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Research Article

Laboratory Performance Evaluation of Physical and Mechanical Road Base Construction Properties

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Abstract: Road-bases are quarry materials, which are used in the construction of road pavements. They comprised a combination of coarse and fine crushed materials, which when placed and compacted at the correct moisture, form a stiff layer. For soil aggregates materials to be used correctly in road construction, it is necessary to know their properties. In addition, road-base material characterizations are needed to design an adequate pavement structure for expected traffics load. This study aims to evaluate the effects of the physical, chemical and mechanical behaviors of soil aggregate as granular road base materials in order to provide rudimentary understanding of roads based materials on mechanistic-empirical pavement design and analysis using experimental laboratory tests. These tests were performed in accordance with American Society for Testing and Materials (ASTM) and British Standards (BS) on fine and coarse soil aggregates along with base aggregate used in pavements. The results show that the optimum moisture content, maximum dry density and plasticity index have significant influence on the behavior of these materials.

Keywords: Maximum dry density, optimum moisture content, pavements, plasticity index, road-bases, soil aggregates

INTRODUCTION

Roads are laid in several layers, namely, sub-grade, sub-base, base and surface layers, which, together, constitute the pavement. Pavements constructed using good quality building materials distribute the forces induced by traffic in such a way that overloading and deformation of the foundation is prevented. Flexible pavements with unbound granular materials as base course layer and thin bituminous surfacings are widely used. The primary function of the base course layer in a flexible pavement is transferring and reducing the stresses and strains induced by wheel loading to the layers underneath without damaging the in-situ soils (subgrade). Understanding the actual behavior of both materials under traffic loads is crucial for advancing pavement analysis and design and it is more significant for the development of novel pavement engineering techniques that could replace the conventional methods (Vegas *et al*., 2011; Ismail *et al*., 2014; Shojaei Baghini *et al*., 2014). The road-base (also referred to as the base course) is the main layer that provides additional strength and load-bearing capacity to the road. This layer commonly consists of crushed and graded materials or selected soils from natural sources, which possess certain characteristics that are known to improve the quality of the road (Johannessen, 2008; FHWA, 1992; Shojaei Baghini *et al*., 2013a). Laboratory testing is a fundamental element of a geotechnical investigation. The primary objectives of these tests are to refine the visual observations through repeatable procedures, conduct field tests as part of the subsurface field exploration program and determine the behavior of soil or rock under the imposed conditions. An ideal laboratory program can provide sufficient data to complete an economical design without incurring excessive tests or costs. Depending on the complications involved in the project, testing may range from a simple soil classification testing to a complex strength and deformation testing (WSDOT, 2013; Shojaei Baghini *et al*., 2013b; AASHTO, 2004; Head, 1992). The purpose of this study is to identify, either by reference or explicitly herein, an appropriate method to assess soil and rock properties and an approach to use that data to establish the final soil and rock parameters that are to be considered in a geotechnical design. These chosen parameters should be based on the results from the field investigation, field-testing and laboratory testing, used separately or in combination.

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Table 1: Grading requirements for final mixtures (ASTM, 2004)

	Design range (Mass percentages passing)		Job mix tolerances	
Sieve size (Square openings)	Bases	Sub-bases	Bases	Sub-bases
50 mm (2 in.)	100	100	-2	-3
37.5 mm (11⁄2 in.)	95 to 100	90 to 100	±5	+5
19.0 mm (3⁄4 in.)	70 to 92	NA	±8	NA
9.5 mm (3⁄8 in.)	50 to 70	NA	±8	NA
4.75 mm (No. 4)	35 to 55	30 to 60	±8	±10
600 µm (No. 30)	12 to 25	NA	±5	NA
75 µm (No. 200)	0 to 8	0 to 12	±3	±5

STANDARD REQUIREMENTS FOR GRADED SOIL AGGREGATE USE IN BASES OR HIGHWAYS

Comprises quality-controlled graded aggregates may be expected to provide appropriate stability and load support for use as highway or airport bases or sub-bases. This requirement delineates the aggregate size variety and ranges in mechanical analyses for standard sizes of coarse aggregate and screenings for use in the construction and maintenance of various types of highways. The gradation of the final composite mixture shall conform to an approved job mix formula, within the design range prescribed in Table 1 according to ASTM D 448, ASTM D 1241 and ASTM D 2940, subject to the appropriate tolerances (ASTM, 2004).

This classification defines aggregate size designations and ranges in mechanical analyses for standard sizes of coarse aggregate and screenings for use in the construction and maintenance of various types of highways. Coarse aggregate retained on the 4.75-mm (No. 4) sieve shall consist of durable particles of crushed stone, gravel, or slag capable of withstanding the effects of handling, spreading and compacting without degradation productive of deleterious fines. Coarse aggregate shall have a percentage of wear, by the Los Angeles Abrasion test, of not more than 50. Fine aggregate passing the 4.75-mm (No. 4) sieve shall normally consist of fines from the operation of crushing the coarse aggregate. The fraction of the final mixture that passes the 75-µm (No. 200) sieve shall not exceed 60 % of the fraction passing the 600-µm (No. 30) sieve. The fraction passing the 425-µm (No. 40) sieve shall have a liquid limit no greater than 25 and shall have a plasticity index no greater than 4. The sand equivalent value of the fine aggregate shall be no lower than 35. The fraction passing the No. 200 sieve shall not be greater than two thirds of the fraction passing the No. 40 (425-µm) sieve.

MATERIALS

Soil aggregate: Crushed granite aggregates taken from Kajang Rocks Quarry (Malaysia) were used in this research as granular base layer material. ASTM C 136 test method was used to determine the grading of materials proposed for use as aggregates. The samples were dried to constant mass at a temperature of 110°C. After selecting sieves with suitable openings, soil aggregates were sieved for a sufficient period and the percentages passing and percentages retained were calculated. The results were used to determine compliance of the particle size distribution with applicable specification requirements and to provide necessary data for control of the production of various aggregate products and mixtures containing aggregates. Figure 1 illustrates the grading curves of soil aggregates within the specification limits for highways or/and airports according to the American Society for Testing and Materials (ASTM) standards.

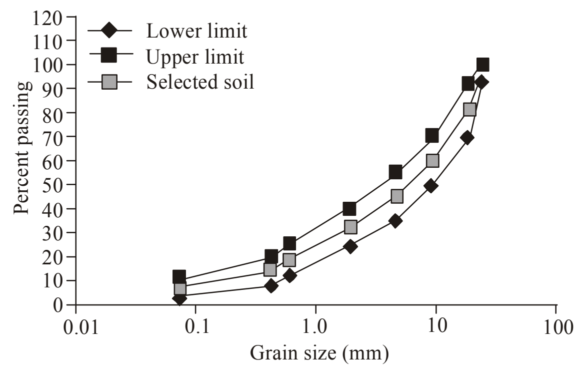

Fig. 1: Grading curves of soil aggregate

EXPERIMENTAL WORKS AND RESULTS

Moisture-density relations of soil aggregate mixture: Soil utilized as an engineering fill (embankments, foundation pads, road bases) is compacted densely in order to obtain satisfactory engineering properties such as shear strength, compressibility, or permeability. Laboratory compaction tests provide the basis for determining the percentage of compaction and water content needed to achieve the desired properties and for controlling the construction to ensure that the required compaction and water content is achieved. A laboratory compaction method, in accordance to ASTM D 698 Method C, was used to determine the relationship between the water content and dry unit weight of soils compacted in a 152.4-mm diameter mold with a 24.4-N rammer dropped from a height of 305 mm producing a compressive force of 600 KN- m/m^3. A soil at a

Fig. 2: Mechanical compactor

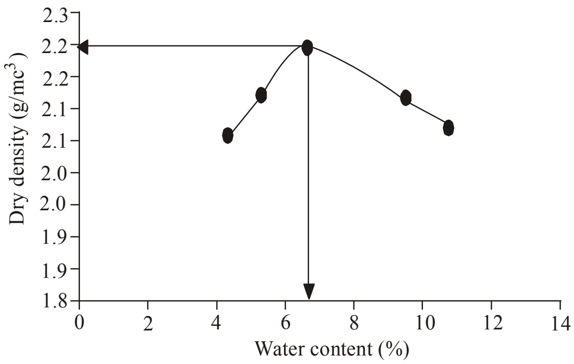

Fig. 3: Moisture-density relations

Fig. 4: pH of soil aggregate

selected water content is placed in three layers into a mold of given dimensions, with each layer compacted by 56 blows with the rammer (Fig. 2).

The procedure is repeated for a sufficient number of water contents to establish a relationship between the dry unit weight and the water content for the soil. This data, when plotted, represents a curvilinear relationship known as the compaction curve. The values of Optimum Water Content (OWC) and standard Maximum Dry Density (MDD) were determined from the compaction curve as shown in Fig. 3.

pH test: The pH is an important soil variable in deciding the solubility of soil minerals, the mobility of ions in the soil and assessing the viability of the soil-plant environment. The ASTM D 4972 test method was used to measure the pH of the soil aggregate. This measurement determines the degree of acidity or alkalinity in soil materials suspended in water. All water used for this test method must be ASTM Type III (defined by specification ASTM D 1193 and prepared by distillation, ion exchange, reverse osmosis, or a combination thereof). Ten grams of air-dried soil passing through a No. 10 sieve (2 mm sieve mesh openings) was placed into a glass container and approximately 10 mL of water was added, mixed thoroughly and allowed to stand for 1 h. After calibrating the pH meter, the pH of the soil was read to the first decimal place. Figure 4 shows the pH meter used in this research. The result of pH test is shown in Table 2.

Classification of soils: The ASTM D 2487 test method classifies soils from any geographic location into categories representing the results of prescribed

Table 2: Properties of soil aggregates

Soil properties	Requirements	Test result	Test method
Water content (%)	NA	6.621	ASTM D 698
Unit weight (g/cm^3)	NA	2.19	ASTM D 698
pH	5.3-Min	8.26	ASTM D 4972
Unified classification	NA	GP-GM	ASTM D 2487
AASHTO Classification	NA	A-1-a	ASTM D 3282/AASHTO M 145
Liquid limit (%)	25-Max	21.4	ASTM D 4318
Plastic limit (%)	29-Max	19.6	ASTM D 4318
Plastic index (%)	4-Max	1.8	ASTM D 4318
Coefficient of curvature (Cc)	NA	2.39	ASTM D 2487
Coefficient of uniformity (Cu)	NA	71.5	ASTM D 2487
Group index	NA	0	ASTM D 3282
Specific gravity (OD)	NA	2.659	ASTM C 127/C 128
Specific gravity (SSD)	NA	2.686	ASTM C 127/C 128
Apparent specific gravity	NA	2.731	ASTM C 127/C 128
Water absorption, %	2	0.973	ASTM C 127/C 128
Linear shrinkage (%)	3	1.5	BS 1377: Part 2
Elongation index (%)	25-Max	13.03	BS 812: Section 105.2
Flakiness index (%)	25-Max	7.68	BS 812: Section 105.1
Average least dimension (mm)	NA	5.5	BS 812: Section 105.1
Sand equivalent (%)	Min-35	84	ASTM D 2419
Los angeles abrasion (%)	50-Max	17.5	ASTM C131
UCS (MPa)	NA	0.25	ASTM D 2166/D 1633
CBR (%)	80-Min	101.32	ASTM D 1883

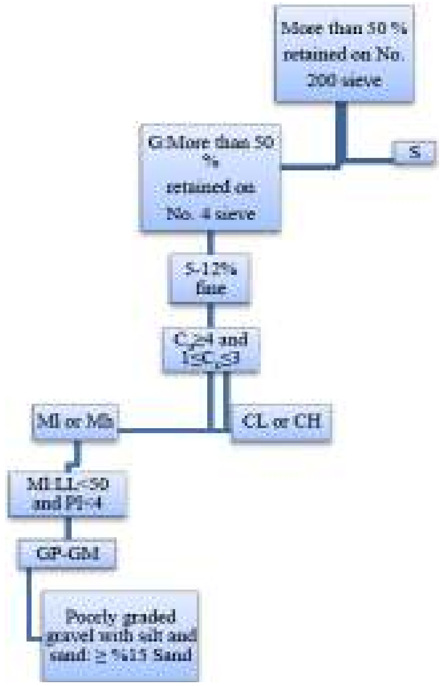

Fig. 5: Classification of soils

First and/or second letters

Letter	Definition
G	gravel
S	sand
M	silt
C	clay
O	organic

Second letter

Letter	Definition
P	poorly graded (uniform particle sizes)
W	well-graded (diversified particle sizes)
H	high plasticity
L	low plasticity

Fig. 6: Definition of symbols

Fig. 7: Preparation of LL specimen

Fig. 8: Casagrande's LL apparatus

laboratory tests to determine the particle-size characteristics, the liquid limit and the plasticity index. In addition, ASTM D 2488 describes a soil to aid in the evaluation of its significant properties for engineering

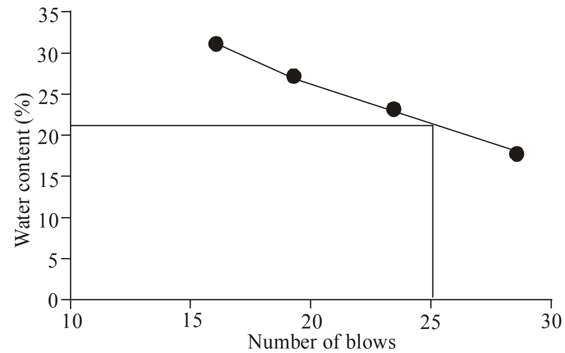

Fig. 9: Calculation of liquid limit

use. Based on the results of visual observations and prescribed laboratory tests, soil is catalogued according to the basic soil groups, assigned a group symbol (s) and name and thereby classified. Figure 5 shows the process of finding the classification of the soil aggregate used in this research. In addition, Fig. 6 indicates the definition of symbol used in this classification. The result of ASTM classification is shown in Table 2.

The AASHTO Soil Classification System was developed by the American Association of State Highway and Transportation Officials and is used as a guide for the classification of soils and soil-aggregate mixtures for highway construction. The AASHTO system uses both grain-size distribution and Atterberg limits data to assign a group classification and a group index to the soil. The group classification ranges from A-1 (best soil) to A-8 (worst soil). Group index values near 0 indicate good soil, while values of 20 or more indicate very poor soil. However, a soil that may be "good" for use as a highway subgrade might be "very poor" for other purposes and vice versa. The result of AASHTO classification is shown in Table 2.

Liquid limit, plastic limit and plasticity index: The aforementioned test methods are used as an integral part of several engineering classification systems to characterize the fine-grained fractions of soils and to specify the fine-grained fraction of construction materials. The liquid limit (LL), plastic limit and plasticity index of soils are also used extensively, either individually or together, with other soil properties to correlate with engineering behaviours such as compressibility, hydraulic conductivity (permeability), compatibility, shrink-swell and shear strength. To achieve LL, a representative portion from the total sample was obtained to provide 150 to 200 g of material passing the 425-µm (No. 40) sieve. It was mixed thoroughly with distilled or demineralized water on a glass plate or mixing dish using a spatula. The water content of the material was adjusted to bring the mixture to a consistency that would require approximately 25 to 35 blows of the LL device to close the groove. The specimen was processed to remove any

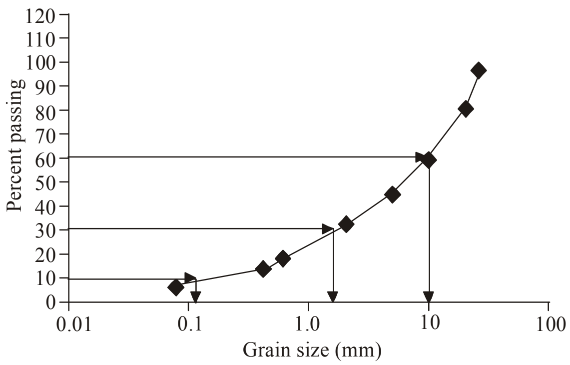

Fig. 10: Distribution curve of soil aggregate

Table 3: The value of D_{10}, D_{30} and D_{60}

D_{10}	D_{30}	D_{60}
0.13	1.7	9.3
C_u		9.3/0.13 = 71.5
C_c		$1.7^2/(9.3*0.13) = 2.39$

material retained on the 425-µm (No. 40) sieve and the LL was determined by performing trials in which a portion of the specimen was spread in a brass cup. It was divided into two by a grooving tool and then allowed to flow together because of the shocks caused by repeatedly dropping the cup in a standard mechanical device. The multipoint liquid limit requires three or more trials over a range of water contents to be performed and the data from the trials plotted or calculated to make a relationship from which the liquid limit is determined. Figure 7 to 9 show the process of determining of LL.

The Plastic Limit (PL) was determined by alternately pressing together and rolling a 20-g or more portion of soil into a 3.2-mm (1⁄8-in.) diameter thread until its water content was reduced to a point at which the thread crumbles and can no longer be pressed together and re-rolled. The water content of the soil at this point is reported as the plastic limit. The Plasticity Index (PI) was calculated as the difference between the LL and PL. The results of LL, PL and PI are shown in Table 2.

Coefficient of curvature and coefficient of uniformity: The shape of the gradation curve can be characterized by a pair of "shape" parameters called the coefficient of uniformity, Cu and the coefficient of curvature, Cc, to which numerical values may be assigned. If Cu is between 4 and 6, or larger, it is considered a well-graded soil and if the Cu is less than 4, the soil is considered poorly or uniformly graded. Uniformly graded implies that the soil consists of identical size particles. For the soil to be well graded, the value of Cu has to be greater than 4 and Cc should range from 1 to 3. Therefore, the higher the value of Cu, the larger the range of particle sizes in the soil. If the Cu value is high it indicates that the soil mass consists of different ranges of particle sizes. By using the two "shape" parameters, Cu and Cc, the uniformity of the coarse-grained soil (gravel and sand) can now be classified as well-graded (non-uniform), poorly graded (uniform), or gap graded (uniform or non-uniform). The C_u and C_c expressed in Eq. (1) and (2) respectively:

$$C_u = \frac{D_{60}}{D_{10}} \tag{1}$$

$$C_c = \frac{D_{30}^2}{D_{60} \times D_{10}} \tag{2}$$

where, D_{60}, D_{30} and D_{10} are the particle sizes corresponding to 60, 30 and 10% finer on the cumulative particle-size distribution curve, respectively as shown in Fig. 10.

Table 3 shows the value of $D_{10,}$ D_{30} and D_{60} respectively. The results of C_u and C_c are shown in Table 2.

Group index: Evaluation of soils within each group is made by means of a Group Index (GI) which is a value calculated from an empirical formula. The ASTM D 3282 classifies soils from any geographic location into groups based on the results of prescribed laboratory tests. These tests determine the particle-size characteristics, liquid limit and plasticity index. The assignment of a group symbol and group index can be used to aid in the evaluation of the significant properties of the soil for highway and airfield purposes. The various groupings of this classification system correlate in a general way with the engineering behavior of soils. In addition, the engineering behavior of a soil varies inversely with its group index. Therefore, this standard provides a useful first step in any field or laboratory investigation for geotechnical engineering purposes. The classifications obtained from section (Classification of Soils) may be modified by the addition of a group-index value. Group-index values should always be shown in parentheses after the group symbol and then the group index is calculated from the following empirical Eq. (3):

$$GI = (F-35)\left[0.2+0.005(LL-40)\right]+0.01(F-15)(PI-10) \tag{3}$$

where,
GI = Group Index
F = Percentage passing No. 200 (75-µm)
LL = Liquid limit and PI is plasticity index

The result of GI is shown in Table 2.

Specific gravity and absorption: ASTM C 127 and ASTM C 128 test methods correspond to the determination of Specific Gravity (SG) and the absorption of coarse and fine aggregates. The SG may

Fig. 11: SSD of coarse aggregate

Fig. 12: SG test set of coarse aggeregate

be expressed as bulk SG, bulk SG saturated-surface-dry (SSD), or apparent SG. Bulk SG is the characteristic generally used for calculation of the volume occupied by the aggregate in various mixtures containing the aggregate, including Portland cement concrete, bituminous concrete and other mixtures that are proportioned or analyzed on an absolute volume basis. Bulk SG is also used in the computation of voids in the aggregate. On the other hand, apparent SG pertains to the relative density of the solid material making up the constituent particles while not including the pore space within the particles, which is accessible to water. Absorption values are used to calculate the change in the weight of an aggregate due to water absorbed in the pore spaces within the constituent particles, compared to the dry condition, when the aggregate is considered to have been in contact with water long enough to satisfy most of the absorption potential. For coarse aggregates, a sample of the aggregate was immersed in water for approximately 24 h to fill the pores completely. It was then removed from water and the particles were dried to evaporate the water molecules on their surface before weighing (Fig. 11). Subsequently, the sample was weighed while submerged in water (Fig. 12). Finally, the sample was oven-dried and weighed a third time. Using the weights obtained and formula in this test method, the three types of SG and absorption were calculated. The bulk SG at 23°C is calculated according to Eq. (4 to 10):

$$G_a = \frac{A}{(A-C)} \quad (4)$$

$$G_{od} = \frac{A}{(B-C)} \quad (5)$$

$$G_{ssd} = \frac{B}{(B-C)} \quad (6)$$

$$G_a = \frac{A}{(A-C)} \quad (7)$$

$$G_{ave} = \frac{1}{\frac{P_1}{100G_1} + \frac{P_2}{100G_2} + \ldots + \frac{P_n}{100G_n}} \quad (8)$$

$$a = \left[\left(\frac{B-A}{A}\right)\right] \times 100 \quad (9)$$

$$a_{ave} = \left[\left(\frac{P_1 \times a_1}{100}\right) + \left(\frac{P_2 \times a_2}{100}\right) + \ldots + \left(\frac{P_n \times a_n}{100}\right)\right] \quad (10)$$

where,

G_{od}	= Bulk Specific Gravity (oven-dry)
G_{ssd}	= Bulk Specific Gravity (saturated surface dry)
G_a	= Apparent Specific Gravity
A	= Weight of oven-dry test sample in air, g
B	= Weight of saturated surface-dry test sample in air, g
C	= Weight of saturated test sample in water, g
G_{ave}	= Average Specific Gravity
$P_1, P_2, ..., P_n$	= Weight percentages of each size fraction present in the original sample,
a	= Absorption, %
$a_1, a_2, ..., a_n$	= Absorption percentages for each size fraction, %

For fine aggregates, the pycnometer was partially filled with water, saturated surface-dry fine aggregate, weighing 500 g, was introduced into the pycnometer and filled with additional water to approximately 90% of the capacity. The pycnometer was manually rolled, inverted and agitated to eliminate all air bubbles. After eliminating all air bubbles, the temperature of the pycnometer and its contents was adjusted to 23°C and the total mass of the pycnometer, specimen and water was determined. The fine aggregates were removed from the pycnometer, dried to a constant mass at 110°C, cooled in air at room temperature for 1 h and then, the mass was determined. Finally, the mass of the pycnometer filled to its calibrated capacity with water at 23°C was determined. The bulk SG at 23°C was calculated using Eq. (11 to 14):

$$G_{od} = \frac{A}{(B+S-C)} \quad (11)$$

$$G_{ssd} = \frac{S}{(B+S-C)} \quad (12)$$

$$G_a = \frac{A}{(B+A-C)} \quad (13)$$

Table 4: Calculation of specific gravity and absorption

	Specific gravity and absorption of aggregate			
Aggregate size, (mm)	4.75>	4.75-9.5	9.5-25	Average
Mass of oven-dry test sample, (g)	500.000	2000.510	3000.200	-
Mass of Pycnometer filled with water, (g)	1181.5	-	-	-
Mass of Pycnometer filled with specimen and water, (g)	1499.89	-	-	-
Mass of saturated-surface-dry test sample, (g),	501.950	2011.550	3073.306	-
Mass of saturated test sample in water, (g)	318.400	1271.230	1899.933	-
Bulk specific gravity (Oven-Dry)	2.724	2.702	2.557	2.667
Bulk specific gravity-Saturated Surface Dray (SSD)	2.735	2.717	2.619	2.695
Apparent specific gravity	2.753	2.743	2.727	2.742
Density (OD), (kg/m^3)	2717.243	2695.468	2550.511	2660.811
Density (SSD), (kg/m^3)	2727.841	2710.343	2612.659	2688.306
Apparent density, (kg/m^3)	2746.421	2736.272	2719.975	2735.050
Absorption (%)	0.390	0.552	2.437	0.973

$$a = \left[\left(\frac{S-A}{A}\right)\right] \times 100 \tag{14}$$

where,
A = Mass of oven dry specimen, g
B = Mass of pycnometer filled with water, g
C = Mass of pycnometer filled with specimen and water, g
S = Mass of saturated surface-dry specimen, g
a = Absorption, %

Table 4 shows the calculation of Specific Gravity and Absorption for coarse nd fine aggregates.

Linear shrinkage: Measuring the shrinkage characteristics of the soil can help delineate clay mineralogy and shrink/swell potential of a geologic deposit. ASTM C 356 and BS 1377 part 2 cover the determination of the total linear shrinkage of a fraction of the soil sample passing the 425-μm test sieve by using linear measurement on a bar of soils. Linear shrinkage, as used in this test method, refers to the change in linear dimensions in the test specimens after they have been subjected to soaking heat for a period of 24 h and then cooled to room temperature. Shrinkage is one of the major causes for volume change associated with variations of water content in soil. A 150 g sample that passes through a 425-μm sieve was prepared and mixed with distilled water until the mass became a smooth homogeneous paste with the moisture level approximately equal to the liquid limit of the soil. The inside of a clean shrinkage mold was greased and the mixture was placed in a mold of dimensions 140 mm length and 12.5 mm radius, as shown in Fig. 13. Care is taken to thoroughly remove all air bubbles from each layer by lightly tapping the base of the mold. The mold was slightly overfilled and then the excess material was leveled with a spatula. The specimen was transferred into an oven and dried at 110°C. It was allowed to cool and then its longitudinal shrinkage, LD, was measured to the nearest millimeter (Fig. 14). The linear shrinkage of the soil was calculated using Eq. (15) as follows:

$$LS = \left(1 - \frac{L_D}{L_0}\right) \times 100 \tag{15}$$

Fig. 13: Preparation of LS specimen

Fig. 14: LS of the sample

Fig. 15: Flakineds index apparatus

Fig. 16: Elongation index apparatus

where,
LS = Linear shrinkage, %
L_D = The length of oven-dry specimen, mm
L_0 = Original length of specimen, mm

The results of LS is shown in Table 2.

Flakiness index, elongation index and average least dimension: For base course aggregates, the presence of flaky particles is considered undesirable as they may cause inherent weakness with the possibility of breaking down under heavy loads. A flaky particle has a thickness (least dimension) less than 0.6 times the mean size of the fraction to which the particle belongs. An elongated particle has a length that is more than 1.8 times the mean sieve size of the sieve fraction to which the particle belongs. Sieve a sufficient but known quantity of coarse aggregates to provide a minimum of 200 pieces on each of the above sieves. The flaky material of each fraction was separated by placing it on the thickness gauge and the total weight was determined in the appropriate slot of material. The total amount of material passing through the gauge was weighed to an accuracy of at least 0.1% of the weight of the test sample. The Flakiness Index (FI) was reported as the total weight of material passing through the various thickness gauges expressed as a percentage of the total weigh of the sample gauge to the nearest whole number. The Elongation Index (EI) was obtained in same way. Figure 15 and 16 show the FI and EI apparatuses. Equation (16) and (17) show the calculation of the value of FI and EI:

$$FI = \frac{M_3}{M_2} \times 100 \tag{16}$$

$$EI = \frac{M_3}{M_2} \times 100 \tag{17}$$

where,

FI = Flakiness index, %
M_3 = Weight of all the particles passing each of the gauges, g
M_2 = Sum of the masses of fraction, g
EI = Elongation index, %

The Average Least Dimension (ALD) of an aggregate particle is the smallest perpendicular distance between two parallel plates through which the particle will just pass. ALD is the arithmetic mean of all the measured least dimensions of the aggregate particles measured. The sieve size through which 50% of the sample passed and the Flakiness Index were determined. ALD is read from the monograph shown in

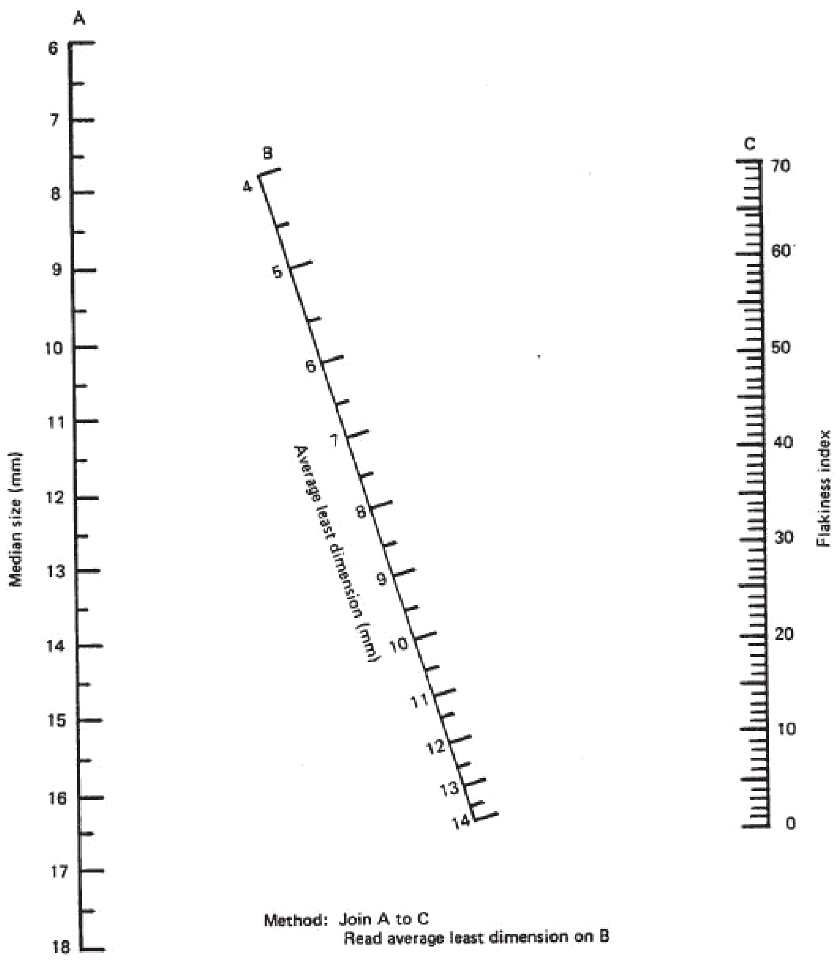

Fig. 17: Monograph to calculate ALD

Fig. 17 by joining the scale for the median size to the scale for FI. The results of FI, EI and ALD are shown in Table 2.

Sand equivalent: The ASTM D 2419 test method assigns an empirical value to the relative amount, fineness and character of the clay-like material present in the test specimen. This test method is intended to serve as a rapid field correlation test. The purpose of this test method is to indicate, under standard conditions, the relative proportions of clay-like or plastic fines and dust in granular soils and fine aggregates that pass the 4.75-mm (No. 4) sieve. For the test, 1500 g of material passing the 4.75-mm (No. 4) sieve was obtained, dried to a constant weight at 110°C and was cooled to room temperature before testing. Calcium chloride solution (102 mm, indicated on the graduated cylinder) was siphoned into the plastic cylinder. The test specimen was poured into the plastic cylinder using the funnel to avoid spillage. The bottom of the cylinder was tapped sharply several times using the heel of the hand to release air bubbles and to promote thorough wetting of the specimen. The wetted specimen and cylinder was allowed to stand undisturbed for 10 min. At the end of the 10-min soaking period, the cylinder was stoppered and the material was then loosened from the bottom by partially inverting and simultaneously shaking the cylinder. After loosening the material from the bottom of the cylinder, the cylinder and its contents were shaken. Following that, the cylinder was set upright on the work-table and the stopper was removed. The specimen was then irrigated using additional flocculating solution forcing the claylike material to form into a suspension above the sand. The cylinder and its contents were allowed to stand undisturbed for 20 min. After a prescribed sedimentation period, the height of flocculated clay was read and the height of sand in the cylinder was determined. The sand equivalent is calculate using following Eq. (18). Figure 18 shows a schematic of cylinder and contents to calculate SE:

$$SE = (\text{Sand Reading}/\text{Clay Reading}) \times 100 \quad (18)$$

where, SE is sand equivalent, %. The results of SE is shown in Table 2.

Los angeles abrasion: The ASTM C 131 test method covers a procedure for testing the size of coarse aggregates that are smaller than 37.5 mm and resistant to degradation by using the Los Angeles testing machine. This test has been widely used as an indicator of the relative quality or competence of various sources of aggregates having similar mineral compositions. The results do not automatically permit valid comparisons to be made between sources distinctly different in origin, composition, or structure. The specification limits need to be assigned with extreme care according to the available aggregate types and their performance history in specific end uses. The sample was washed and oven dried at 110°C to substantially constant mass and separated into individual size fractions. The test sample and the charge was placed in the Los Angeles testing machine and the machine was rotated at a speed of 30-33 r/min for 500 revolutions. Subsequently, the material was discharged from the machine and a preliminary separation of the sample was done on a sieve coarser than the 1.70-mm (No. 12) sieve. The material coarser than the 1.70-mm (No. 12) sieve was washed and oven-dried at 110°C to substantially constant mass and the mass was determined in grams to the nearest whole number. Figure 19 shows the Los Angeles Abrasion apparatus. The loss (difference between the original mass and the final mass of the test sample) was calculated as a percentage of the original mass of the test sample. The result of the Los Angeles abrasion test is shown in Table 2.

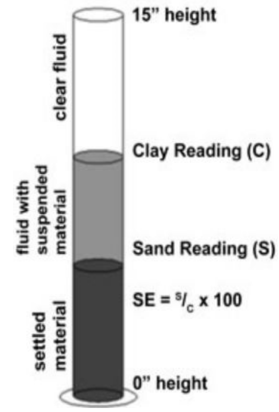

Fig. 18: Measurement of SE

Fig. 19: Los Angeles abrasion equipment

Unconfined compressive strength: The primary purpose of the Unconfined Compressive Strength (UCS) test is to obtain the approximate compressive strength of mixture that possess sufficient cohesion to permit testing in the unconfined state (ASTM, 2004; Ismail *et al*., 2014). The mixture was prepared according to ASTM D 698, ASTM D 558 and ASTM D 1632 using a metal cylindrical specimen with an internal diameter of 101.60 mm and 116.4 mm in height (Fig. 20). The average UCS of the specimens was obtained using a hydraulic compressive strength

Fig. 20: Compaction equipment

Fig. 21: UCS testing machine

Fig. 22: CBR apparatuses

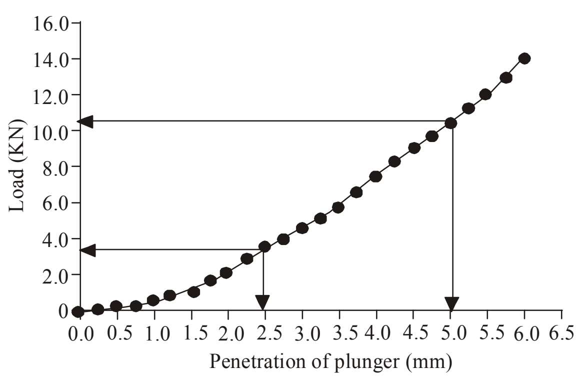

Fig. 23: CBR calculation

machine (Fig. 21) by applying load to a constant rate within the limits of 140±70 kPa/s according to ASTM D 1633. Finally, the unit compressive strength (MPa) was calculated by dividing the maximum load (N) by the cross-sectional area (mm^2). The result of UCS is presented in Table 2.

California bearing ratio test: ASTM D 1883 test method covers the determination of the California Bearing Ratio (CBR) of pavement subgrade, subbase and base/course materials from laboratory compacted specimens. This test method is used to evaluate the potential strength of subgrade, subbase and base course material, for use in road and airfield pavements. The CBR value obtained in this test forms an integral part of several flexible pavement design methods. The sample was handled and specimens for compaction were prepared in accordance with the procedures given in part 4.1 (Moisture-Density Relations of Soil aggregate mixture) according ASTM D 698 for compaction in a 6-in. (152.4-mm) mould. The penetration piston was seated with the smallest possible load, but in no case in excess of 10 lbf (44 N). Both the stress and penetration gages were set to zero. This initial load is required to ensure satisfactory seating of the piston and shall be considered as the zero load when determining the load penetration relation. The load was applied on the penetration piston so that the rate of penetration was approximately 0.05 in. (1.27 mm)/min (Fig. 22).

The load readings was recorded at penetrations of 0.025 in. (0.64 mm), 0.050 in. (1.27 mm), 0.075 in. (1.91 mm), 0.100 in. (2.54 mm), 0.125 in. (3.18 mm), 0.150 in. (3.81 mm), 0.175 in. (4.45 mm), 0.200 in. (5.08 mm), 0.300 in. (7.62 mm), 0.400 in. (10.16 mm) and 0.500 in. (12.70 mm). The penetration load in kilonewton (KN) was calculated and the load-penetration curve was plotted. Bearing Ratio was calculated Using corrected load values taken from the load- penetration curve for 0.100 in. (2.54 mm) and 0.200 in. (5.08 mm) penetrations. The bearing ratios was evaluated for each by dividing the corrected load by the standard stresses of 1000 psi (6.9 MPa) and 1500 psi (10.3 MPa) respectively and multiplying by 100. Figure 23 shows the values of load (KN) for penetration of 2.54 mm and 5.08 mm are 3.5 KN and 10.4 KN respectively.

The following calibrated Equation 18 is applied to calculate the load and Eq. (19) and (20) show the calculation of CBR. The result of CBR is presented in Table 2:

$$P = A + B(X) - C(X)^2 \tag{19}$$

where,
P = load (KN)
A = 0.0467635
B = 0.03331777
C = 0.0000004
X = Gauge Reading (Top)

$$CBR = \frac{P}{6.9} \times 100 \tag{20}$$

$$CBR = \frac{P}{10.3} \times 100 \tag{21}$$

where,
CBR = California Bearing Ratio, (%)
P = Load (KN)

DISCUSSION AND CONCLUSION

For soil aggregate materials to be used effectively in road construction, it is necessary to understand their properties. In addition, road-base materials must be characterized to design an adequate pavement structure for an expected traffic. The results of this study focused on the physical, chemical and mechanical behaviors of soil aggregate that are used as granular road-base materials in order to provide a rudimentary understanding of these materials on a mechanistic-empirical pavement design and analysis using experimental laboratory tests. These tests were performed in accordance with the ASTM and BS on fine and coarse soils and base aggregates used in pavements. The results of these tests are summarized in Table 2. The suitability of the road-base material would have to be verified by conducting laboratory testing. The pavement designer is required to provide a specification that ensures that the quality and consistency of the required road-base materials are confirmed and that the engineering properties of the pavement base materials are realistic and achievable.

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