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Behaviors of Intermediate Layer "BASE COURSE" used on the Construction of Highway

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ABSTRACT

Pavements are a layered system, each layer is distinguished by different materials as required by traffic and subgrade conditions. A base course is an intermediate layer constructed of high quality stone aggregates: quality based on physical properties such as gradation, hardness, and texture. This thesis presents the results of a comprehensive experimental testing program that was conducted to examine the behavior of unbound granular base materials under cyclic loading and to evaluate the effect of the stress level and moisture content on strain behavior. Three base materials, namely granite, limestone and sandstone, were selected. Three different types of RLT tests were used including: resilient modulus, single-stage, and multi-stage RLT test. The single-stage and multi-stage RLT tests results were analyzed within the framework of the shakedown theory. The results of this study showed that for resilient modulus the materials preformed the following, with the materials listed highest to lowest: limestone, granite and sandstone; while for permanent deformation, the materials were listed highest to lowest: sandstone, limestone and granite. In addition, the results demonstrated that the change in slope (m) of shakedown limits with the degree of saturation was more pronounced at lower stress levels (elastic limit) than that at higher stress levels (plastic limit). Finally, the results showed a significant effect of degree of saturation on the intercept of the shakedown limits at both low and high stress levels. The change in intercept was greater forlimestone than sandstonefor changesindegree of saturation.

I. INTRODUCTION

Pavement structures are built to support loads induced by traffic vehicle loading andto distribute them safely to the subgrade soil. A conventional flexible pavement structure consists of a surface layer of asphalt (AC) and a base course layer of granular materials built on top of a subgrade layer. Pavement design procedures are intended to find the most economical combination of AC and base layers' thickness and material type, taking into account the properties of the subgrade and the traffic to be carried during the service life of the roadway. Pavement materials are required to: (1) spread wheel loads to reduce the load on the soft underlying subgrade (soil) and/or other weaker pavement materials;(2) not fail in shear (i.e. shoving or rutting) with the applications of wheel loads;(3) have a minimal deformation, where most of the deformation occurs in the subgrade.

The two main structural failure mechanisms considered in the design of a flexible pavement structure are permanent deformation (rutting) and fatigue cracking. Rutting is the result of an accumulation of irrecoverable strains in the various pavement layers. For thin to moderately thick pavements, subgrade and granular base layers contribute most to rutting of a pavement. Fatigue cracking has been defined as the phenomenon of fracture under repeated or fluctuating stress having a maximum value generally less than the tensile strength of the material (Ashby and Jones, 1980).

Although base course layer is an intermediary element of the pavement structure, its correct functioning in the road pavement layers is vitally important. The major structural function of a base layer is to distribute the stresses generated by wheel loads acting on the wearing surface so that the stresses transmitted to the subgrade will not be sufficientlygreat to result in excessive deformation or displacement of that foundation layer. Also, while transferring these stresses, the base layer must not undergo excessive permanent deformation and withstand shoving.

Material Characteristics is a principal factor entered into flexible pavement design methods to determine layer thickness and type. Unbound base course materials are considered for pavement design primarily on their physical properties with exception of resilient modulus, which is a performance parameter expressing stiffness that replaced the structural support value in 1986. Although an improvement, the resilient modulus alone does not duly characterize the functionality of the unbound granular material layer. As stated earlier, in addition to transfer of loading to the subgrade, the material must be capable of safely handling stresses without excessive deformation. Leading to further improve on the characterization of the granular material, the permanent deformation component must be accounted for and included with the resilient modulus to fully evaluate the engineering behavior of the granular material to ensure proper functionality of the base course layer.

1.1 Problem Statement

A principal component included in the design of flexural pavements is thecharacterization of those materials that make up the pavement layers. Aiding in the development of the M-E design guide, areas identified by Strategic Highway Research Program (SHRP) and other M-E implementation projects such as Federal Highway Admistration (FHWA) and National Cooperative Highway Research Program (NCHRP) require further material characterization research include:

- Resilient Modulus for granular materials
- Permanent Deformation properties for granular materials

Overall performance of a pavement structure depends highly on the proper characterization of material properties.Currently, Granular Materials are characterized on the basis of physical properties such as gradation, plasticity, hardness, durability, and on the basis static shear strength tests. These properties are determined either empirically (correlations) or from testing procedures that do not properly consider the relevance to the cyclic loading behavior of the material. These physical properties or strength characteristics from static load testing are insufficient to characterize the dynamic response of materials within a pavement layer. For this reason, to simulate accurate field conditions, the UGMs must undergo cyclic loading to characterize the dynamic response behavior. The observed distresses in the field (rutting, flexural cracking) are a direct result of the dynamic traffic loading, thus characterizing materials behavior with cyclic loading will aid as a predictor for field performance.

II. Methodology

This chapter includes a description of the research methodology used in this study. The chapter outlines detailed information about the physical properties and experimental testing.

2.1 Experimental Testing Program

An experimental testing program was performed on three types of unbound granular materials used in construction of base course layers. The tested materials included: limestone, sandstone, and granite materials. All materials were selected from 1.5 inch sieve crushed run materials provided by the appropriate quarries. Different laboratory tests were first conducted to screen the physical properties that are typically used in the selection and evaluation of base course material. The performed test included sieve analysis (ASTM C136-06), Standard Proctor (ASTM D 792), specific gravity and absorption, and coarse aggregate angularity (ASTM D 5821). Materials were sampled in accordance with ASTM C702 .In addition, Micro-Deval (ASTM D 6928) test were conducted to examine particle degradation of the considered material.

Tri-axial tests were conducted used in this study to characterize the shear strength properties of base course granular materials in their field construction conditions and examine their response under cyclic loading. To do this, two types of tri-axial test were employed: static tri-axial test compression test (SCT) and repeated load tri-axial testing (RLT). The triaxial tests conducted in this study are described below.

2.2 Testing Setup of Triaxial Tests

All triaxial tests were performed using the Material Testing System (MTS) 810 machine (Figure 3.1) with a closed loop and a servo hydraulic loading system. Theapplied load was measured using a load cell installed inside the triaxial cell. This type of set up reduces the equipment compliance errs as well as the alignment errors. The capacity of the load cell used was ± 22.25 kN. The axial displacement measurements were made using two Linearly Variable Differential Transducers (LVDT) placed between the top platen and base of the cell to reduce the amount of extraneous axial deformation measured compared to external LVDTs. Air was used as the confining fluid to the specimens. Figure 2.1 illustrates the testing setup.

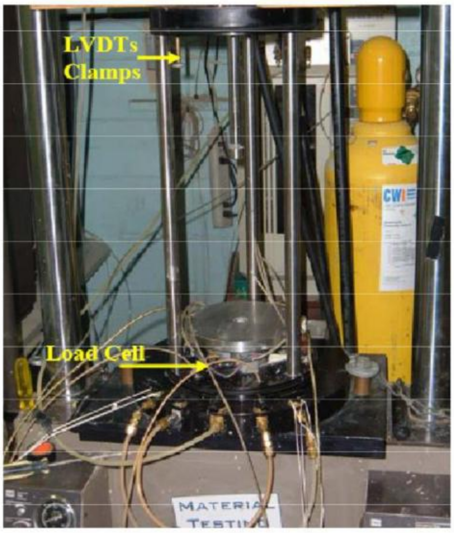


Figure 2. 1 Tri-axial Testing Machine

2.3 Sample Preparation

AASHTO-T307 recommends that a split mold be used for compaction of granular materials. Therefore, all samples were prepared using a split mold with an inner diameter of 150 mm and a height of 350 mm. The material was first oven dried at a pre-specified temperature and then mixed with water at the specified moisture content. The achieved water contents were within ±0.5 percent of the target value. For single-stage RLT test and static shear strength test, the material was placed within the split mold and compacted using a vibratory compaction device to achieve the prescribed dry density determined from the standard Proctor test. For the multi-stage samples utilized for shakedown the target moisture content was varied on the wet and dry side of optimum moisture content and then vibratory compacted to maximum to max dry density as determined from standard proctor test. To achieve a uniform compaction throughout the thickness, samples were compacted in six-50 mm layers. Each layer was compacted until the required density was obtained; this was done by measuring the distance from the top of the mold to the top of the compacted layer. The smooth surface on top of the layer was lightly scratched to achieve good bonding with the next layer. The achieved dry densities of the prepared samples were within ±1 percent of the target value. Samples were enclosed in two latex membranes with a thickness of 0.3 mm. Figure 2.2 illustrates the preparation procedure of limestone samples.

2.3.1 Static Triaxial Compression Test

As many pavement structures do not fail by shear, the RLT triaxial tests are considered more representative of actual performance in the road. Nevertheless, the monotonic triaxial compression tests provide valuable parameters that can be used to

Figure 2.2 Preparation of Testing Limestone Samples.

evaluate strength and stiffness of pavement materials. Furthermore, it is commonly thought that safe stress states for a pavement material are related to their ultimate shear strength.

Drained triaxial compression tests were first performed to obtain the shear strength properties of the different materials considered. The triaxial compression tests were performed at three different confining pressures: 2, 7 and 10 psi (14, 48, and 69 kParespectively). The strain rate used in those tests was less than ten percent strain per hour to ensure that no excess pore water pressure developed during testing. Two response parameters were recorded for each static triaxial test: ultimate shear strength (USS) and residual shear strength (RSS).

III. Analysis of Results

This chapter presents the results of the experimental testing program that was conducted to evaluate physical properties and to characterize the behavior of the course materials under static as well as cyclic loading.

3.1 Physical Properties Test Results

Figure 3.1 shows the gradation obtained from the sieve analysis and hydrometer tests for the considered materials, while Table 3.1 present a summary of the physical properties test conducted on those materials. It is noted that all materials had the same maximum nominal aggregate size of 25 mm. Furthermore, they were classified as A-1-b and GW/sand according to the American Association of State Highway and Transportation (AASHTO) classification system, and the Unified Soil ClassificationSystem (USCS), respectively. However, there were some differences between the materials in the percent of fines passing sieve size 0.075 mm, such that the granite had lowest percentage of about 5, while the crushed lime stone had the highest percentage of 13.5. The gradation of the three materials considered was further evaluated using the power-law method suggested by Ruth et al. (2002) . The power-law shown in Equation 4.1 characterizes the slope and the intercept constants of the coarse and fine aggregate portions of the aggregate gradations. The divider sieve between the coarse and fine aggregate used in the power law analysis was chosen to be 4.75 mm (No.4) sieve. Table 3.1 presents the power law gradation parameters for all the aggregate structures in this study. It is noted that the granite had the highest nCa coefficient, followed by thesandstone, then the crushed limestone. This indicates that the granite had the coarsest gradation followed by the sandstone. However, the nfa value of the all materials was similar. It is noted that a higher n_{fa} value indicates that the fine portion of an aggregate gradation is finer.

P = a (d) and P = a (d)

Where,P_{CA} and P_{FA} = percent by weight passing a given sieve that has an opening of width d.

a_{CA} = intercept constant for the coarse aggregate

n_{CA} = slope (exponent) constant for the coarse

d = sieve opening width, mm

a_{FA} = intercept constant for the fine aggregate

n_{FA} = slope (exponent) for the fine aggregates

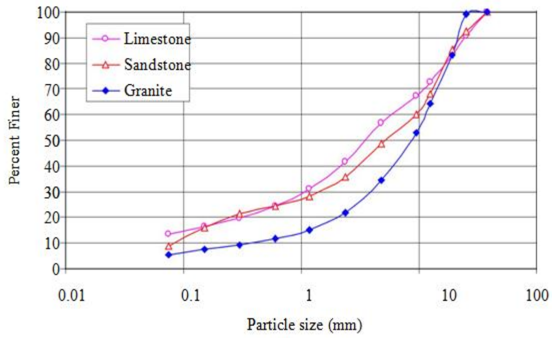


Figure 3. 1 Particle Size Distribution of Tested Aggregates

Table 3.1 shows that the considered aggregate had absorption values ranging from 0.9 to 2.1 percent. Furthermore, the table shows that the considered aggregates had a low percentage of loss in the Micro-Deval test; however the granite had the lowest value of 5%. Many studies suggested the low percentage of loss indicates the ability of the material to resist degradation during construction and under traffic loading (Hossain et al. 2008) Therefore, all materials are considered to be durable and resist degradation.

Table 4.1 also shows the maximum dry unit weight and optimum moisture content obtained in the Standard Proctor test. It is noted that there are some differences in the values obtained from the Standard Proctor test between the three aggregate materials; however, the current specification does not have any limitations on those values, but uses them as reference to which materials in the field are to be compacted. Table 3.1 shows that the degree of saturation for sandstone aggregate at the optimum conditions determined in the Standard Proctor test was at least ten percent higher than those of the other materials.

Table 3.1 Physical Properties Results

Property	Limestone	Sandstone	Granite
G_s	2.708	2.642	2.671
Absorption,%	1.7	2.1	0.9
Micro-Deval, Loss%	13.0	11.5	5.5
Max $_{dry}$ in Standard Proctor (lb/ft^3)	142.0	136.2	132.0
Optimum Moisture Content,%	6.5	7.1	6
Degree of saturation,%	80.7	88	76.3
AASHTO classification	A-1-b	A-1-b	A-1-b
USCS classification	GW/sand	GW/sand	GW/sand
Coarse aggregate angularity, (%)	100	100	100
Power Law Analysis of Gradation			
a_{Ca}	36.0	27.1	15.313
n_{Ca}	0.28	0.37	0.55
a_{Fa}	29.9	28.0	14.792
n_{Fa}	0.32	0.36	0.38
Shear Strength Properties			
Peak friction angle	52.2	51.2	57.7
Cohesion- ultimate shear strength (psi)	3.65	3	0
Residual strength friction angle	47.5	46.5	49
Cohesion- residual shear strength (psi)	2	2	0

3.2 Static Triaxial Tests Results

Drained triaxial compression tests were on granite, limestone and sandstone samples. The achieved dry unit weight and moisture content of the tested samples were close to those specified in the field for construction of UGM base course layers in Louisiana, which specifies that the materials should be mixed at the optimum moisture content and compacted to 95% of the maximum dry unit weight as determined in standard Proctor test. Figure 4.1 through Figure 4.3 present the average stress-strain curves obtained from the drained triaxial compression tests conducted on three samples for each material. The figures show that at the tested confining pressures and dry unit weight the samples behaves as a loose granular material, such that they exhibit an increase in shear strength with increasing strain, which is referred to as strain hardening, and eventually reached peaked strain level ranging from 2- 4%.

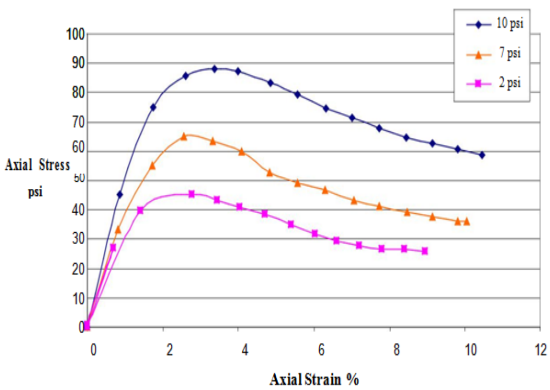


Figure 3. 2 Stress-Strain Curves for Granite Static Compression Test

3.3.1 Single-Stage RLT Test Results-Resilient Strain

Figure 3.9a presents the average vertical resilient strain curves obtained from the results of the single stage RLT test that were conducted on the three types of base course materials investigated in this study. The resilient strain had a similar trend in all materials, such that it initially increased, then decreased as the number of load cycles increased until reaching an asymptote at about 6,000 load cycles, and hence reaching a steady resilient response. The reason for this behavior is that during the primary post compaction stage, the sample accumulates more deviatoric strain in the horizontal direction (perpendicular to the direction on which the cyclic load is applied), causing the Poisson's ratio to decrease slightly; this results in an increase in the sample stiffness and hence a decrease in the resilient strain. It should be noted that the number of cycles needed for the sample to reach a steady resilient response increases as the imposed deviatoric stress is increased.

Figure 3.9a shows that the sandstone material had a much higher resilient strain than the other two materials, and hence a much smaller resilient modulus. Furthermore, the crushed limestone had lower resilient strain than the granite. These results are also illustrated in Figure 4.9b, which presents the resilient modulus value measured after 10,000 cycles in the single-stage RLT tests and those predicted using the universal resilient modulus model (Equation 2.13) based on $k_{1\text{-}3}$ coefficient obtained from the M_rRLT test. This figure shows that the predicted values were very similar to those measured, indicating the reliability of this model prediction.

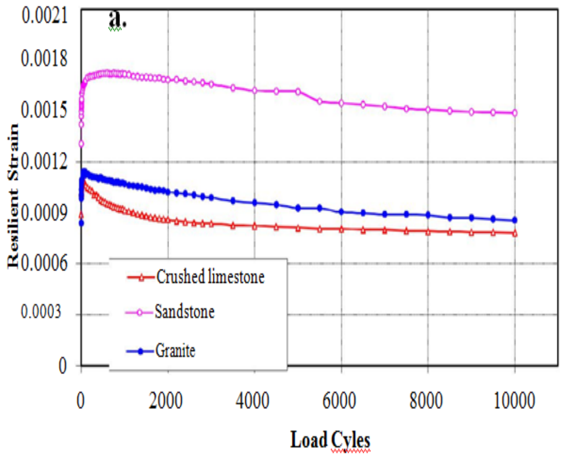
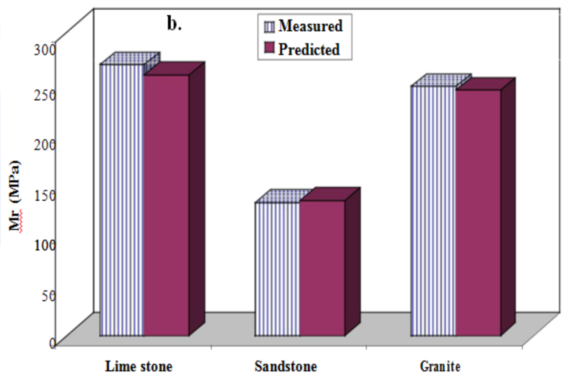


Figure 3. 9 Results of Single-Stage RLT Test a) Resilient Strain Variation of Load Cycles b) Measured and Predicted Resilient Modulus Values



3.3.2 Single-Stage RLT Test Result-Permanent Strain

Figures 3.10 presents the average vertical permanent strain curves obtained from the results of the single-stage RLT test for the three materials considered in this study. Averages were calculated from triplicate samples; coefficients of variation are equal to or less than 15%. For the three materials, the primary and secondary stages were only experienced during this type of RLT test. The sandstone experienced by far the largest permanent strain. Furthermore, the crushed limestone had accumulated a greater permanent strain than the granite. It is noted that the three materials had similar behavior during the initial load cycles, hence, during the primary post-compaction stage; however, the differences between the materials in the permanent strain behavior were detected during the secondary stage. This indicates that differences in permanent strain for the considered materials did not mainly result from discrepancies in the materials' initial

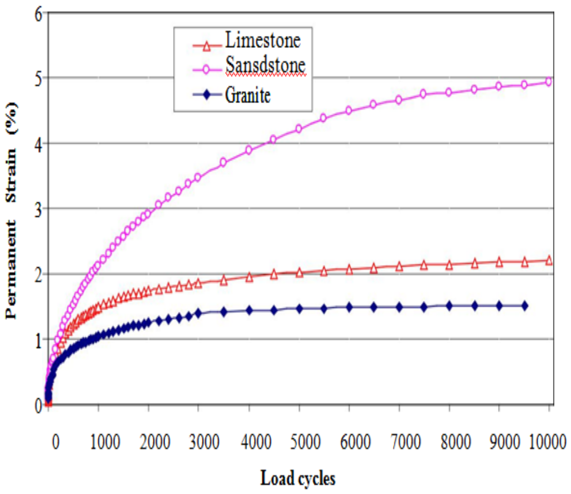


Figure 3. 10 Vertical Permanent Strain Variations with Number of Cycles

IV. CONCLUSION

This paper documented the results of a laboratory testing program that was conductedto characterize the behavior of unbound granular base materials under different loading conditions, and examine the effect of different physical properties on this behavior. Three different types of granular base materials were investigated in this study, namely limestone, sandstone, and granite. Physical properties and static and repeated load triaxial tests were performed on the considered materials. Three different types of RLT tests were used in this study including resilient modulus, single-stage, and multi-stage RLT tests. The results of the single-stage and multi-stage were analyzed within the framework of the shakedown theory. Based on the results of this paper, the following conclusions can be drawn. From the results, it is evident that degree of saturation affects the shakedown behavior of granular material. The degree of saturation is inversely related to the intercept of the shakedown limits allowing shakedown beyond the static failure line.

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