm diameter x 6.5 mm deep pressure cells were positioned horizontally in the formation material (Figure 1) at heights of 300 mm (D), 500 mm (C), 700 mm (B) and 900 mm (A) above the soil base, with their centers coincident with the central vertical axis of the loading pad. Two 0-10 bar cells were placed towards the top of the bin at positions C and D, one 0-5 bar cell at position B and one 0-2 bar cell at position A.. The pressure cells were calibrated in a water-filled triaxial cell at a range of appropriate cell pressures. When placing the pressure cells, the soil was compacted to a height of 25 mm above the desired height of the cell. A 50 mm-deep recess for the cell and cables was then dug out of the compacted soil. Layers of coarse-to-fine soil were placed under and over the cells, with the fines closest to the face of the cells. The next 50 mm layer of soil was added and compacted. This method of placement protected the pressure cells from possible damage due to compaction, as there was at least 75 mm of soil between the pressure cell and the vibrating hammer. Rubber tubing was placed along the pressure cell cables for extra protection.

Loading rig construction

A repeated-load testing machine was designed and constructed to apply pressures, similar to that of a truck tire, to the aggregate materials (Figure 1). The load frame was designed to withstand several hundred thousand cycles of loading up to 40 kN with minimal deflections. It comprised two simply supported steel frames of universal beams (Steel Grade 43) constructed in parallel on common steel base-plates. 305 x 165 UB 40 sections were used for both the beams and columns. Two No. 700 x 700 mm holes were broken out of the existing concrete floor and eight No. M20 Gr. 8.8 x 180 bolts were sunk to a depth of 140 mm in a 200 mm depth of C40 concrete (28 day strength = 40 N mm^{-2}) to provide a suitable reaction for the base-plates of the frame. Flat strips of metal were welded to the ends of the bolts to ensure complete grip in the concrete.

The design of the loading pad was similar to that used by Davitt (1982). The pad comprised a 200 mm diameter x 45 mm thick rubber disc (Dunlop, England) and was identical to that used in truck tires. The rubber was bonded to a robust steel frame, which was bolted to a universal joint which, in turn, was screwed onto the end of the actuator piston. The purpose of the universal joint was to ensure that the surface of the pad remained parallel to the surface of the soil should any differential deformation occur during testing. In this study, vertical pressures were applied to the unbound surface layer, and resulted in a combination of vertical, horizontal and shear stresses in the completion and formation layer materials. Similar loading techniques have been used in other studies (Moghaddas Tafreshi and Khalaj, 2007).

The whole system was controlled and monitored by a programmable servo-amplifier that was mounted within an electrical enclosure. The programmable servo-controller (PSC) was programmed to drive the actuator to the desired loading cycle. The loading cycle had a 3-second duration (frequency = 0.33 Hz.), and comprised 1 second of loading and 2 seconds of recovery. Different levels of loading were applied in a cyclic manner to the aggregate material. Each level of loading was applied for a maximum of 50,000 cycles. The average contact area of a truck tire on a road is 175 mm x 225 mm (0.0394 m^2). A load of 29.43 kN over this area yields an applied pressure of 746 kPa.

The loading procedure adopted was as follows:

50 x 10^3 cycles at an applied pressure of 500 kPa (lightly-loaded axle)

50 x 10^3 cycles at an applied pressure of 750 kPa (normally-loaded axle)

50 x 10^3 cycles at an applied pressure of 1000 kPa (heavily-loaded axle).

The two lower loadings provided a level of conditioning in the completion material, which can occur on a forest access road during drainage and tree planting activities in peatland, and during about 20 years of limited site service traffic prior to harvesting and extraction of the tree trunks, which would subject the road to the heaviest loading. Unpaved roads are typically designed for 100,000 axle passes (Giroud and Han 2004a), so this test examined the performance of the aggregates in an extreme loading scenario.

The physical responses measured during the repeated load testing on the completion materials were:

(1) permanent surface deformation versus number of loading cycles using the lscts and dial gauges

(2) resilient surface deflections versus number of loading cycles

(3) resilient pressures in the pressure cells, located in the formation soil directly under the loading pad .

The completion material and the top 50 mm layer of the formation material were removed after each test. The top 50 mm layer was replaced with soil compacted at its OWC. A previous study on a similar completion material, 150 mm deep, which was subjected to 50 $x10^3$ cycles at 500 kPa, followed by 50 $x10^3$ cycles at 750 kPa and 50 $x10^3$cycles at 1000 kPa, resulted in little deformation in the formation material (about 2 mm) at the higher maximum resilient pressures (Rodgers et al., 2009). As the formation material was compacted in 50 mm layers, it was considered adequate to remove only the top 50 mm layer.

Six full-scale tests were performed on the different materials and are listed in Table 1. Surface deformations of the completion layer and resilient pressures in the formation layer were measured during cyclic loading for the following road structure arrangements and conditions: (i) 250 mm-deep (Test 1) and 150 mm-deep (Test 2) layers of granite, and 250 mm-deep (Test 3) layer of shale completion materials; (ii) capping of a 250 mm-deep shale layer with 200 mm-deep (Test 4) layer of crushed limestone graded to the specification of a WMM (a well-graded crushed rock); (iii) the effect of water addition to the surface of the limestone (Test 5), and (iv) the 1000 mm-deep formation soil on its own (Test 6). In Tests 1-5 the completion materials were tested on top of the 1000 mm deep formation soil. For Test 4, 200 mm of a crushed limestone was compacted at a water content of 3.3% onto a 250 mm layer of the shale to determine if the shale would perform satisfactorily as a sub base material. After completion of this 150 x 10^3 cycle test, 10 mm of water was sprayed onto the surface of the limestone and allowed to soak for one hour. This test (Test 5) was then started at an applied pressure of 500 kPa for a duration of 4 x 10^3 loading cycles and a comparison was made between the physical responses of this unbound layering in the dry state and in the wet state. In Test 6, the formation soil was tested under a repeated loading of 150 kPa without a completion material to determine its performance, and to provide resilient data for finite element program calibration to estimate E values for the formation soil. Approximately 3 - 4 weeks were required for the preparation, loading and analysis of Tests 1-4, 6.

Identification of model parameters

The validity of Eqn. 1 for the prediction of the rut depth, s, was investigated by comparing the measured versus the predicted values of s. As the aggregates were un-reinforced and unpaved, J = 0 and N_c=3.14. The radius of the tire contact area, r, was 0.1 m. As Eqn. 1 is only valid for CBR_{fl} ratios less than or equal to 5 (Giroud and Han 2004a), the E-values of the formation and completion layers were used to calculate R_E. In order to estimate the E values of the formation and completion materials for Eqn. 1, a series of elastic-plastic simulations with estimated E values were conducted using SIGMA/W (SIGMA/W, GEO-SLOPE International Ltd., Alberta, Canada), until the resilient pressures and deflections from these simulations were close to those recorded in the repeated loading experiments carried out in the test rig. The formation and completion material cohesion, c, and soil friction angle, Φ, - some of the parameters required for SIGMA/W to model residual responses - were determined from shear box tests. The Poisson's ratio, υ, for the formation and completion material was after Evdorides and Snaith (1996).

SIGMA/W contains three separate programs, Define, Solve and Contour. The Define program involves the plotting of the system geometry. Numeric parameters are defined by manually inputting the values. The Solve program is used to compute the deformations and stress changes. The Contour program graphs the computed parameters. SIGMA/W comprises eight elastic and plastic constitutive soil models, all of which may be applied to two-dimensional plane strain and axisymmetric problems. From the graphs of permanent deformation versus number of cycles, all the materials tested showed signs of plastic behavior due to their continual increase in permanent deformation with increasing number of loading cycles. The elastic-plastic model was therefore used to model the experimental results.

RESULTS AND DISCUSSION

Placement of materials

The granite aggregate was compacted at an average water content of 6.6 % (OWC, 8.3%) with an average dry density of 2.1 Mg m^{-3}; the shale was compacted at an average water content of 9.1% (OWC, 10.3%) with an average dry density of 1.5 Mg m^{-3} (Table 1).

Soil classification tests

The results from the soil classification tests are shown in Table 2 and the particle size distributions are in Figure 2. The formation was a well-graded formation soil, achieving high levels of strength, with a CBR of 15%, when compacted at its OWC. This soil was sensitive to water - an increase of 3% in water content resulted in a reduction in CBR to 2%.

The shale aggregate was a mud shale; poorly graded, flaky and lacking in fines, making it difficult to compact. The aggregate was low in strength and durability. The results of the ACV and the AIV tests for the shale, tested in its dry state, indicated that it could just meet most specifications for these tests. However, the CBR of the proposed shale completion aggregate was only 25.2% at its OWC and the ratio of its CBR ratio to that of the formation material (15% at OWC) was less than 2, indicating that a completion shale layer might not provide adequate strength in unbound road construction (Hammitt, 1970). The granite was a well-graded, sandy gravel, achieving high degrees of compaction. The aggregate was high in strength and durability and the CBR ratio of the granite to the formation material was approximately 7. The limestone aggregate had good strength and durability, was relatively well-graded and the CBR ratio of the limestone aggregate to the formation material was approximately 10.

Resilient pressures

In Test 1 (250 mm of granite), the pressure in Cell D (100 mm from the surface of the formation material), for an applied pressure of 1000 kPa, was approximately 98 kPa (Figure 3). In Test 2 (150 mm of granite), a reduction in the completion layer thickness from 250 mm to 150 mm resulted in an increase in the Cell D pressures. Test 3 (250 mm of shale) was stopped after $10x10^3$ cycles, at an applied pressure of 500 kPa, due to excessive deformations of the material. The pressures in Cell D in Test 3 increased from an initial value of 64 kPa to 104 kPa, suggesting a consistent weakening of the shale under loading. It can be concluded that the shale is a poor road making material; it failed dramatically under the low pressure of 500 kPa. This poor performance may be due, in part, to the low CBR ratio between the completion and formation materials (Giroud and Han 2004b). In Test 4 (200 mm of limestone aggregate on 250 mm of shale), the pressures in Cell D, at an applied pressure of 500 kPa, increased only from 31 kPa to 38 kPa in approximately 30 x 10^3 cycles of loading in comparison with the increase from 64 kPa to 104 kPa for the shale completion material at the same applied pressure in Test 3. After the $150x10^3$ cycle test on the limestone over shale sub base was completed, the material was wetted, and the test was restarted at the applied pressure of 500 kPa for a duration of $4x10^3$ cycles (Test 5), during which the pressures in Cell D increased from 50 kPa to 57 kPa (Figure 3) - an increase of about 55% due to the addition of the water.

Resilient deflections

The maximum resilient deflection at the centre of the loading pad in Test 1 (250 mm of granite) was approximately 1.2 mm for an applied pressure of 1000 kPa (Figure 4). In Test

2, a reduction from 250 mm to 150 mm in the granite thickness resulted in a maximum resilient deflection of 1.6 mm for an applied pressure of 1000 kPa, a 33% increase on the maximum deflection measured at the same pressure in Test 1. In Test 5 the addition of water to the limestone aggregate layer had a significant negative effect on its performance under loading. The maximum resilient deflection measured, at an applied pressure of 500 kPa, ranged from 0.72 mm - 0.8 mm in the dry state to 1.15 mm - 1.2 mm in the wet state - an average increase of 55%. Zakaria and Lees (1996) also found that the deflections of densely graded brick and quartz aggregates, when wetted, increased considerably under pressures of 210 kPa at less than 2000 cycles of loading.

Permanent deformations

In Test 1, the overall permanent deformation in the 250 mm-thick granite completion layer, directly under the loading pad, was 4.5 mm (Figure 5) after 150 x 10^3 cycles. This was not very much different from the surface deformation for the 150 mm granite completion layer in Test 2. The addition of the limestone layer on top of the shale in Test 4 improved on the performance of the shale alone in Test 3 but the limestone/shale performance was still poorer than the performances of the two granitic completion layers. The combined limestone/shale layering in Test 4 produced significantly lower resilient stresses than in Test 3 and prevented any deformation occurring in the formation material. However, the permanent deformations were much greater for the combined material test (Test 4) than for the granite completion layers in Tests 1 and 2.

Prediction of rut depths under repeated loadings

As all the materials tested showed a continual increase in permanent deformation with increasing number of loading cycles (Figure 5), the elastic-plastic SIGMA/W model was used in modelling the experimental results. The effectiveness of Eqn. 1 in predicting rut depths is dependent on: (i) the measured undrained cohesion of the formation soil and (ii) the E values of the formation and completion materials, estimated from SIGMA/W. Estimates of the E-values were made through calibrating resilient pressure and deflection values from the finite element model, SIGMA/W, with results measured in the rig experiments, and these E estimates are given in Table 3. The E values for the formation soil were estimated firstly from the resilient pressures and deflections measured in Test 6. These formation E estimates were then used to estimate the granite aggregate E values from the resilient results in Tests 1 and 2; Table 4 shows the excellent calibration achieved for the 250 mm thick granite aggregate layer - also excellent for the 150 mm thick granitic layer. The shale aggregate E value was obtained similarly from the resilient results in Test 3, but the calibration was only moderate for the shale aggregate. The limestone aggregate E value was then estimated from calibrating the Test 4 resilient results using the previously calibrated formation and shale E estimates; this Test 4 calibration was good for the limestone/shale aggregates.

The estimated E values of the granite - for similar stresses and densities used in the present study - were close to those given by (i) Hopkins et al. (2007) in their Figure 40, and (ii) Boudali and Robert (1997) from the equation for resilient modulus, $M_R = k_1.\ \theta^{k_2}$, where, for a granite aggregate, k_1 has a value of 8139 kPa, k_2 a value of 0.6, and θ is the sum of the principal stresses. The estimated E values of the limestone aggregate appear low in comparison with the granite values, particularly at the higher applied pressures of 750 and 1000 kPa; this could have resulted from the availability of only an estimated E shale aggregate value at the low pressure of 500 kPa in Test 3.

The estimated E values from SIGMA/W were used to predict the rut depths (permanent deformations) in the granite aggregates at three cycles – 50,000, 100,000 and 150,000 - for Tests 1 and 2 using Eqn. 1 (Table 5). When the E values of 29 MPa for the top 50 mm of the formation layer and 37 MPa for the formation soil below the top 50 mm layer were used in SIGMA/W, the simulated results of the permanent deformations in the two granite aggregate completion layers were within 2 mm of those measured in the loading rig.

The calculated and modeled rutting depths were of the same order as other studies. Zakaria and Lees (1996) measured rut depths of between 4 and 9 mm in brick and quartz aggregates, which were subjected to a tire contact pressure of up to 210 kPa in laboratory experiments. In the same study, the rut depth increased by between 30 and 80% when the material was saturated (the water content was not specified in the study). Other factors such as tire inflation and wheel load may also impact on rut depth, as Douglas (1997) found that significantly shallower ruts formed in a Gault clay aggregate, subjected to tracking wheel loads at up to 10,000 passes, when the tire inflation pressure was reduced from 690 kPa to 345 kPa.

CONCLUSIONS

The main observations from the testing were:

1. The formation material was a well graded soil capable of high strength when compacted at the OWC; however, its strength reduced significantly when the water content increased by 2 – 3 % above the optimum.
2. Granite aggregate, with a thickness of just 150 mm, is a good completion material capable of supporting applied pressures of 1000 kPa for 50,000 cycles with resulting permanent deformations of less than 5 mm.
3. Shale at a depth of 250 mm is a poor quality completion aggregate, but can be made serviceable with a 200 mm top layer of high quality limestone aggregate.
4. The use of Eqn. 1 to estimate the rut depth for un-reinforced, unpaved granite aggregate gave permanent deformations within 2 mm of the experimental measurements.
5. The resilient performance of granitic aggregate on top of a silty sandy formation soil can be modeled using the finite element program SIGMA/W.

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CAPTIONS FOR FIGURES

Figure 1 The laboratory loading apparatus.

Figure 2 Particle size distributions for the materials.

Figure 3 Resilient pressures in Cell D, measured 900 mm above the base of the formation layer, versus number of cycles.

Figure 4 Maximum resilient deflections measured at the surface of the completion layer versus number of cycles.

Figure 5 Maximum permanent deformations measured at the surface of the completion layer versus number of cycles.

Table 1. List of full-scale tests.

Test number	Material	Thickness [a] m	Optimum water content %	Initial water content %	Final dry density Mg m^{-3}
1	Granite	0.25	8.3	6.6	2.1
2	Granite	0.15	8.3	6.3	2.0
3	Shale	0.25	10.3	9.1	1.5
4	Limestone on top of shale [b]	0.20+0.25		3.3	
5	Limestone on top of shale with water addition	0.20+0.25		3.3	
6	Formation layer	1.00	13.5	13.6	1.5

[a] Thickness refers to the completion layer. The formation material had a thickness of 1000 mm for all tests.

[b] 200 mm of a crushed limestone, graded to the specification of a wet mix macadam, was compacted, at a water content of 3.3%, onto a 250 mm layer of the shale at a water content of 9.1%.

Table 2. Summary of BS laboratory results.

Laboratory tests	Limits	Formation	Completion		
			Granite	Shale	Limestone
Natural water content (%)		21.5			
Liquid limit		32.3	43.8	52.4	18.3
Plastic limit (%)		22.6	NP	33.9	NP
Plasticity index (%)	0-6	9.7	0	18.5	0
Specific gravity (Gs)		2.6	2.7	2.5	2.7
Max. dry density (Mg m^{-3})		1.8	2.2	1.6	2.3
Optimum water content (%)		13.5	8.3	10.3	6.1
California bearing ratio (%) [a]	2/30	15.0	115.0	25.2	156.0
Flakiness index (%)	<35.0		26.4	78.0	19.6
MSSV (%)	>75.0		92.9	10.1	93.8
Water absorption value (%)	<2.0		2.9	6.2	0.3
Aggregate crushing value (%)	<35.0		18.5	34.2	21.3
Dry aggregate impact value (%)	<35.0		17.5	31.0	15.2
Wet aggregate impact value (%)			18.6	37.0	
Aggregate abrasion value (%)	<10.0		2	39.2	11.0
Effective size, d_{10}, (mm)		3.9×10^{-3}	1.7×10^{-1}	5.5	0.7
Uniformity coefficient, C_u		44.9	73.5	8.2	15.0
Coefficient of curvature, C_c		0.7	1.2	1.6	3.4

[a] The minimum allowable in situ CBR for a subgrade soil is 2%. Type A sub base material should have a CBR of 30%.

Table 3. Estimation of resilient moduli [a]

Material	Thickness	Applied pressure (kPa)		
		500	750	1000
	m	Calculated resilient moduli, E (MPa)		
Formation	0 – 0.05	29	29	29
	0.05 – 1	37	37	37
Granite	0.25	295	450	550
Granite	0.15	320	460	750
Shale	0.25	60		
Limestone on top of shale [b]	0.20+0.25	225	225	225

[a] The resilient moduli for the soils were estimated by calibrating the SIGMA/W finite element model of the soils and loadings to provide resilient deflections under the centre of the loading pad that were equal to the experimental results.

[b] 0.2 m of a crushed limestone, graded to the specification of a wet mix macadam, was compacted, at a water content of 3.3%, onto a 0.25 m layer of the shale with an assumed constant resilient modulus of 60 MPa for the three applied pressures

Table 4. Experimental and modelled values for resilient deflection at the surface layer of the completion material and pressure at depths of 0.1, 0.3, 0.5 and 0.7 m from the surface of the formation layer.

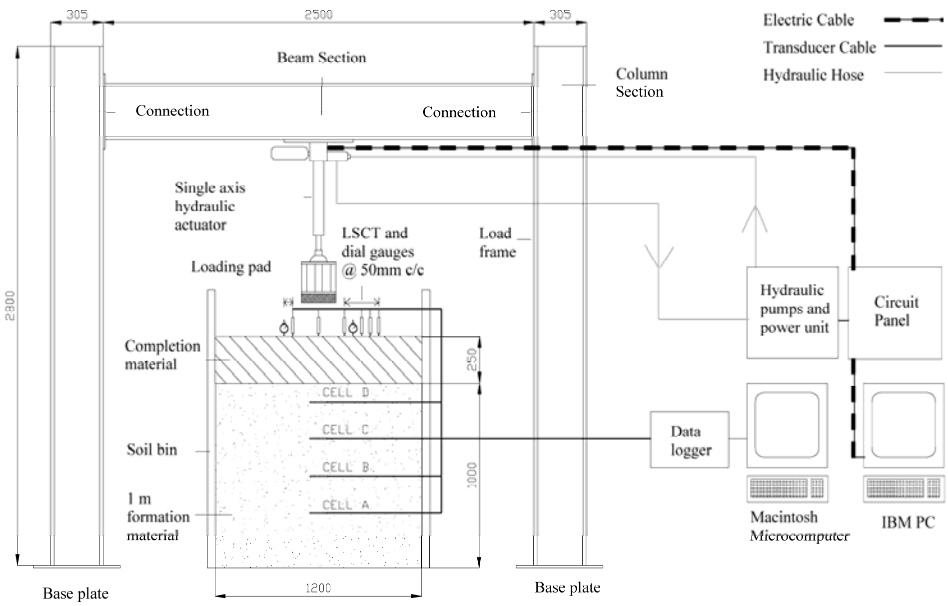
Soil physical responses	Test no.[a]	Applied pressure (kPa)					
		500		750		1000	
		Measured	Modelled	Measured	Modelled	Measured	Modelled
Resilient deflection (mm)	1	0.70	0.70	0.92	0.92	1.16	1.16
Pressure at 0.7m (kPa)	(250 mm	10.5	9.0	15.8	14.0	21.6	19.0
Pressure at 0.5m (kPa)	granite)	12.0	13.0	18.6	21.5	25.0	27.0
Pressure at 0.3m (kPa)		26.0	24.0	40.0	36.0	54.6	47.5
Pressure at 0.1m (kPa)		51.0	51.0	73.5	71.0	97.7	92.0
Modulus of elasticity (MPa)		295		450		550	
Resilient deflection (mm)	2	0.96	0.97	1.33	1.34	1.59	1.60
Pressure at 0.7m (kPa)	(150 mm	13.3	11.0	21.5	17.0	28.3	22.0
Pressure at 0.5m (kPa)	granite)	17.4	16.3	26.0	25.5	33.8	33.0
Pressure at 0.3m (kPa)		39.1	33.0	58.8	50.0	74.4	64.0
Pressure at 0.1m (kPa)		95.9	84.0	124.7	120.0	148.2	146.0
Modulus of elasticity (MPa)		320		460		750	
Resilient deflection (mm)	3	1.73	1.76				
Pressure at 0.7m (kPa)	(250 mm	15.1	10.0				
Pressure at 0.5m (kPa)	shale)	17.2	15.0				
Pressure at 0.3m (kPa)		44.6	30.0				
Pressure at 0.1m (kPa)		104.0	72.0				
Modulus of elasticity (MPa)		60					
Resilient deflection (mm)	4	0.76	0.76	1.2	1.18	1.6	1.6
Pressure at 0.7m (kPa)	(200 mm	9.07	5.7	11.9	9.8	15.5	13.8
Pressure at 0.5m (kPa)	limestone /	9.32	8.0	13.0	13.5	17.2	19.0
Pressure at 0.3m (kPa)	250 mm	17.3	13.5	26.4	22.0	32.5	31.0
Pressure at 0.1m (kPa)	shale)	38.0	24.0	50.3	39.0	57.3	54.0
Modulus of elasticity (MPa)		225		225		225	

[a] 1 - 0.25 m of granite; 2 – 0.15 m of granite; 3 – 0.25 m of shale; 4 – 0.2 m of limestone on 0.25 m shale

Table 5. Comparison of modelled (Eqn. 1) versus measured rut depths.

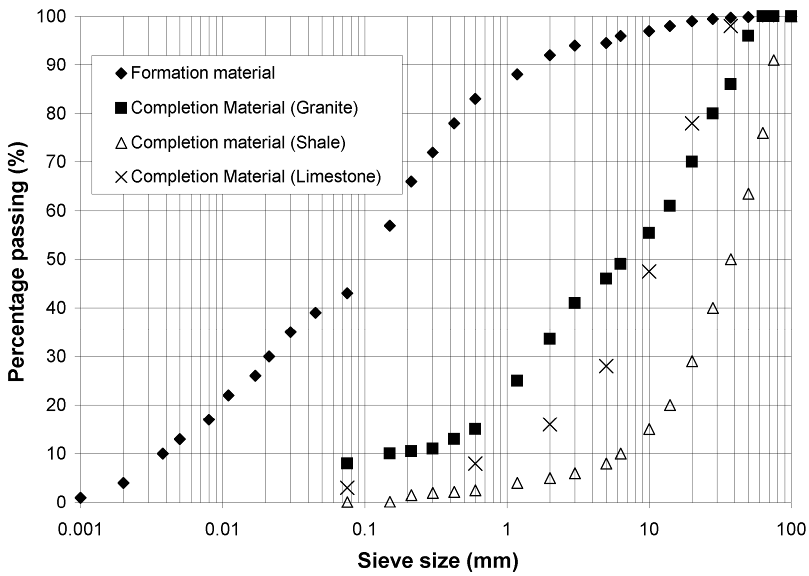
Material	Depth (m)	No of cycles					
		0 – 50,000 at 500 kPa		0 – 50,000 at 500 kPa 50,000 – 100,000 at 750 kPa		0 – 50,000 at 500 kPa 50,000 – 100,000 at 750 kPa 100,000 – 150,000 at 1000 kPa	
		Calculated (mm)	Measured (mm)	Calculated (mm)	Measured (mm)	Calculated (mm)	Measured (mm)
Formation	1						
Granite	0.25	1.1	2.6	2.6	3.6	4.4	4.5
Granite	0.15	2.1	4.1	4.6	5.0	7.4	6.0

Figure 1.



305
2500
305
Electric Cable
Transducer Cable
Hydraulic Hose
Beam Section
Column
Section
Connection
Connection
Single axis
hydraulic
actuator
LSCT and
dial gauges
@ 50mm c/c
Load
frame
Loading pad
Hydraulic
pumps and
power unit
Circuit
Panel
2800
Completion
material
250
CELL D
CELL C
CELL B
CELL A
Soil bin
Data
logger
1000
1 m
formation
material
Macintosh
Microcomputer
IBM PC
Base plate
1200
Base plate

Figure 2



Formation material
Completion Material (Granite)
Completion material (Shale)
Completion Material (Limestone)
Percentage passing (%)
Sieve size (mm)
0
10
20
30
40
50
60
70
80
90
100
0.001
0.01
0.1
1
10
100

Figure 3

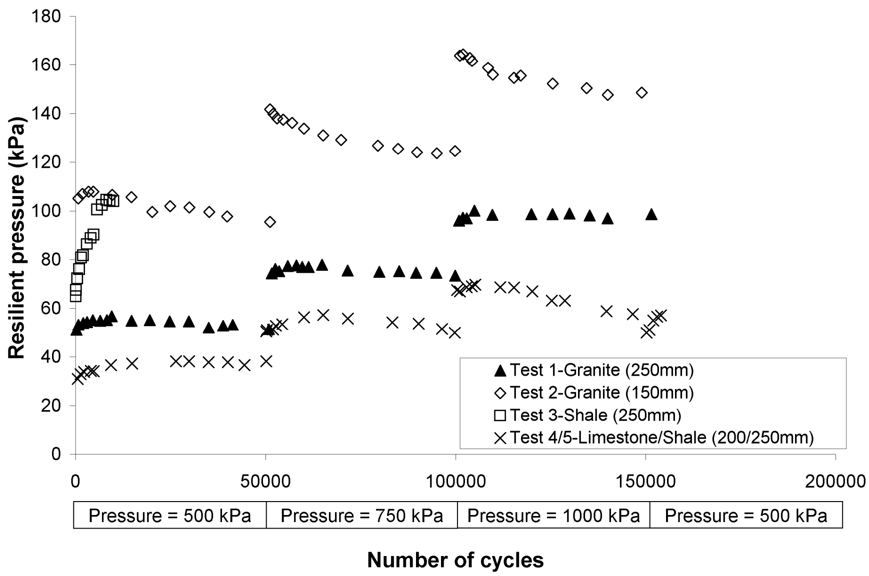
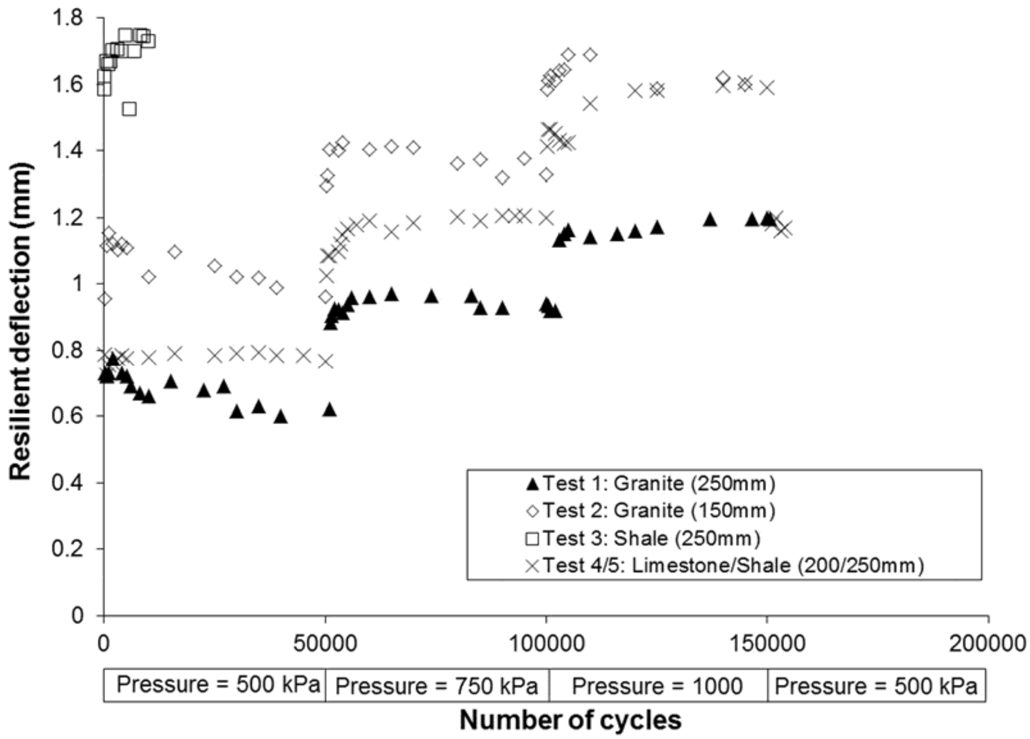
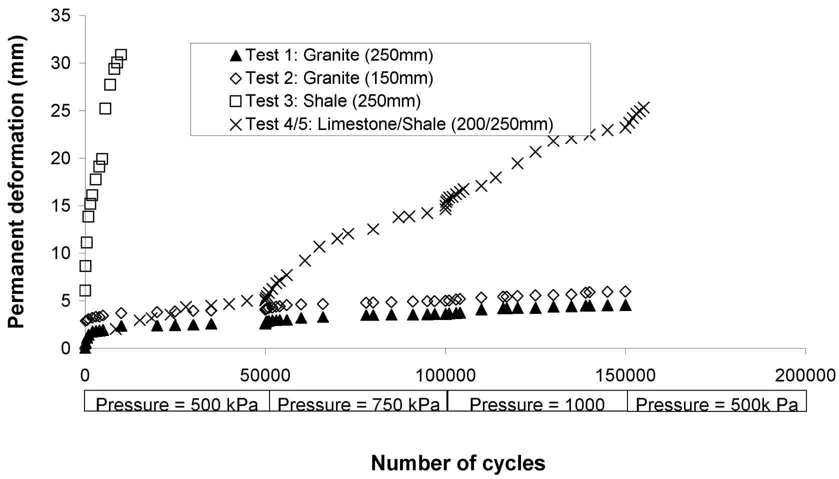

Resilient pressure (kPa)
180
160
140
120
100
80
60
40
20
0
0
50000
100000
150000
200000
Test 1-Granite (250mm)
Test 2-Granite (150mm)
Test 3-Shale (250mm)
Test 4/5-Limestone/Shale (200/250mm)
Pressure = 500 kPa
Pressure = 750 kPa
Pressure = 1000 kPa
Pressure = 500 kPa
Number of cycles

Figure 4



1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
Resilient deflection (mm)
0
50000
100000
150000
200000
Test 1: Granite (250mm)
Test 2: Granite (150mm)
Test 3: Shale (250mm)
Test 4/5: Limestone/Shale (200/250mm)
Pressure = 500 kPa
Pressure = 750 kPa
Pressure = 1000
Pressure = 500 kPa
Number of cycles

Figure 5


35
30
25
20
15
10
5
0
Permanent deformation (mm)
Test 1: Granite (250mm)
Test 2: Granite (150mm)
Test 3: Shale (250mm)
Test 4/5: Limestone/Shale (200/250mm)
0
50000
100000
150000
200000
Pressure = 500 kPa
Pressure = 750 kPa
Pressure = 1000
Pressure = 500k Pa
Number of cycles