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16. Abstract

It has been well demonstrated that a positive subsurface drainage is beneficial in enhancing pavement performance and thus extending pavement service life. Typical permeable base materials include asphalt/cement-treated, open-graded aggregates and unbound aggregates. Although asphalt/cement-treated, open-graded permeable bases perform well based on the past engineering practice, they are expensive solutions and less desirable for some roadways when compared to unbound aggregates, especially for low- to medium-volume roadways. In these situations, it is possible to use a properly graded unbound aggregate that is adequately drainable and structurally stable during the construction and service lifetime after the roadway is open to traffic.

This study is to determine a proper/optimum gradation by conducting laboratory testing for unbound aggregates of Mexican limestone that are commonly used in Louisiana highways. However, there is trade-off between structural stability and permeability of unbound aggregates. The increase of permeability is often at the cost of structural stability or vice verse. Therefore, the criteria for selecting an optimum gradation are: (1) an adequate permeability to drain the infiltrated-water from the pavement as quickly as possible; and (2) a sufficient structural stability to support the traffic loading. The permeability of unbound aggregate is quantified by its saturated hydraulic conductivity while its structural stability is characterized by various laboratory tests on the strength, stiffness, and permanent deformation of the material. A series of laboratory tests, including constant-head permeability, California Bearing Ratio (CBR), Dynamic Cone Penetrometer (DCP), tube suction (TS), monotonic load traiaxial tests, and repeated load triaxial (RLT) tests, were conducted on Mexican limestone with different gradations. The gradations under investigation include coarse and fine branches of Louisiana class II gradation, New Jersey gradation medium, and an optimum gradation (fine and coarse branches). The optimum gradation is the result of a series of laboratory trial-error tests.

The results from laboratory tests indicate that: (1) the coarse branch of Louisiana class II gradation outperform the fine counterpart in terms of permanent deformation and hydraulic conductivity; (2) CBR and DCP values may not be good properties to differentiate performance of unbound aggregate with different gradations; and (3) an optimum gradation is identified, which outperforms current Louisiana class II base gradation in terms of both structural stability and permeability.

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Effect of Drainage in Unbound Aggregate Bases on Flexible Pavement Performance

by

Mingjiang Tao, Ph.D., P.E.

Murad Abu-Farsakh, Ph.D., P.E.

Louisiana Transportation Research Center
4101 Gourrier Avenue
Baton Rouge, LA 70808

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ABSTRACT

It has been well demonstrated that a positive subsurface drainage is beneficial in enhancing pavement performance and thus extending pavement service life. Typical permeable base materials include asphalt/cement-treated, open-graded aggregates and unbound aggregates. Although asphalt/cement-treated, open-graded permeable bases perform well based on the past engineering practice, they are expensive solutions and less desirable for some roadways when compared to unbound aggregates, especially for low- to medium-volume roadways. In these situations, it is possible to use a properly graded unbound aggregate that is adequately drainable and structurally stable during the construction and service lifetime after the roadway is open to traffic.

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The results from laboratory tests indicate that: (1) the coarse branch of Louisiana class II gradation outperform the fine counterpart in terms of permanent deformation and hydraulic conductivity; (2) CBR and DCP values may not be good properties to differentiate performance of unbound aggregate with different gradations; and (3) an optimum gradation is identified, which outperforms current Louisiana class II base gradation in terms of both structural stability and permeability.

ACKNOWLEDGEMENTS

The Louisiana Department of Transportation and Development (DOTD) financially supported this research through the Louisiana Transportation Research Center (LTRC) under State Project No. 736-99-1307 and LTRC Research Project No. 04-4GT. The authors would like to thank Vulcan Material Company for providing the base course materials used in this study. The comments and suggestions of Zhongjie "Doc" Zhang, Pavement and Geotechnical Administrator of LTRC, are gratefully acknowledged. The authors also appreciated the assistance from the geotechnical, pavement, and bituminous groups of the LTRC. The student workers from Louisiana State University also played an important role in conducting the laboratory tests at LTRC.

IMPLEMENTATION STATEMENT

This study identified an optimum range of unbound aggregate gradation (optimum-fine to optimum-coarse) that satisfies both permeability and structural stability as a drainable base material. The researchers' recommendation, based on this study, is that the LA DOTD considers using the optimum gradation when unbound drainable base layers are used in pavement structure, which are expected to have better long-term performance than non-drainable bases. Field test sections can help in validating the constructability and long-term benefits of using drainable bases in pavements. However, since the variation between the optimum-fine and optimum-coarse aggregate gradations is narrow, it may be difficult to achieve and control in the field. To achieve this gradation in the field will require running a pug-mill on the site and a spreader (paving machine) to lay the material. This additional handling by the material suppliers and the contractor will be associated with an extra cost of \$18.50 per cubic yard (or ≈ 25%) over the current bid prices.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

Beneficial effects brought about by the inclusion of a subsurface drainage system in pavements can only be realized with a properly designed and constructed system that consists of all essential components and a drainable layer with adequate drainability and sufficient structural stability.

Essential Components of a Subsurface Drainage System

The presence of free moisture in pavement layers has been well documented in the literature and found responsible for many premature failures observed in both flexible and rigid pavements. The detrimental effects of free moisture in pavement include: (1) stripping in hot mix asphalt (HMA); (2) pumping in concrete pavement with subsequent faulting, cracking, and general shoulder deterioration; (3) the reduction of stiffness and strength in pavement layers; (4) debonding among pavement layers; and (5) overall, the reduction of pavement service life [*1*]. The real destructive force of free moisture in pavement layers lies in the development of high pore pressures with saturated pavement base layers under repetitive traffic loading, which results in a significant loss of shear strength and stiffness in the pavement base and subgrade layers.

Free moisture is largely from the infiltration through pavement surface joints or cracks. To mitigate the moisture-induced distresses, it is imperative to drain free moisture out of pavement structures as quickly as possible by a subsurface drainage system. A complete pavement drainage system consists of a permeable aggregate base layer, longitudinal drains, and transverse outlet systems daylighted to surface drainage channels, as shown in Figure 1. Although the performance of a subsurface drainage system depends on all of its individual components, the permeability of permeable base layer is one of the most critical factors in designing such a system.

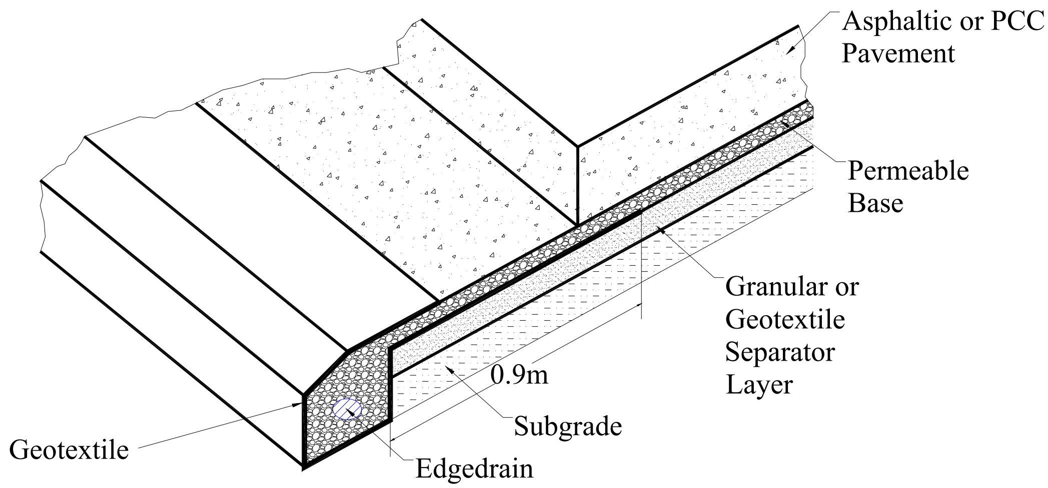


Figure 1: Components of a subsurface drainage system (after [*1*])

Drainability Requirement

The principle in designing the subsurface drainage system is that infiltrated water should be drained out of the pavement system within a reasonable amount of time. The process of water infiltrating into the pavement system is complicated, and an accurate estimation of infiltration rate is still difficult to determine due to the non-uniformity of the pavement surface. Cedergran et al. [*2*] proposed one method for calculating the infiltration rate on the basis of precipitation rate (inches/hour) and an infiltration coefficient, depending on pavement type. The infiltration coefficient ranges from 0.33 to 0.50 for flexible pavements and 0.50 to 0.67 for rigid pavements. Figure 2 shows generalized rainfall intensities for a two-year frequency, one-hour duration rainfall, which also represents the average worst storm that occurs each year from a hydrologic standpoint.

For southern Louisiana areas, which have a precipitation rate of 2.2 inches and using the infiltration coefficients suggested for flexible pavement, the resulting infiltration rate would range from 0.45 to 2.2 $ft^3/day/ft^2$. Due to high uncertainties associated with this approximation method, 2.2 $ft^3/day/ft^2$ infiltration rate is suggested for an adequate subsurface drainage design.

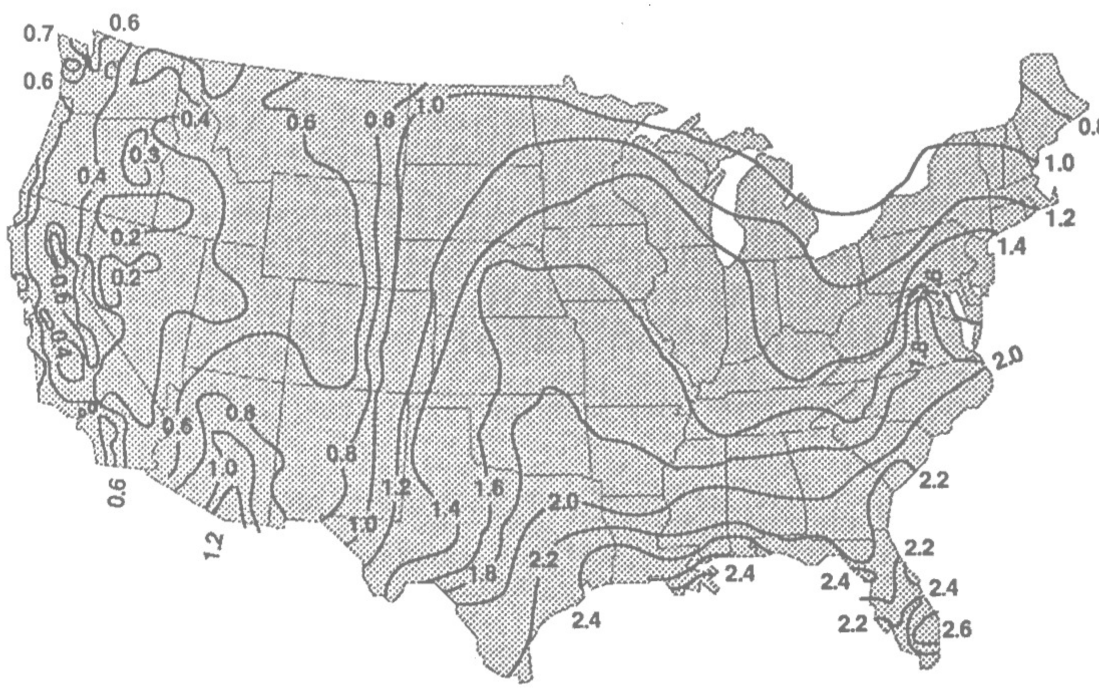


Figure 2: Maximum one-hour duration/two-year precipitation in the United States (After [*2*])

The time to drain the water out of the pavement system depends on multiple factors, such as infiltration rate, flow-path gradient, flow-path length, and hydraulic conductivity of the pavement material. For typical pavement geometry, the water flow is primarily horizontal. The flow-path gradient, S_R, a key factor for horizontal flow analysis, is a resultant slope of cross slope (S_C) and longitudinal gradient (S_L), as illustrated in Figure 3.

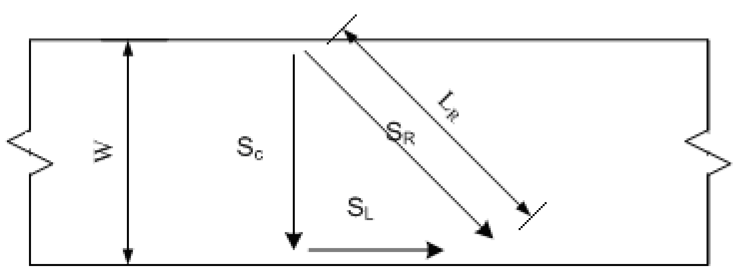


Figure 3: Typical pavement flow-path gradient

Then, the resultant slope (S_R) and length of flow path (L_R) through the pavement base can be calculated by using equations (1) and (2):

$$S_R = \sqrt{\left(S_C^2 + S_L^2\right)} \qquad (1)$$

$$L_R = W\left[1+\left(\frac{S_L}{S_C}\right)^2\right]^{1/2} \quad (2)$$

Where: S_R, S_C, and S_L are defined previously; and W is the width of drainage path.

The equations above suggest that the increase of pavement cross slope result in the decrease in the length of flow path at a given longitudinal gradient. The consequence of decreasing flow path length is a reduction in drainage time and the reduced moisture-related damage to the pavement. Therefore, it is important to take the pavement geometry into consideration in the design of a subsurface drainage system. Although this steady-state flow analysis had been a simple approach in hydraulic design of a permeable base, two major problems were associated with this approach–estimating the design rainfall rate and estimating the portion of rainfall that enters the pavement made this approach less preferable than the time-to-drain approach.

Different than the above steady-state flow analysis is the time-to-drain approach in hydraulic design of a permeable base, which is based on flow entering the pavement until the permeable base is saturated. There are two approaches for characterizing the time to drain–one is the AASHTO percent drained (50 percent) and the other is 85 percent saturation. AASHTO percent drained (50 percent) assesses the drainability of pavement subsurface system, on the basis of the time within which 50 percent of infiltrated water is drained, as shown in Table 1 [*3*]. The pavement rehabilitation manual also recommends a criterion to evaluate drainage quality of a pavement based on the time with which the amount of infiltrated water is drained to the extent that the degree of saturation for the base layer is below 85 percent [*4*]. This criterion is tabulated in Table 2.

Table 1: Criterion of pavement drainability (AASHTO pavement design guide)

Quality of drainage	Time to drain 50% infiltrated water
Excellent	2 hours
Good	1 day
Fair	7 days
Poor	1 month
Very Poor	Does not drain

Table 2: Criterion of evaluating drainability of a pavement (Pavement Rehabilitation Manual)

Quality of drainage	Time to drain[a]
Excellent	< 2 hours
Good	2-5 hours
Fair	5-10 hours
Poor	> 10 hours
Very Poor	>> 10 hours

Note: [a] time to drain infiltrated water so that the degree of saturation in the base layer is less than 85 percent

Casagrande and Shannon proposed a relationship, as given in Eq. (3), for approximating the time to achieve 50 percent drainage under unsteady-state flow conditions [5].

$$t_{50} = \frac{n_e L^2}{2k\left(H + S_R L_R\right)} \tag{3}$$

Where: t_{50} = the time to achieve 50 percent drainage; n_e = effective porosity of permeable base material; k = the coefficient of hydraulic conductivity; L = length of drainage layer; H = thickness of drainable layer; and other symbols are defined in preceding sections.

Barber and Sawyer suggested a chart, with which the drainage time for any degree of drainage for a given slope condition can be determined [6]. This procedure can also be used to determine the required hydraulic conductivity for achieving a desired degree of drainage.

It can be seen that the hydraulic conductivity of a pavement base layer required to effectively drain the infiltrated water is not a fixed value, which is actually depending on a variety of factors, with some of them listed below:

- Infiltration rate that depends on climatic conditions at pavement sites and pavement surface conditions (spacing of cracks on pavement surface, joints conditions, etc.)
- Pavement geometry (cross slope, longitudinal gradient, width of pavement, and number of lanes)
- Thickness of permeable base, and

- Degree of drainage required to minimize moisture-induced damage to the pavement that, in turn, depends on the gradation and water susceptibility of base aggregate.

Structural Stability of Pavement Permeable Base

A satisfactorily performing permeable base should also have adequate resistance to permanent (plastic) deformation under construction and normal traffic loading while at the same time having the minimum drainability as discussed in the previous section. Therefore, it is imperative to consider structural stability in the optimization of permeable base materials.

Traditionally, the CBR has been used to quantify structural stability of a permeable base material because it is a simple and rapid procedure to characterize pavement materials. Although the CBR has long been used by pavement engineers in characterizing pavement base/subbase and subgrade soils, it does not relate well to stiffness of soils at low strains, which is of primary interest in pavement design. Brown suggests that resilient modulus and the potential of developing permanent (plastic) strains under repeated loading be parameters for pavement design [*7*]. However, none of these parameters can reliably be correlated with CBR results.

As a rapid and effective quality control tool to characterize pavement materials in the field, the DCP has gained increasing popularity in pavement engineering. Many correlations between the penetration index determined from DCP and strength parameters have been developed. However, all the published relationships are only applicable to certain soil types and conditions, without a generalized relationship for all cases.

Repeated load triaxial test (RLT) is arguably superior to static tests such as CBR or DCP since it can characterize pavement material response under repeated loading that simulates traffic loading conditions. Both resilient modulus and permanent deformation of a pavement material can be determined from this test, with the former being an essential input parameter in pavement design and the latter reflecting the rutting potential of a pavement material in field conditions.

From this literature review, some findings can be summarized below:

- There exist various key factors that affect the determination of the minimum hydraulic conductivity of a permeable base layer or the time to achieve a certain percentage of drainage. These key factors include properties of base aggregates, geometry of pavements, climatic conditions, and pavement surface conditions.

- Due to the dependence of minimum required hydraulic conductivity or the appropriate drainage time on multiple factors, there is no consensus on the minimum value for the coefficient of hydraulic conductivity or the time to achieve a given percentage of drainage.
- Similar to hydraulic conductivity, the minimum structural stability required for a permeable aggregate base is not well established, neither material parameter to quantify the structural stability of a permeable aggregate. Therefore, it is warranted to identify or develop a laboratory testing procedure that can provide a better characterization of structural stability in optimizing permeable aggregates.

Unbound Permeable Aggregate Base Layer

Cement/asphalt-stabilized, open-graded aggregates are often used as permeable base materials because of their good structural stability and high permeability. Unfortunately, these bonded permeable materials may not be a good option to low- to medium-volume highways due to their high costs. Several state DOTs (Departments of Transportation) are interested in unbound permeable aggregate, which can be a more cost-effective alternative when properly graded. To ensure proper performance, a permeable unbound aggregate should have adequate permeability while remaining structurally stable during construction and throughout pavement service life. However, there is a trade-off between permeability and structural stability of unbound aggregates (i.e., the increase in permeability is generally accompanied by the decrease in structural stability, or vice versa). It was reported that both permeability and structural stability of an unbound aggregate are affected largely by its particle size distribution (gradation), particle shape, and angularity [*8*]. Therefore, it is feasible to identify a proper/optimum gradation for a certain aggregate that will satisfy both permeability and structural stability requirements. How to identify such an optimum gradation for a crushed limestone to be used in Louisiana's highway is the focus of this study.

OBJECTIVE

The main objective of this study is to optimize gradation of Mexican limestone aggregate for use as an unbound drainable base material that has adequate permeability while staying structurally stable during the construction time and the pavement's service life.

SCOPE

A series of laboratory tests were conducted to optimize the gradation of Mexican limestone aggregate material, which is currently used in the construction of pavement base layers in Louisiana's highways, for application use as drainable bases to satisfy both permeability and structural stability criteria. Basic and specific properties of the Mexican limestone with various gradations were determined. Five different gradations (Louisiana class II-coarse, Louisiana class II-fine, New Jersey-medium gradation, optimum-coarse gradation, and optimum-fine gradation) were examined in terms of their permeability and structural stability. The laboratory tests included constant-head hydraulic conductivity tests, tube suction tests, California bearing ratio (CBR) tests, dynamic cone penetrometer tests, monotonic load triaxial tests, and repeated load triaxial tests. Based on the results of laboratory tests, a range of optimum aggregate gradation was recommended.

METHODOLOGY

To identify an optimum gradation for the Mexican limestone aggregate having sufficient permeability while staying structurally stable for use as a drainable base, extensive laboratory tests were performed to determine the basic physical properties, permeability, and structural stability of five different gradations. These gradations include Louisiana class II coarse, Louisiana class II fine, New Jersey medium gradation, optimum-coarse gradation, and optimum-fine gradation, and will be defined later. The laboratory tests included standard geomaterial tests, constant head hydraulic conductivity test, California Bearing Ratio (CBR) tests, Dynamic Cone Penetrometer (DCP) tests, Monotonic load triaxial test, and repeated load triaxial (RLT) test.

Physical Properties Tests

Basic physical properties of Mexican limestone, including specific gravity, gradation analysis, plasticity index (PI), and moisture–density relationship, were determined in accordance with respective ASTM specifications (ASTM D854, ASTM D422, ASTM D4318, and ASTM D698). These physical properties were used to provide some preliminary characterization and classification for the tested unbound aggregate. Also, the moisture-density relationship for specimens with different gradations provides the information of optimum moisture content and maximum dry density that will be used in preparing samples for other tests.

Test Procedure to Generate New Gradations

Since the process of determining an optimum gradation that meets both permeability and structural stability is “trial-and-error” in nature, the Mexican limestone specimens in different gradations were required during this study. Different gradations were obtained by first sorting original unbound aggregate into different particle size groups and then remixing these sorted groups into a desired proportion. This sorting and remixing process is illustrated by a schematic diagram in Figure 4, and Figure 5 shows a photo of the shaker used to separate original unbound aggregate into different particle size groups.

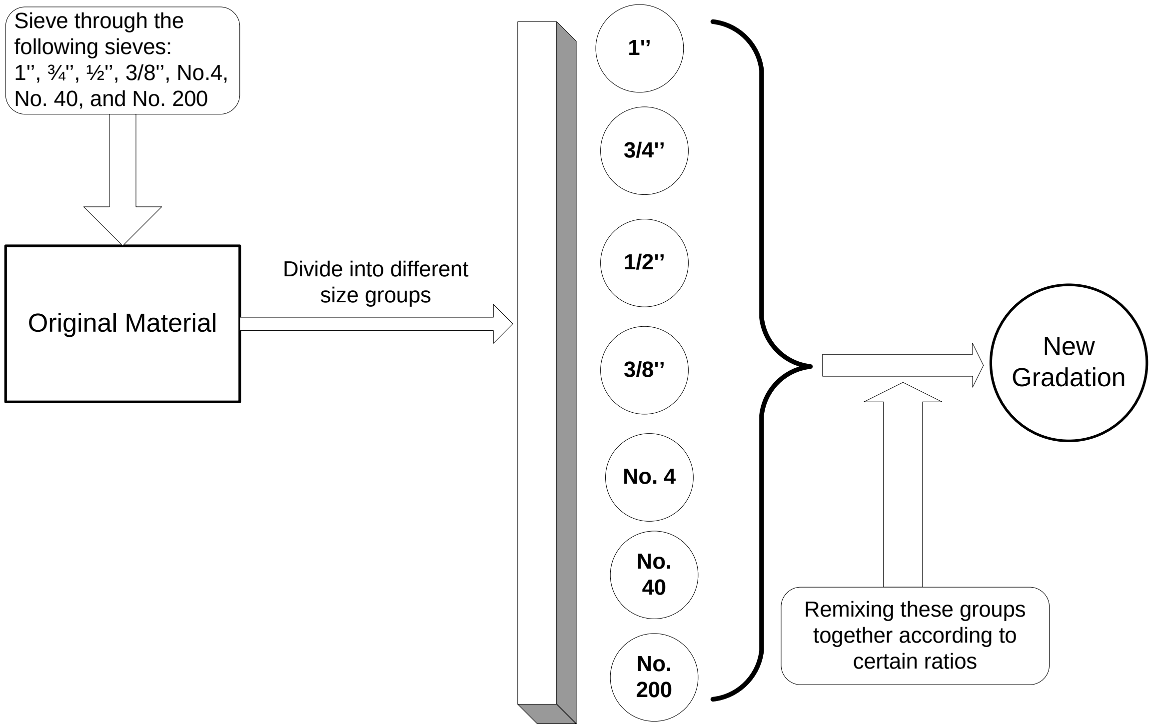


Figure 4: Testing procedure to obtain different particle size gradations

Figure 5: Picture of shaker machine for sorting unbound aggregate

Constant-Head Hydraulic Conductivity Test

Constant–head hydraulic conductivity tests were performed to quantify drainability of Mexican limestone specimens at different gradations. Figure 6 illustrates the setup for the constant–head hydraulic conductivity test, and Figure 7 depicts the device components used in this test. As depicted in Figure 7, the constant–head hydraulic conductivity test consists of a storage water tank that allows the air originally dissolved in the tap water to seep out of the water; a smaller constant–head tank supplying water to test specimens; a rigid-wall permeameter with dimensions of 6 inches diameter and 18 inches height that is customized to test compacted unbound aggregates with large particle size (up to 1 inch particle diameter) and permeability (up to 5,000 ft/day); and a control panel with burettes to measure hydraulic heads during the tests. Three manometer ports (designated as bottom, middle, and top manometer) were drilled 6 inches apart along with axial direction of the permeameter to measure hydraulic head losses during the test. For low to medium hydraulic conductivity specimens, the bottom and middle manometers that are 6 inches apart from one another were used, whereas the bottom and top ones (12 inches apart from each other) were used to have more accurate measurements of hydraulic head losses in specimens with large hydraulic conductivities. The constant–head hydraulic conductivity tests were carried out in accordance with ASTM D2434, with tested specimens prepared at their optimum moisture content and maximum dry density that were determined from standard Proctor test.

Tube Suction Tests

Tube suction test developed by Scullion and Saarenkto [*9*] is a procedure to qualitatively approximate free moisture content in soils through capillarity action by measuring its dielectric constant. The measured dielectric constant of a given soil specimen gives indication of moisture susceptibility of tested soil. The testing procedure can be briefly described as follows, also illustrated in Figure 8:

- Measure the required amount of soil and water, or cement, according to predetermined moisture and dry density, cement content, for a mold 4 inches (100 mm) in diameter and 7 inches (180 mm) in height, and mix them thoroughly
- After the mixture sits for 30 minutes, pour the water-soil mixture into the cylindrical mold in 4 layers
- Compact each layer with a 10 lb. hammer at predetermined blows

- Cure raw soil specimens wrapped in a plastic bag at room temperature for 1 day; cure cement-stabilized specimens wrapped in a plastic bag in a 100 percent humidity room with a temperature of 73°F (23°C) for 7 days
- Put the specimens in oven with a temperature of 104°F (40°C) for 7 days until the weight of specimens are almost constant, and
- Put the dried specimens in a pan with 0.787 inches (20 mm) water and start to take DV readings daily by using a Percometer v.3 capacitance probe until the readings become constant.

More detail about TS tests can be referred to Scullion and Saarenkto [9].

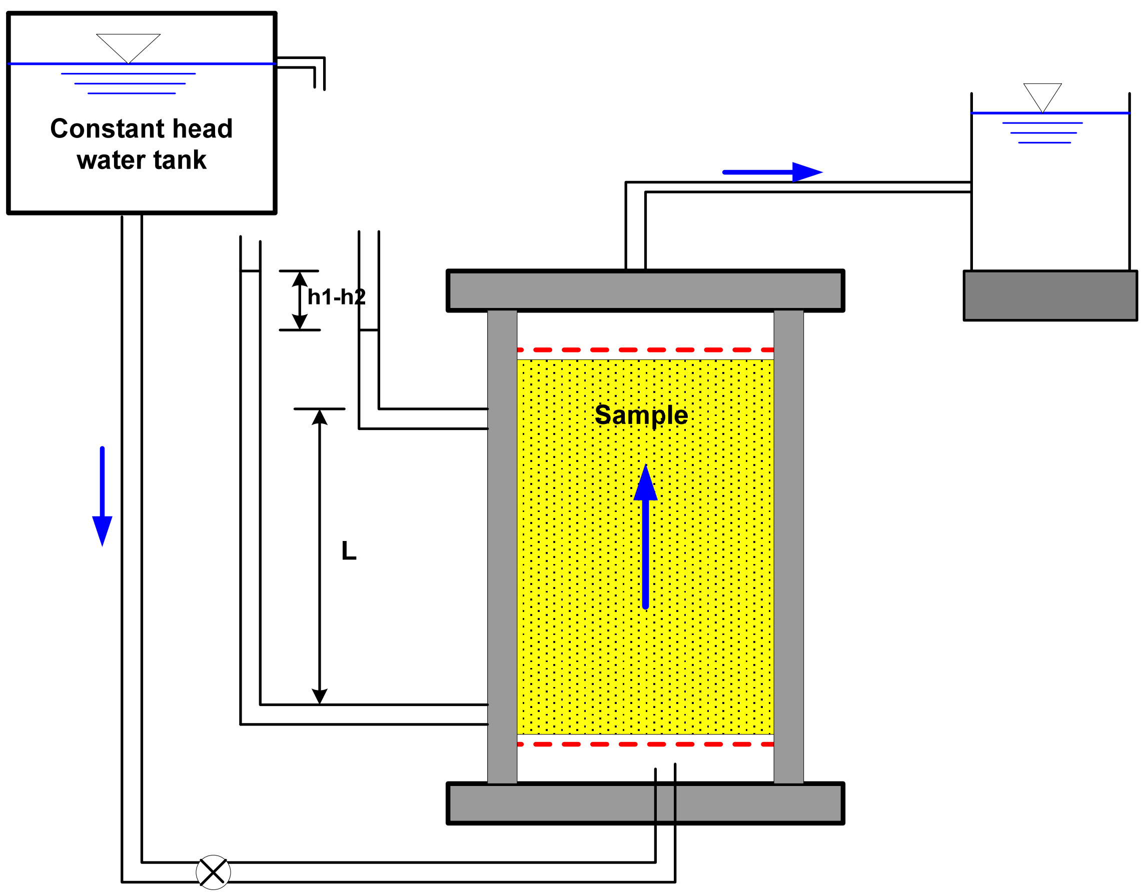


Figure 6: Schematic diagram for set-up of constant-head hydraulic conductivity test

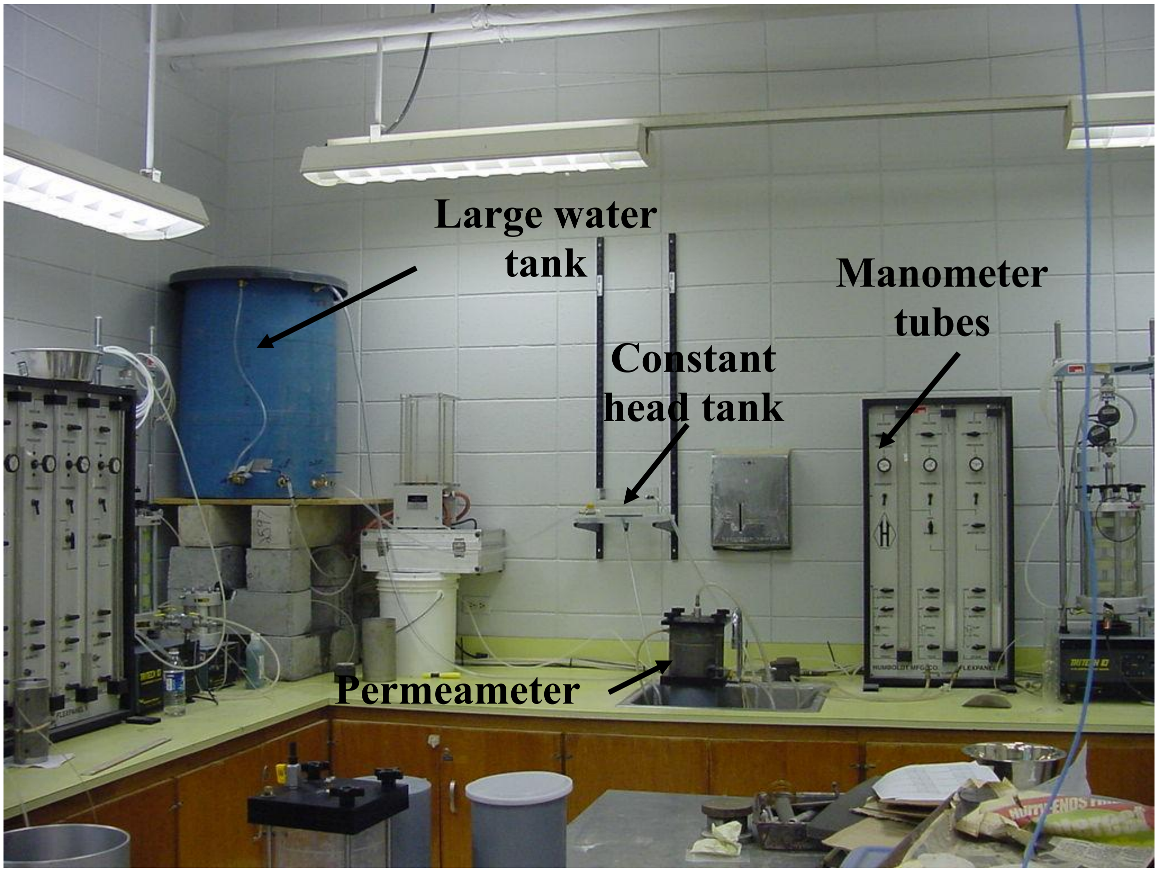
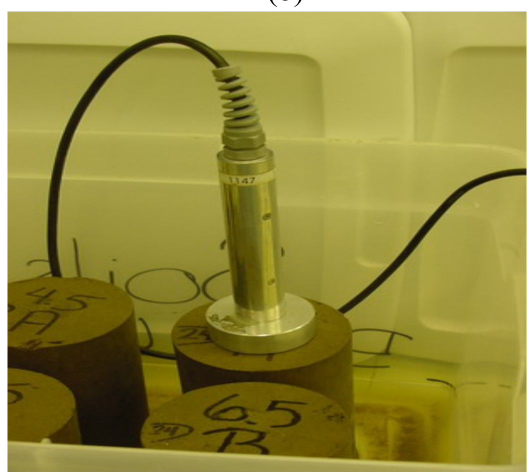


Figure 7: Picture of setup for constant-head hydraulic conductivity test

Shear Strength and Structural Stability Tests

In the use of permeable unbound aggregate base, the structural stability due to limited amounts of fines being allowed is the major concern. Structural stability implies that unbound aggregate maintains integrity with negligible amount of permanent (plastic) deformation during construction and throughout pavement service life, depending on shear strength, stiffness, and the level of applied loading among other factors. California bearing ratio (CBR) and dynamic cone penetrometer (DCP) tests are commonly used to characterize structural stability of pavement base materials due to the ease of conducting these tests. Since both CBR and DCP are primarily used for determining shear strength rather than stiffness of soils under monotonic loading triaxial tests, repeated load triaxial (RLT) tests were also performed to fully characterize structural stability of Mexican limestone in various gradations. Each of these testing procedures is briefly described below.

(a) (b)



(c) (d)

Figure 8: Tube suction test procedure: (a) sample compaction; (b) samples ready for TS test; (c) capacitance probe; and (d) taking readings

CBR

The CBR test is a relatively simple testing procedure that is commonly used to characterize shear strength of pavement base, subbase, and subgrade soils. It is normally performed on compacted, reconstituted samples, although it can also be conducted on undisturbed samples or on soils in the field. In this study, Mexican limestone specimens in various gradations were prepared at their respective optimum moisture content and maximum dry density for the CBR test. For each gradation under investigation, both unsoaked and soaked CBR tests were performed. The former is penetrated after the specimen is prepared and the latter is tested after the specimen is soaking in water for 96 hours to evaluate the influence of saturation on the CBR value. All the CBR tests were performed in compliance with ASTM D1883.

DCP

The Dynamic Cone Penetrometer (DCP) is a simple and effective tool for evaluating in-situ strength of pavement layers and subgrades [*10*]. It consists of an upper fixed 22.7 inch (575 mm) travel rod with 17.6 lb. (8 kg) falling weight hammer, a lower rod containing an anvil, and a replaceable 60° cone of 3/4 inches (20 mm) diameter. It provides continuous measurements of in-situ strength of subgrade soils without sampling. The test involves lifting and dropping 17.6 lb. (8 kg) hammer to strike the anvil and penetrate the 3/4 inches (20 mm) diameter cylindrical cone from the surface down to the required depth. The DCP test was conducted on compacted aggregate in a 12 in. × 12 in. × 12 in. pit in the center of a steel frame box, as shown in Figure 9. The adjacent soil that provides lateral confinement for the DCP specimens is compacted silty clay left from previous research project. Mexican limestone specimens in various gradations were compacted into the pit at their respective optimum moisture content and maximum dry density. All the DCP tests were performed in accordance with ASTM D6951.

Figure 9: DCP test conducted in the pit of a steel-frame box

Monotonic Load Triaxial Tests

Monotonic load triaxial compression tests are usually used to evaluate the strength and stiffness of tested materials. Drained conventional triaxial tests were performed under 3 psi (21 kPa) confinement pressures on Mexican limestone and Kentucky limestone specimens for comparison. The strain rate used in these tests was 0.00033 in./sec. (0.0084 mm/se.c). Specimens of 6 in. diameter × 12 in. height were prepared similar to the RLT specimens' preparation as will be discussed below.

Repeated Load Triaxial (RLT) Tests

Repeated loading triaxial test (RLT) is customarily the procedure to determine resilient modulus of pavement materials in the laboratory.

Specimen Preparation

A 6 in. by 13 in. split mold and a vibratory compaction device were used for preparing samples, as shown in Figure 10. Two membranes were used to prevent any damage caused by coarse particles during specimen preparation, with the aid of vacuum to achieve a good contact with the mold. Samples were prepared by six two-inch lifts to achieve the uniform compaction throughout the specimen. A predetermined amount of the materials was poured into the mold at each lift. Each layer was then compacted until the required density was obtained as indicated by the distance from the top of the mold to the surface of the compacted layer. The surface of each lift was then lightly scratched to achieve good bonding with the next lift. The compacted samples were 6 in. × 12 in. (diameter by height) cylinders.

Resilient Modulus Tests

The resilient modulus tests were conducted in accord with AASHTO T307-99.

Permanent Deformation Tests

The permanent RLT test consisted of first conditioning the samples in the same procedure used in the AASHTO T307-99 and applying 10,000 repeated load cycles. The purpose of the conditioning step is to remove the majority of the irregularities on the top and bottom surfaces of the test sample and to suppress most of the initial stage of permanent deformation. After the conditioning, samples were subjected to 10,000 load cycles at a constant confining pressure of 10 psi (69 kPa) and a peak deviatoric stress of 15 psi (103.5 kPa). The confinement pressure value was chosen in light of anticipated lateral pressure within a base course layer that was reported in different studies [*11*]. Each loading cycle consisted of the same haversine shaped load–pulse, with a 0.1-second loading duration and a 0.9-second rest period, with the load–pulse shown in Figure 11. The RLT tests were stopped after 10,000 load cycles or when the sample reached a permanent vertical strain of 7 percent.

Figure 10: Compaction of RLT test samples

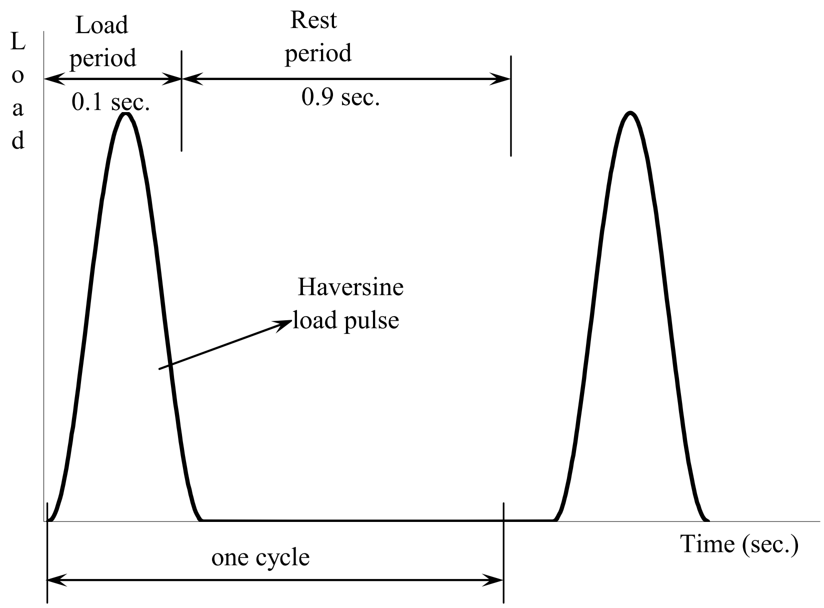


Figure 11: Haversine load pulse used in RLT tests

DISCUSSION OF RESULTS

Test results from laboratory studies on Mexican limestone will be summarized and discussed in this section.

General Properties of Mexican Limestone in Various Gradations

Tested Material

Mexican limestone is a brown aggregate that is often used as base material in Louisiana highways. Louisiana class II gradation (designated as LA II in this report), specified by Louisiana Department of Transportation and Development (LA DOTD), was first evaluated to provide a benchmark for other gradations. The acceptable variation of LA II gradation ranges between two boundaries, lower or fine boundary and upper or coarse bounder as shown in Figure 12; thereafter will be referred to as LA II-coarse and LA II-fine. Since the range bounded by LA II-coarse and fine branches is relatively wide, the properties of each branch were evaluated individually in this study.

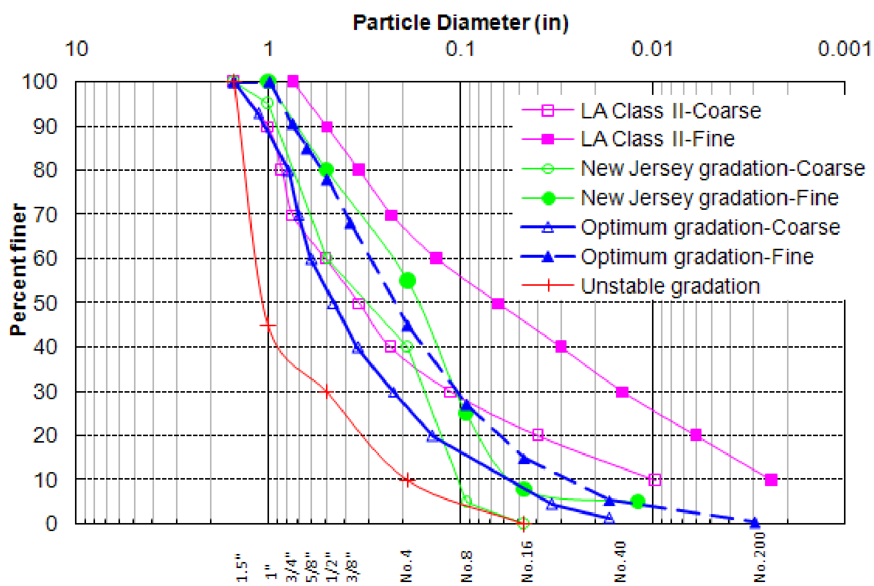


Figure 12: Particle size gradations of tested specimens

New Jersey permeable unbound aggregate gradation recommended by Federal Highway Administration (FHWA) is also included in Figure 12. Only the medium gradation within the New Jersey gradation range was studied for providing some reference information in the process of gradation optimization. After a series of trial-and-error tests, along with referring to reported results in the literature, an optimum gradation was identified, shown also in Figure 12. The range of modified optimum gradation per particle size is presented in Table 3. The criterion in optimizing gradation is based on two standpoints: 1) the permeability quantified by saturation hydraulic conductivity that should be equal to or larger than 1,000 ft/day; and 2) the relative structural stability compared with that of LA II gradation. The parameters related to particle size distribution, such as the coefficient of uniformity, coefficient of curvature, and fines content for these gradations are tabulated in Table 4. Unified soil classification and AASHTO classification for Mexican limestone with these gradations are also included in Table 4.

Table 3: Range of modified optimum gradation

Particle diameter (in)	Optimum-fine gradation	Optimum-coarse gradation
	(%) passing	(%) passing
1.5"	100	100
1"	100	86.5
3/4"	90.5	74
5/8"	85	64
1/2"	78	53
3/8"	68	44
No.4	45	25.3
No.8	27	14.7
No. 16	15	7.4
No. 40	5.5	1.5
No. 200	0.5	0

Table 4: Parameters related to particle size for different gradations

Gradation	C_u	C_c	ρ_{200} (%)	USCS/AASHTO
LA II-coarse	53.09	2.59	5.0	GW/A-1-a
LA II-fine	55.61	0.64	12.0	GW-GM/A-1-b
New Jersey medium	4.86	0.71	3.0	GP/A-1-a
Optimum-coarse	17.86	2.5	<1.5	GW/A-1-a
Optimum-fine	37.46	4.29	3.0	GW/A-1-a

Note: C_u = coefficient of uniformity; C_c = coefficient of curvature; ρ_{200} = percent of fines (passing through No. 200 sieve).

Its specific gravity is 2.54 and absorption is 5.7 percent. The fine portion (passing through No. 40 sieve) of Mexican limestone is found to be nonplastic. The standard Proctor compaction curves for Mexican limestone with the above gradations are shown in Figure 13. Among these gradations, New Jersey medium gradation had the lowest maximum dry density due to its relatively more uniform gradation. For LA II and optimum gradations, the fine branches had higher maximum dry densities than their coarse counterparts. The maximum dry densities for the optimum gradation were generally lower than those of LA II gradations. The optimum moisture contents and maximum dry densities for these gradations are summarized in Table 5, which were used to prepare testing specimens for other laboratory tests.

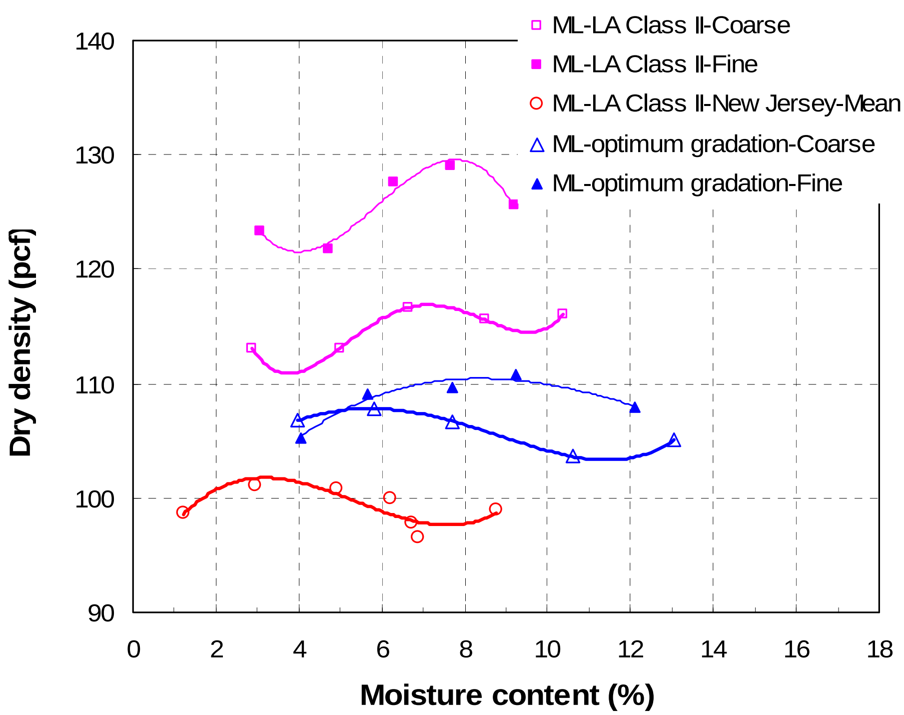


Figure 13: Standard Proctor compaction curves for Mexican limestone with different gradations

Table 5: Summary of optimum moisture content and maximum dry density for Mexican limestone with different gradations

Gradation	OMC (%)	MD (pcf)
LA II-coarse	6.6	116.5
LA II-fine	7.5	128.8
New Jersey medium	3.2	101.7
Optimum-coarse	5.8	107.8
Optimum-fine	8.5	110.7

Note: OMC = optimum moisture content; and MD = maximum dry density

Hydraulic Conductivity Test Results

A typical result from constant–head hydraulic conductivity tests is illustrated in Figure 14, with the slope of the curve representing hydraulic conductivity. Apparently, the relationship between flow rate, q, and hydraulic gradient, i, is nonlinear, especially at the larger hydraulic gradient. Nevertheless, the hydraulic gradient for pavement base layer is usually very small and water flows through pavement base layers under laminar flow conditions. Therefore, Darcy's law is valid for water flow within pavement base layers and the hydraulic conductivity in this study is determined at hydraulic gradients of the same order of magnitude anticipated in the field pavement base layers. The coefficient of hydraulic conductivity is calculated by using Darcy's law:

$$k = \frac{Q}{Ati} \tag{4}$$

Where: k = the coefficient of saturate hydraulic conductivity; Q = the volumetric flow; A = the bulk cross section area of the specimen; t = the time during which the volumetric flow Q is measured; and i = the hydraulic gradient.

The hydraulic conductivity coefficients for Mexican limestone with these gradations are summarized in Table 6. Among these gradations in question, all but LA II-fine branch met the permeability recommended by FHWA (1,000 ft/day). Also it can be noted that both branches of the optimum gradation have adequate permeability in terms of hydraulic conductivity coefficient.

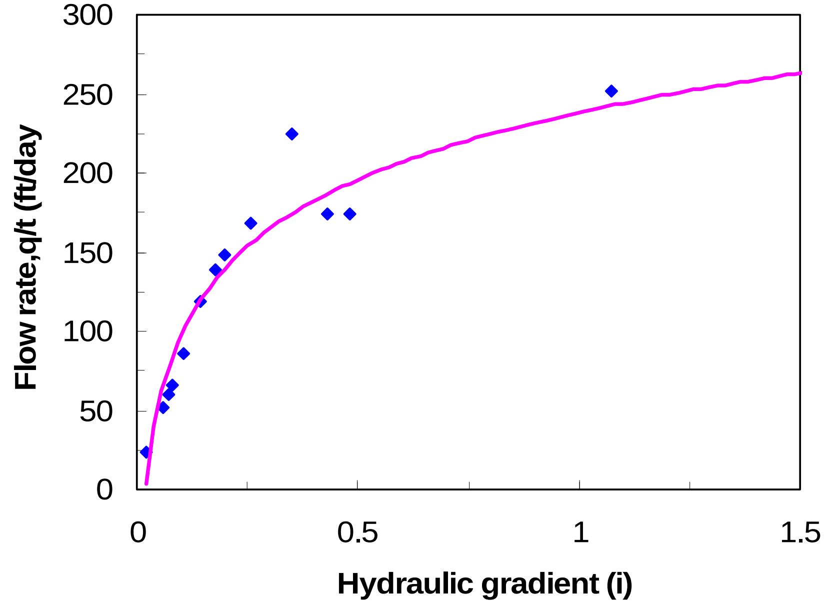


Figure 14: Relationship between flow rate and hydraulic gradient

Table 6: Summary of saturate hydraulic conductivity coefficient for Mexican limestone with different gradations

Gradation	Dry density[a]	Ks[b]
	(pcf)	(ft/day)
LA II-Coarse	116.0	2,278
LA II-Fine	129.0	151
New Jersey-Medium	104.0	2,837
Optimum-Coarse	107.6	3,369
Optimum-Fine	124.0	2,277

Note: [a] = compacted at the optimum moisture content; [b] = saturate hydraulic conductivity coefficient.

Tube Suction Test Results

Dielectric value of TS specimen is generally increasing as the test proceeds until a maximum value is reached. This maximum value is referred to as maximum DV, a value that gives an indication of the maximum free moisture content a specimen can absorb under capillarity suction. The maximum DVs for these gradations are shown in Figure 15, along with the threshold values suggested by Scullion and Saarenekto [*10*]. Based on their successful preliminary studies, Scullion and Saarenekto [*10*] proposed a maximum

DV criterion for assessing the quality of base materials: good base aggregates have the maximum DV of less than 10; marginal base aggregates have the maximum DV ranging from 10 to 16; and poor base aggregates have the maximum DV exceeding 16. Based on this criterion, Figure 15 indicates that New Jersey medium gradation is good; LA II-fine gradation is poor; and the rest of gradations are marginal, in terms of water susceptibility. This also suggests that the more fines content a gradation has, the larger the maximum DV is and, thus, the higher water susceptibility. Both capillary suction and mechanical compaction are closely related to the water affinity of an aggregate; That is, higher water affinity generally is accompanied with a higher maximum DV and a higher optimum moisture content. Therefore, possible correlation between the maximum DVs and optimum moisture contents is investigated by plotting them against each other. Figure 16 shows that there is a strong correlation between these two parameters. Such a correlation is of practical implication for screening pavement base materials, by providing a simple and quick approach to estimate water susceptibility of an aggregate. For example, in light of the regression relationship illustrated in Figure 16, an unbound aggregate is likely to be water susceptible with the optimum moisture content (based on the Standard Proctor test) exceeding 8.7 percent.

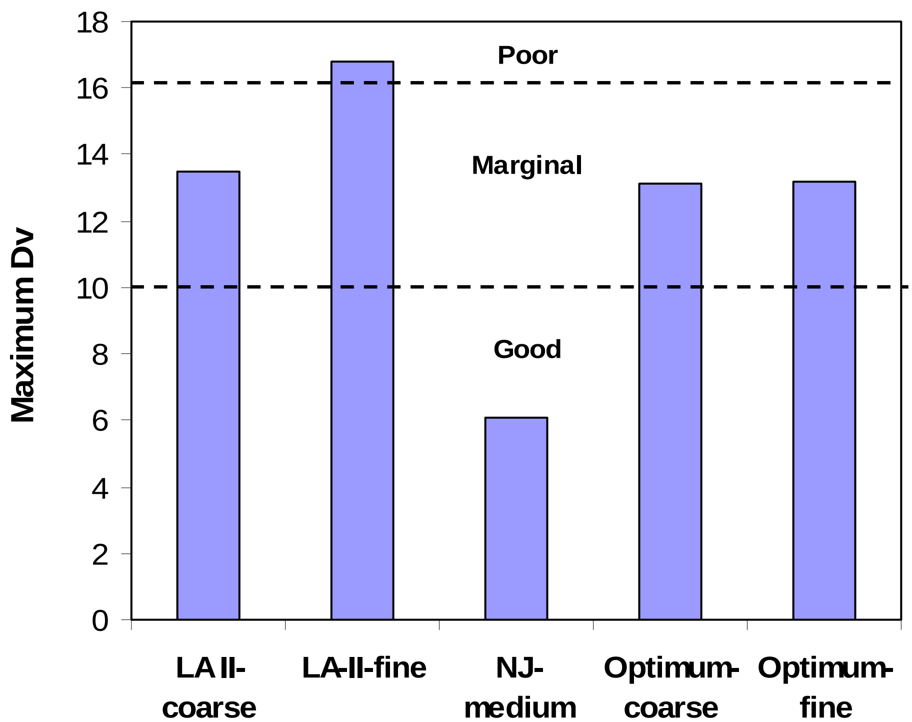


Figure 15: Maximum DV for Mexican limestone with different gradations

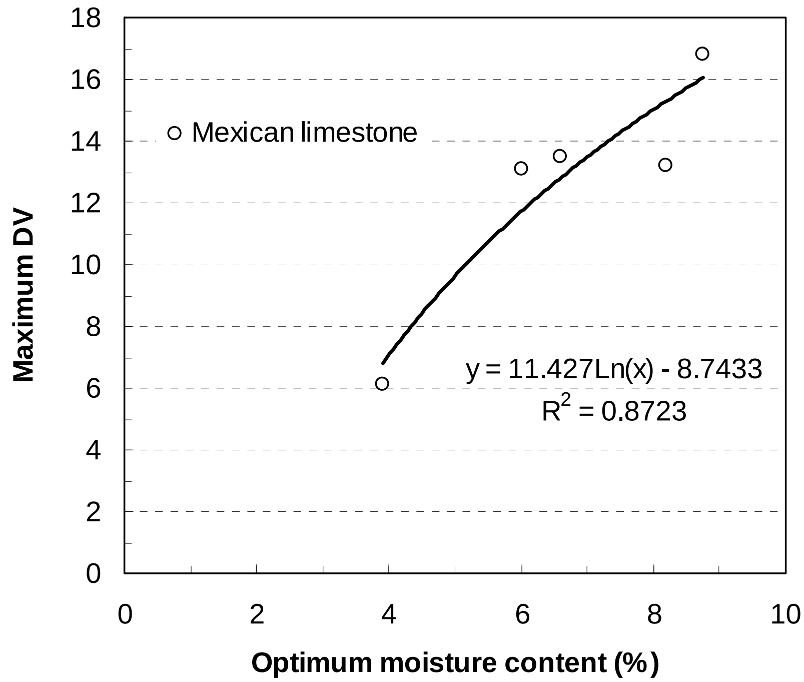


Figure 16: Correlation between maximum DV values and optimum moisture contents determined from Standard Proctor tests for Mexican limestone with different gradations

Results from Shear Strength and Structural Stability Tests

CBR Tests

All CBR specimens with different gradations were compacted at their respective optimum moisture contents and maximum dry densities as determined from the standard Proctor test. The CBR results at penetrations of 0.1 inches and 0.2 inches for tested specimens are plotted in Figures 17 and 18, respectively. The LA II-coarse and the optimum coarse gradation obtained much higher CBR values, compared to the other gradations; The CBR values at 0.2 inches penetration are also noted to be larger than those at 0.1 inches penetration. CBR values after soaking are generally larger than those tested in unsoaked conditions for all tested gradations except LA II course, which is contrary to what is expected for the influence of saturation on CBR. This observation may suggest that CBR may not be a good indication of the influence of saturation on shear strength of coarse materials, such as those tested in this study.

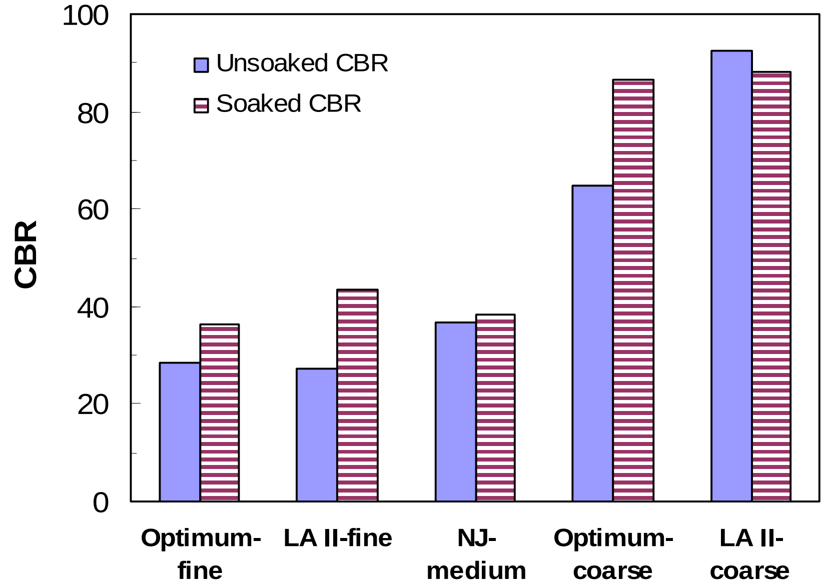


Figure 17: CBRs at 0.1 inches penetration for Mexican limestone at different gradations

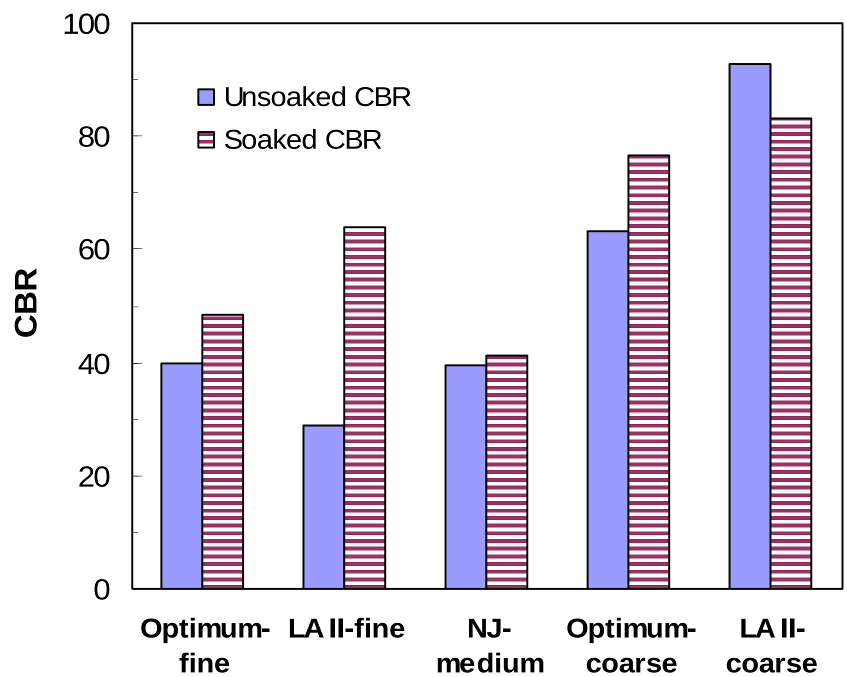


Figure 18: CBRs at 0.2 inches penetration for Mexican limestone at different gradations

DCP Results

DCP results are often represented by dynamic cone penetration index, DCPI, which is the averaged penetration per blow (mm/blow) over the thickness of the tested layer. A higher DCPI value generally implies weaker shear strength for a given aggregate. For a good aggregate base material, its DCPI is equal to or less than 3 mm/blow according to Louisiana DOTD Engineers' past experience. DCPIs for tested gradations are shown in Figure 19.

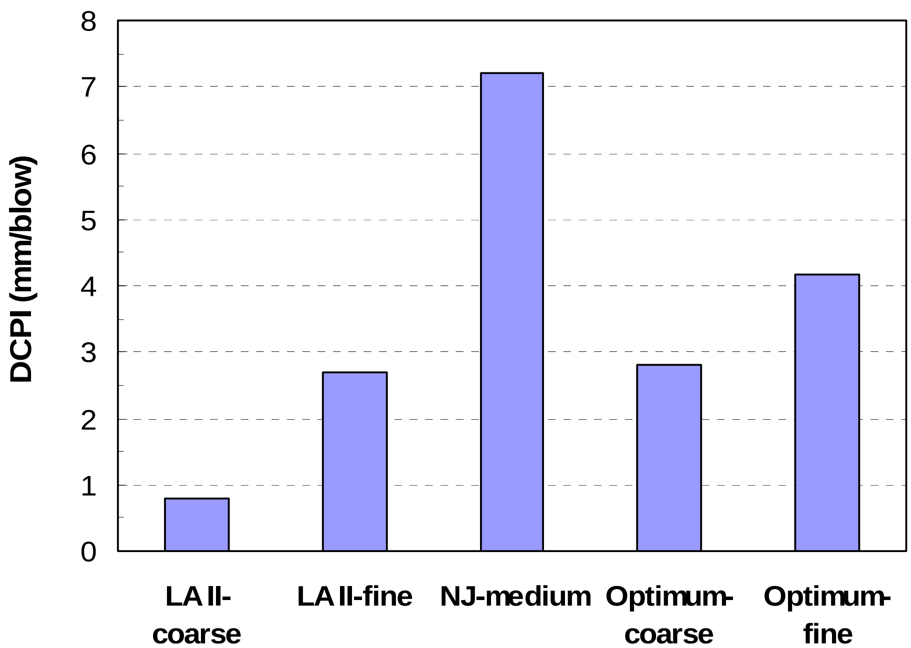


Figure 19: DCP result summary for Mexican limestone with different gradations

Monotonic Loading Triaxial Test Results

Monotonic loading triaxial tests were conducted on Mexican limestone specimens. The specimens were prepared at their optimum moisture contents and maximum dry densities as determined from the standard Proctor tests. Figure 20 presents the stress-strain curves obtained from the triaxial compression tests on the Mexican limestone specimens. The figures show that, at the optimum conditions, Mexican limestone has high shear strength. However, it experiences strain softening behavior post-peak shear strength, where the shear stress decreases with the strain increase until reaching a stabilized value, referred to as the residual shear strength.

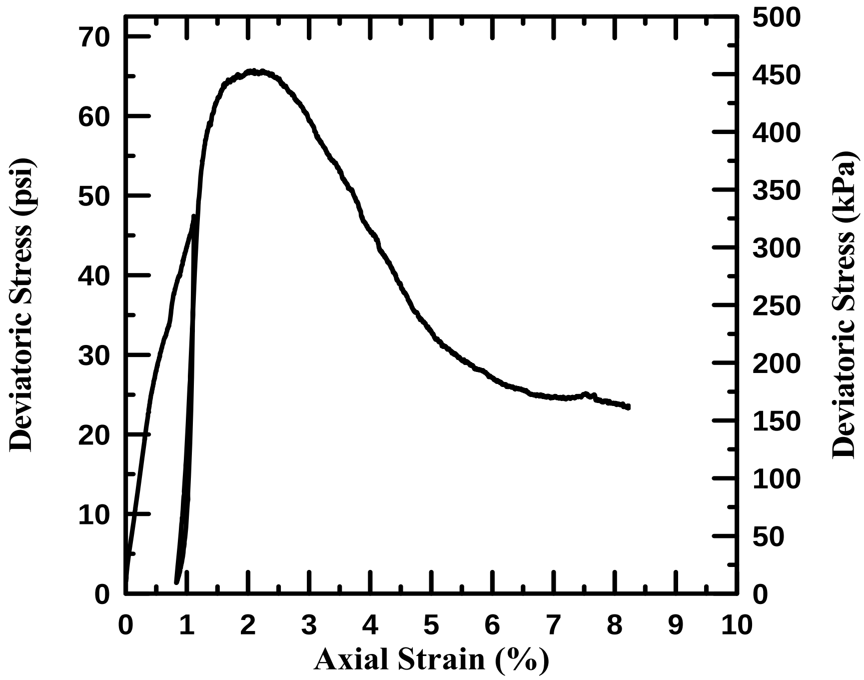


Figure 20: Results of Monotonic Loading Triaxial Test

RLT Test Results

All RLT tests were conducted on specimens at their respective optimum moisture content and maximum dry density as determined from the standard Proctor testing procedure. Both resilient modulus, M_r, and permanent deformation, ε_p, can be determined from RLT tests. Resilient modulus is a parameter to characterize stiffness of pavement materials under repeated loading, with the consideration of the influence of stress levels (both confining pressure and deviatoric stress) and the nonlinearity induced by traffic loading. Resilient modulus has been an essential input parameter in the current AASHTO empirical pavement design guide in selecting pavement layer thickness, receiving more attention in the upcoming AASHTO mechanistic-empirical pavement design guide. A typical RLT test result is depicted in Figure 21, with marked recoverable axial strain and cumulative permanent axial strain at a certain loading cycle.

Based on its definition, resilient modulus can be determined:

$$M_r = \frac{\sigma_d}{\varepsilon_r} \tag{5}$$

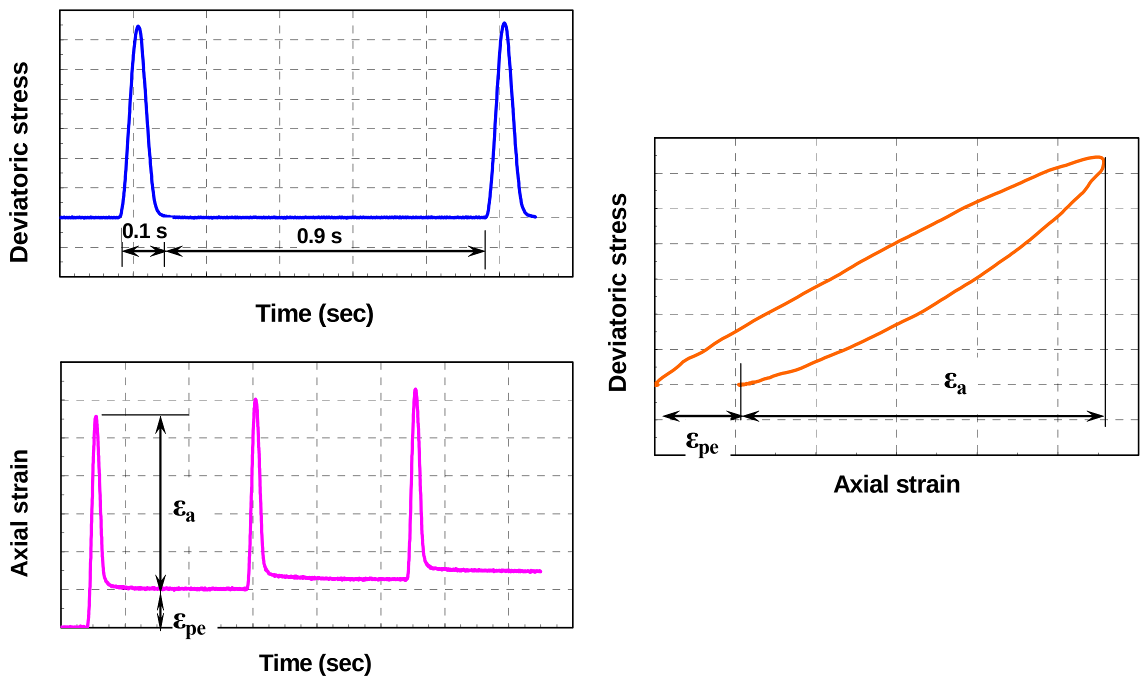


Figure 21: Typical results from a RLT test

And recoverable axial strain is calculated as:

$$\varepsilon_r = \frac{\delta_{r\_N}}{L_0\left(1-\varepsilon_{p\_(N-1)}\right)} \qquad (6)$$

Where σ_d = deviatoric stress; ε_r = recoverable axial strain; L_0= original length of a specimen; δ_{r_N} = recoverable deformation at the N^{th} loading cycle; and $\varepsilon_{p\_(N-1)}$ = the cumulative permanent axial strain at the $(N-1)^{th}$ loading cycle.

The resilient moduli for these gradations under investigation are shown in Figure 22. As expected, the resilient mouduli increased with confining pressures for all the gradations under investigation. However, the influence of deviatoric stress on resilient mouduli is less well-defined, which virtually depends on the gradation and the magnitude of confining pressures. To facilitate the use of resilient modulus data in the pavement design, especially in the new mechanistic-empirical pavement design, a generalized equation is recommended by NCHRP report 1-37A [*12*]:

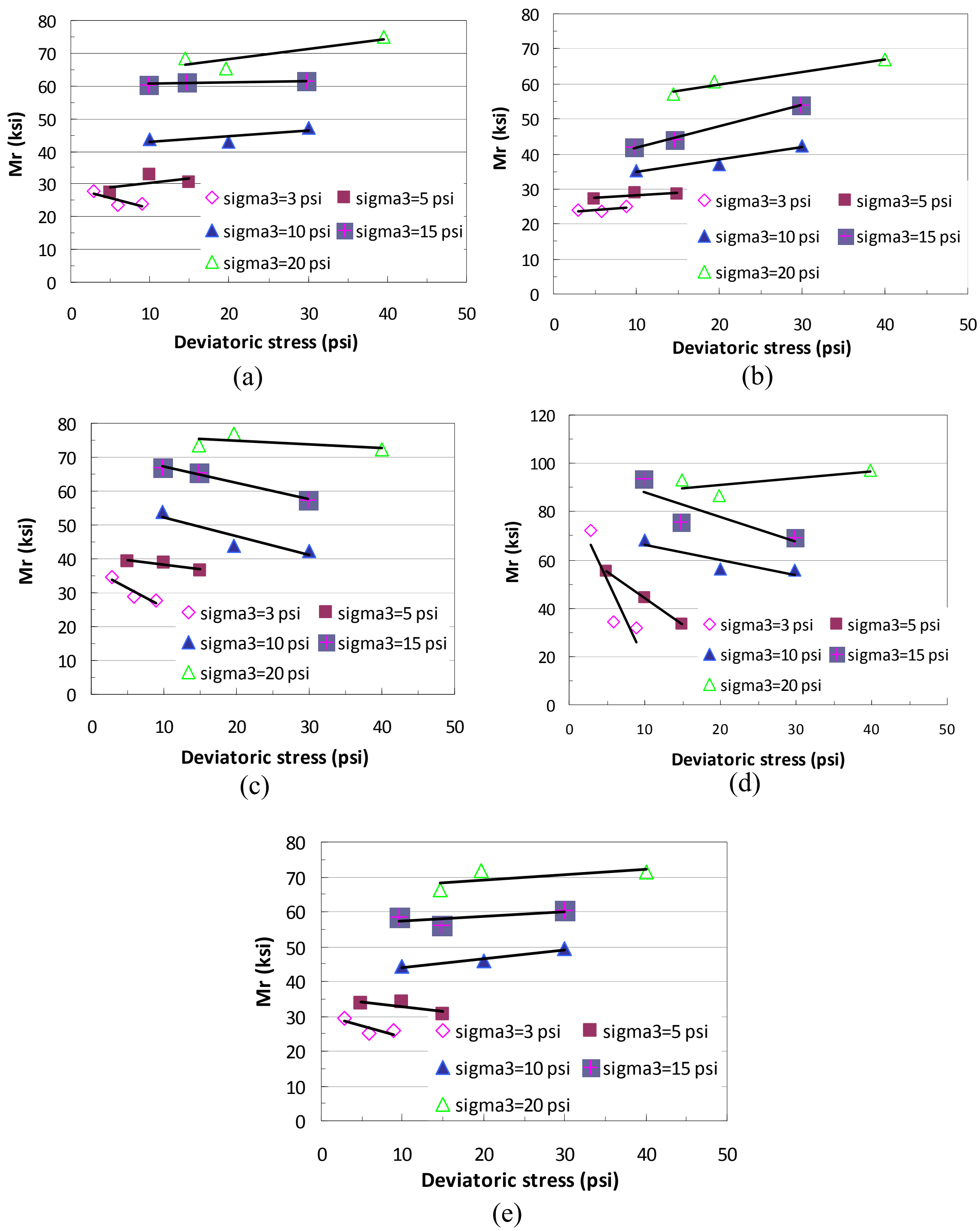


Figure 22: Resilient moduli of Mexican Limestone at various gradations at corresponding optimum compaction conditions: (a) LA II-coarse; (b) LA II-fine; (c) NJ-medium; (d) Optimum-coarse; and (e) Optimum-fine

$$M_r = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3} \tag{7}$$

Where:

M_r = resilient modulus, psi; θ = bulk stress= $\sigma_1 + \sigma_2 + \sigma_3$; σ_1, σ_2, and σ_3 = major, intermediate, and minor principal stress;

τ_{oct} =Octahedral shear stress= $\frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2}$; P_a = atmospheric pressure;

k_1, k_2, and k_3 = regression constants (obtained by fitting resilient modulus tests data to the equation).

By using the equation, regression constants were determined for each of the gradations, which are summarized in Table 7 and can be directly used in the MEPDG (Mechanistic-Empirical Pavement Design Guide) developed under NCHRP project 1-37A [*12*].

Table 7: Regression constants for different gradations

Material & gradation	k1	k2	k3	R^2
LA II-Coarse	1656.87	0.79	-0.66	0.95
LA II-Fine	1523.16	0.65	-0.32	0.92
New Jersey-medium	2346.08	0.72	-0.97	0.91
Optimum-coarse	3071.28	0.85	-1.59	0.93
Optimum-fine	1784.90	0.65	-0.31	0.96

Permanent deformation–For granular materials, both resilient and permanent (plastic) deformation occur under repeated loading, even at a small magnitude. Their resilient response has been characterized by resilient modulus and extensively studied. In order to fully characterize the behavior of a pavement granular material, its permanent deformation characteristic should also be truly understood since it is closely related to rutting distress often observed on flexible pavements. Since permanent deformation of a granular material depends on its stiffness, shear strength, magnitude of loading, and stress history and other factors, it is a parameter to reflect the performance potential of a material in field pavement conditions. As introduced in the previous section, the permanent deformation can be calculated from the laboratory RLT tests as follows:

$$\varepsilon_{p\_N} = \frac{\delta_{p\_N}}{L_0\left(1-\varepsilon_{p\_(N-1)}\right)} \quad (8)$$

Where: ε_{p_N} = the cumulative permanent axial strain up to the Nth loading cycle; δ_{p_N} = the cumulative permanent axial deformation up to the Nth loading cycle; and $\varepsilon_{p\_(N-1)}$ = the cumulative permanent axial strain by the end of (N-1) loading cycles.

The permanent deformations for tested gradations are summarized in Figure 23, with larger values associated with relatively finer gradations (e.g., LA II-fine and the optimum gradation-fine). It can also be noted that the optimum gradation had smaller permanent strain than that of LA II counterparts. The following power model is used to correlate the permanent strain with the number of load cycles:

$$\varepsilon_p = aN^b \quad (9)$$

Where: ε_p = permanent strain in 10^{-3}; a and b = regression parameters; and N = the number of load cycles.

Regression parameters, as well as the corresponding coefficients of determination for these different gradations, are tabulated in Table 8.

Table 8: Regression parameters of permanent strain model

Gradation	Model parameters		R^2
	a	b	
LA II-coarse	1.39	0.25	0.999
LA II-fine	3.56	0.29	0.992
New Jersey-medium	1.52	0.25	0.999
Optimum-coarse	1.24	0.24	0.999
Optimum-fine	4.96	0.20	0.969

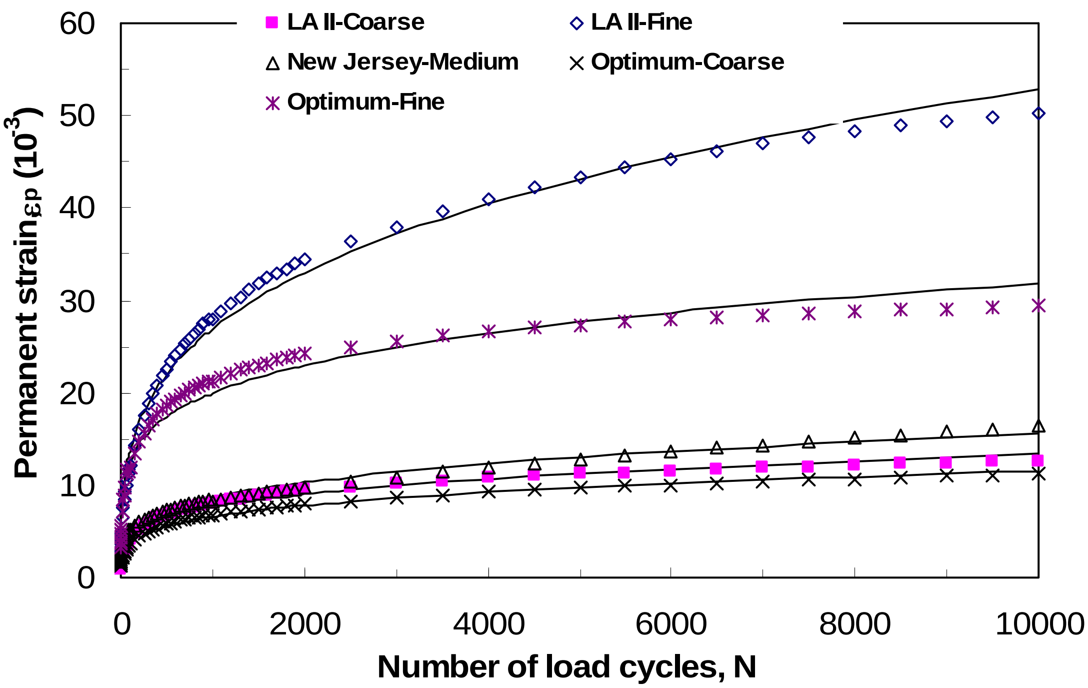


Figure 23: Variation of permanent deformation with number of load cycles for Mexican limestone specimens with different gradations

Besides the magnitude of permanent strain, the rate at which permanent strain developed over load cycles implies the accumulation trend of permanent strain in subsequent loading and thus provides further important information about structural stability of tested aggregates. Permanent strain rates for different gradations are illustrated in Figure 24 by plotting accumulative permanent strain versus permanent strain rate (10^{-3}/cycle). Figure 24 indicates that coarser gradations (optimum-coarse, LA II-coarse, New Jersey-medium) had a much smaller permanent strain rate at the end of the RLT tests, compared to two finer gradations (LA II-fine and optimum-fine). Compared to LA II-fine, the optimum fine gradation had a much smaller permanent strain rate and accumulative permanent strain. More importantly, the optimum fine gradation had a concave downward shape that implies more stable responses during subsequent loading, whereas LA II-fine had a concave upward shape that indicates less stable responses during subsequent loading [*13*]. Different behavior of permanent deformation obtained from this study implies that the optimum fine gradation had an improved structural stability compared to LA II-fine gradation.

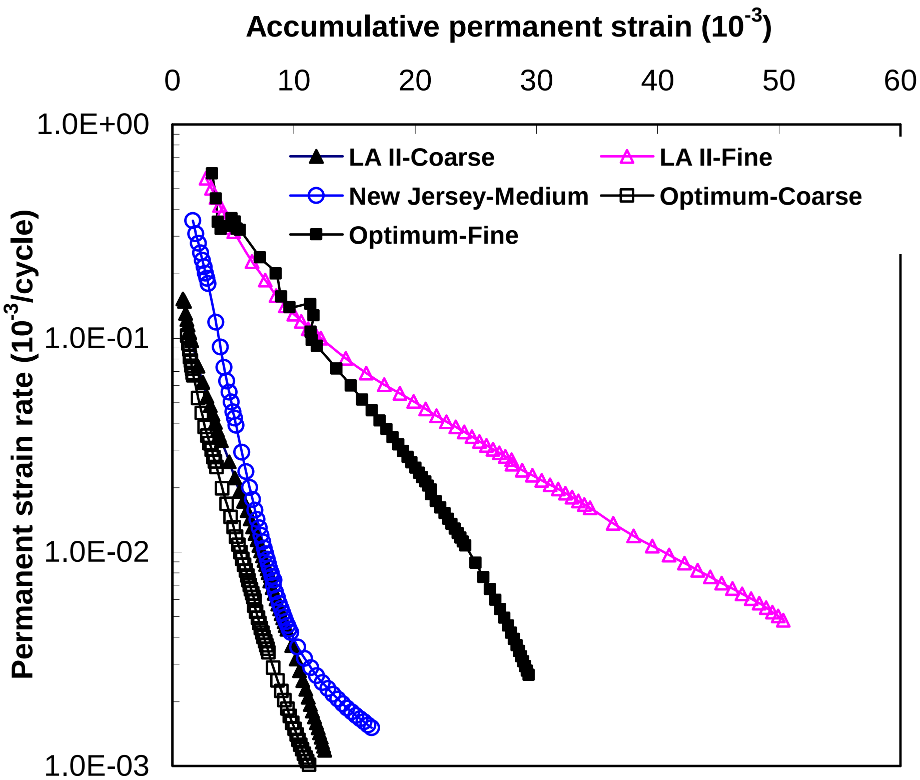


Figure 24: Variation of permanent strain rate with accumulative permanent strain for Mexican limestone specimens with different gradations

Correlations of Different Test Results

Although CBR and DCP tests are usually employed to characterize the structural stability of pavement materials, the appropriateness of these tests are of concern because there is a lack of information linking these test results with the field performance of pavement materials (such as rutting and cracking). The following section is devoted to comparing possible correlations among simple DCP and CBR tests with the RLT tests that more realistically represent responses of pavement materials in the field conditions.

Possible correlations between DCP values and other tested properties are examined in Figure 25, by plotting DCPI against unsoaked CBR at 0.1 inch, resilient modulus, and permanent deformation strain. Among these relations shown in Figure 25, only DCPI versus resilient modulus had a significant correlation, with their coefficients of determination R^2 larger than 0.7; whereas, there was no strong correlation between DCPI and unsoaked CBR or permanent deformation strain.

Other correlations among CBR, resilient modulus, and permanent strain are illustrated in Figure 26. There is a strong correlation between unsoaked 0.1 inch CBR and permanent strain, as illustrated by their coefficients of determination. No significant correlation exists between resilient modulus and permanent strain, which suggests that resilient modulus alone, may not provide full characterization for pavement materials. However, weak correlations between resilient modulus and CBR (at 0.1 inch), or permanent strain and resilient modulus, could have been attributable to a statistical outlier (New Jersey-medium gradation in both cases). Thus, more tests on other aggregates are required to confirm or refine the observations from this study.

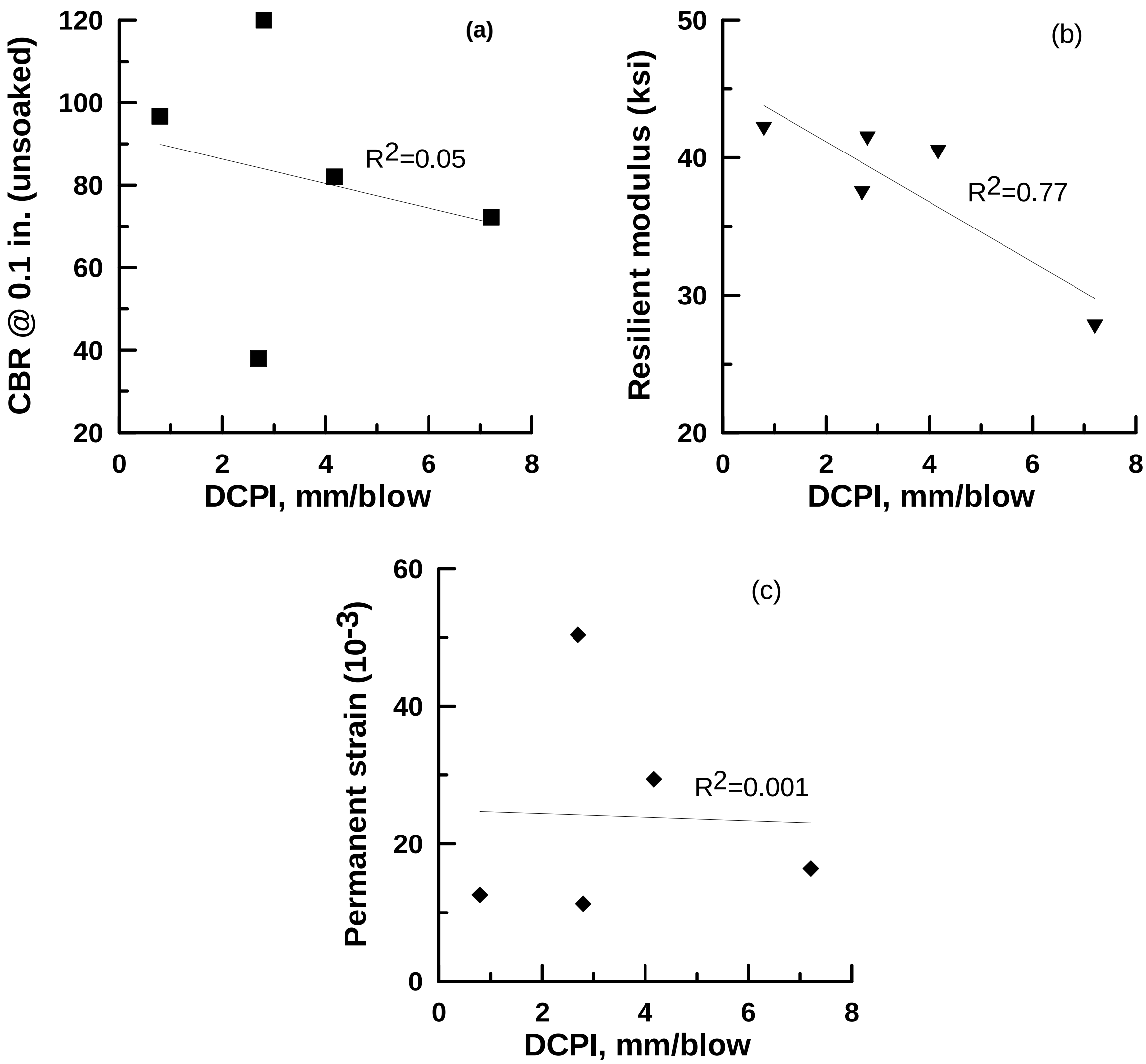


Figure 25: Correlation between DCPI and other properties

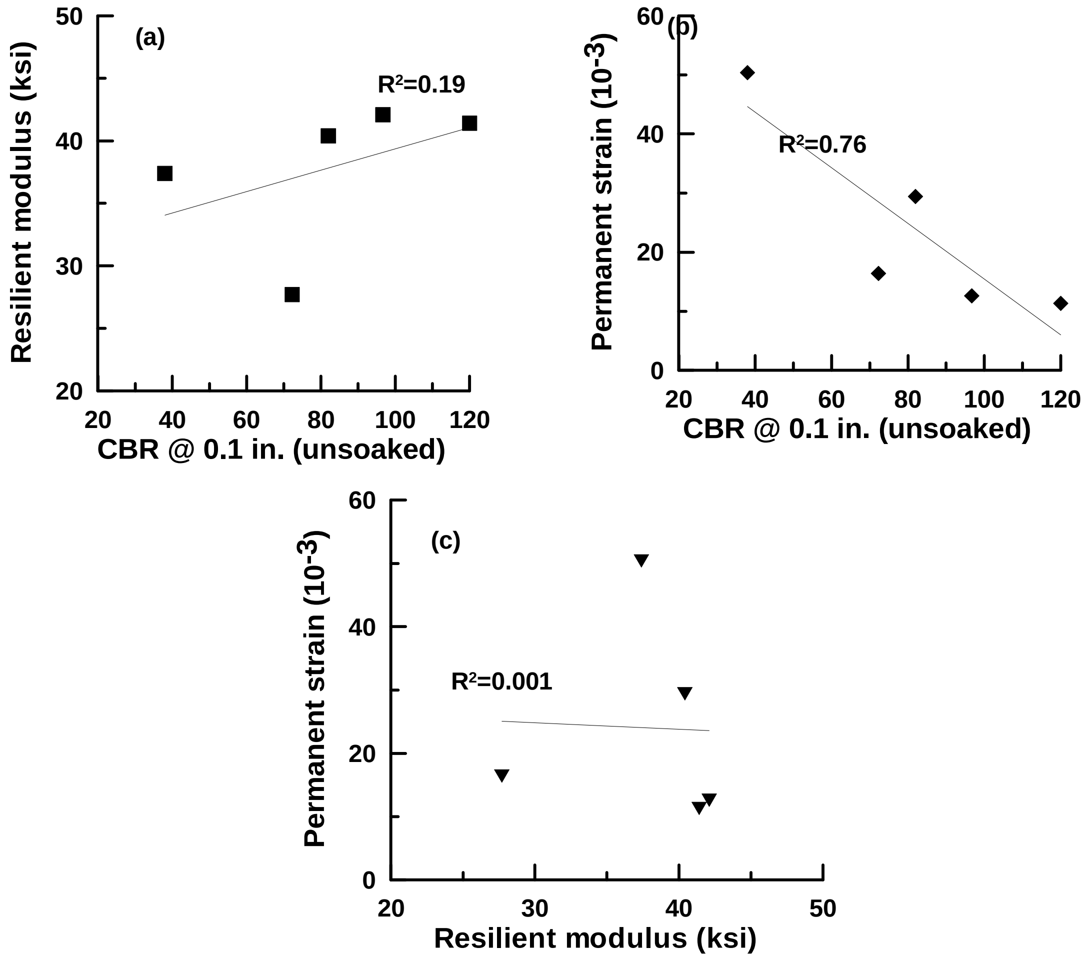


Figure 26: Correlation among CBR, resilient modulus, and permanent strain

To elucidate the influence of particle size gradation on shear strength and stiffness of the aggregate, a ranking system is given to the different gradations based on the laboratory test results, as summarized in Table 9. The different gradations are ranked in terms of their corresponding CBR, DCP, resilient modulus, permanent deformation magnitudes, and hydraulic conductivity, with A representing the best performance and E representing the worst performance. Apparently, different rankings are rendered by following different testing procedures. However, for the overall ranking, more weight should be given to laboratory tests that are directly related to the long-term performance of the Mexican limestone base layer, e.g., resilient modulus for structural stability, permanent deformation for pavement rutting distress, and hydraulic conductivity for base drainability.

Table 9: Ranking of different gradations as drainable base on the basis of stability (shear strength and stiffness) and permeability

Gradation	CBR unsoaked	CBR soaked	DCP	Mr	ε_p	K_s	Overall rank
LA II-coarse	A	A	A	B	B	C	B
LA II-fine	C	E	B	E	E	E	E
NJ-medium	E	C	E	B	C	B	C
Optimum-coarse	B	B	C	A	A	A	A
Optimum-fine	C	C	D	D	D	C	D

Note: Mr = Resilient modulus; ε_p = permanent deformation; K_s = hydraulic conductivity coefficient

CONCLUSIONS

A laboratory testing program was conducted to optimize the gradation of Mexican limestone aggregate material to satisfy both permeability and stability criteria needed for use as a drainable base in pavement structures. Based on this study, the following conclusions can be drawn:

- All gradations except LA II-fine achieved the permeability criterion recommended by FHWA, which is 0.35 cm/sec. (1,000 ft/day).

- The LA II-coarse and optimum-coarse gradations achieved higher shear strength and resilient modulus values; while the NJ-medium and LA II-fine had lower strength and modulus. The optimum-fine gradation had intermediate strength and stiffness values. This finding suggests that neither a very uniform gradation nor one with excessive fines content will perform well as a pavement base material.

- Overall, the Mexican limestone base material has gained strength when compacted at optimum moisture contents and maximum dry density. However, it experiences strain-softening behavior, post-peak shear strength, in which the shear stress decreases with the strain increase until reaching the residual shear strength.

- The results of the tube section tests showed that the LA II-fine gradation of Mexican limestone has a relatively high dielectric value (an indication of free moisture content absorption under capillarity suction) and, thus, can be considered as a poor base in terms of water susceptibility. The rest of the gradations are considered in general marginal bases. This result suggests that, the more fines the gradation has, the larger the DV is and, thus, the higher the water susceptibility, which can adversely affect the performance of the Mexican limestone base layer. The results of another research study [*14*] showed that the strength, stiffness, and permanent deformation of Mexican limestone base material are very sensitive to the moisture content.

- There is a strong correlation between the optimum moisture content determined from the Standard Proctor test and the maximum dielectric values [$DV = 11.4 \ln(w_{opt}) - 8.7$], with a coefficient of determination $R^2 = 0.87$. Such correlation provides a quick approach to predict water susceptibility of unbound aggregates as pavement base materials.

- The results of repeated loading triaxial tests showed that coarser gradations (optimum-coarse, LA II-coarse, New Jersey–medium) had much smaller permanent deformation and strain rate compared to finer gradations (optimum-fine, LA II-fine). However, the optimum-fine gradation had less permanent and strain rate, hence, an improved structural stability compared to LA II-fine gradation.

- Different performance rankings were obtained for the aforementioned gradations, based on the results of different structural stability tests and the permeability requirement. However, more weight should be given to laboratory tests that are directly related to the long-term performance of the Mexican limestone base layer, e.g., resilient modulus for structural stability, permanent deformation for pavement rutting distress, and the hydraulic conductivity for base drainability. Accordingly, the optimum-course gradation will be ranked A and LA II-fine gradation will be ranked E. The reader should realize that these rankings are based merely on laboratory tests that need to be validated and/or correlated with field test data.

- Good correlations exist between the DCPI and resilient modulus and between the unsoaked CBR and permanent strain. However, no significant correlations exist between the DCPI and unsoaked CBRs, between the DCPI and permanent strain, and between the resilient modulus and permanent strain.

- The results indicate that there is a narrow range of variation between the optimum-coarse and the optimum-fine aggregate gradations that would provide a stable and drainable pavement base layer with improved performance as compared to LA class II gradations. This narrow acceptable drainable range might be difficult to achieve in the field. Achieving this gradation in the field would require running a pug-mill on the site and a spreader (paving machine) to lay the material. This additional handling by the material suppliers and the contractor, is estimated to cost $18.50 per cubic yard (or ≈25%) over the current bid prices.

RECOMMENDATIONS

Based on the results of this research study, we recommend that DOTD consider using the proposed optimum gradation range if an unbound drainable base is used. In this situation, the field test sections should be built to verify/validate the results of this study. Since the range of gradation between optimum-coarse and optimum-fine that satisfy the permeability and stability requirements is narrow and might be difficult to achieve in the field, a pug-mill is needed to run on the site and a spreader to lay the material. This will be associated with an estimated extra cost of \$18.50 per cubic yard (or ≈25%) over the current average cost.

The laboratory test results indicated that the finer the Mexican limestone gradation is, the higher its water susceptibility and the weaker the base material is in terms of strength, stiffness, and permanent deformation. Therefore, it is recommended that DOTD consider tightening the specification of fine gradation side by moving toward the coarse side to achieve a better long-term performance.

With respect to the issue of drainable base, we recommend considering stabilized open-graded aggregates with very high permeability. To ensure its long-term stability, geogrids, asphalt or cement can be used to stabilize the open-graded base material. A research project is therefore needed to evaluate the most efficient method for stabilizing the open-graded, drainable base material, and to study the strength, stiffness, and permanent deformation of open-graded stabilized specimens.

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