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Evaluation of the Suitability of Local Materials for Pavements' Subbases; A Saudi Case Study

Moustafa A. Kamel[a,b]
[a] Assistant Professor, Civil Eng. Dept., College of Engineering, Taif University, Taif, 21974, KSA
[b] Permanent Address: Public Works Dept., College of Engineering, Mansoura University, Mansoura, ٣٥٥١٦, Egypt
Email: mostafakamel2000@yahoo.com

:الملخص

يستخدم الركام فى طبقات الأساس والأساس المساعد لطبقات الرصف المرن ونظرا للتكلفة العالية وأحيانا لندرة الأحجار الطبيعية التى تصلح للاستخدام مواد فى طبقات الرصف كان من الضرورى البحث عن بدائل أخرى لتقليل تكلفة إنشاء الطرق وكذلك الحفاظ على المصادر الطبيعية للركام لذلك كان من الضرورى البحث عن مواد طبيعية محلية رخيصة تصلح للاستخدام فى طبقات الرصف. ومن هذا المنطلق فيقوم هذا البحث على تقييم مواد طبيعية رخيصة ومتاحة ومتوافرة بمنطقة الدراسة (مدينة الطائف – المملكة العربية السعودية) ومدى ملائمتها للاستخدام فى طبقات الأساس المساعد فى قطاعات الرصف. وبناء عليه فقد تم إختيار مخلفات كسارات الأحجار الطبيعية والمواد المتوافرة فى قاع الوديان ومخرات السيول المتواجدة بكثرة فى منطقة الدراسة بالاضافة إلى الرمال الخشنة المتوافرة فى مساحات شاسعة بمنطقة الدراسة. وقد تم إجراء إختبارات الروتينية لتصنيف تلك المواد وكذلك إختبارات نسبة تحمل كاليفورنيا والتحميل الثلاثى الاستاتيكى والمتكرر عند ضغط محاط متنوع بالاضافة إلى إختبار التحميل الثلاثى المتكرر. وقد بينت النتائج أن المواد الثلاثة المختارة تصلح للاستخدام كطبقة أساس مساعد فى قطاع الرصف حيث حققت جميعها متطلبات تلك الطبقة طبقا لمواصفات الأشتو.

Abstract

Three locally available materials in Taif City, KSA namely, aggregates' crushers' Dust, Materials collected for the vallies' beds and Coarse sand are selected. These materials were tested under CBR, Static Triaxial under different confining pressures and Repeated Triaxial tests under varying deviator stresses, to evaluate their suitability as sub bases for flexible pavements. Results revealed that all selected materials were suitable to be used in sub base layers for flexible pavements since, all materials satisfy the strength and durability requirements of sub bases.

Keywords: Flexible Pavement; Subbase; Repeated Triaxial; Local Materials

1. INTRODUCTION

The quality of pavement design is greatly dependent on the accuracy and manner in which the material properties are evaluated. Aggregates form the greater part of the pavement. Layers of aggregates bear the main stresses occurring in the road and resist wear from surface abrasion. Also, the pavement structure response under load is very sensitive to the base/subbase properties as these materials dilate under shear. Therefore, a realistic characterization of base/subbase materials is needed for the success of the pavement design especially, for the mechanistic approach, Praveen Kumar et. al, 2006.
This means adequate stress-strain relationships are required for the purposes of analysis

and information is also needed on failure criteria and the development of permanent strain under repeated loading, which contributes to the development of surface ruts. Uzan, 1999 specified some reasons for the difficulty of the characterization of granular base and subbase materials like, the response of granular materials is strongly nonlinear. Under shear, they exhibit volume dilatancy in both resilient and total deformation.

Flexible pavement layers transmit the vertical or compressive stress to the lower layers by grain-to-grain transfer through the points of contact in the granular structure. A well-compacted granular structure consisting of well-graded aggregates can transfer the compressive stresses through a wider area and thus form a good flexible pavement layer. The subbase layer is provided between the subgrade and the pavement to serve one or more of the following purposes; providing uniform support; increasing the supporting capacity above that provided by the subgrade soil; minimizing the detrimental effects produced by the subgrade soil; minimizing or eliminating the detrimental effects of frost action and preventing pumping, Huang, 2004. Therefore, subbase materials should have greater stability and bearing power, better capability to drain accumulated water and less susceptibility to volume changes and frost action than the subgrade. The sub-base or base courses of granular water bound layers with materials such as crushed rock, crushed aggregates, riverbed material, gravel, sand, soil-aggregate mix do this work well. On the other hand, the scarcity and high costs of granular materials make it difficult to provide thicker granular layers in pavement structures. Therefore, it is necessary to think of better alternatives to reduce the construction costs and to save natural resources of aggregates. For this concern, the idea of using locally available materials in the subbases has existed for years.

Many researchers around the world concentrated efforts on to understand the use of naturally and artificial material in subbase layer in pavement construction. Sufficient research has been carried out in stabilizing the subbase materials by mixing two or more suitable materials to increase their strength. But very limited work has been carried out to understand the deformation characteristics of these materials under repeated triaxial loading. Introduction of local materials in road construction requires complete understanding of their behavior under static and repeated loading. Cocks et. al, 2016 conducted an extensive experimental work to support the engineering judgment of using naturally occurring materials for pavements in western Australia. Kazmee and Tutumlure, 2015 developed a characterization technique for local materials jn terms of source, composition, and particle size and shape properties for using as pavement subgrade/granular subbase. In terms of rutting performance evaluation, they found some materials to be beneficial for use over weak subgrade. Gautum et. al, 2009 recommended some guidelines and test protocols for the use of local materials for roadway bases and subbases. Sumit and Malik, 2016 carried out an experimental study on use of locally available materials for pavements. They recommended optimal proportions of some stabilizers to be added to some local materials that will lead to higher values of unconfined compressive strength as well as CBR values.

EXPERIMENTAL PROGRAM

1.1. Materials Selection

Three different local materials collected from locations close to Taif city, KSA with varying properties were selected for this study. These materials are aggregate crushers dust, materials collected from the bed of valleys and coarse sand. These materials were hereafter designated as materials A, B and C respectively. Classification and all necessary routine tests were conducted for this issue. Table 1 shows the engineering properties of materials A and B.

Table 1: Classification of Selected Materials

Material Type	Aggregate crushers dust	Coarse Sand
Material Designation	Material A	Material C
Uniformity Coefficient (Cu)	9.32	8.89
Coefficient of Curvature (Cc)	0.905	1.29
Specific Gravity (SG)	2.24	2.35
Maximum Dry Density (MDD) (kN/m^3)	20.43	21.65
Optimum Moisture Content (OMC) (%)	8.62	10.2
Classification as per AASHTO	A3	A-1-b

Material B is an aggregate material collected from the bed of valleys. The maximum size of aggregate was kept 20 mm. Therefore, a grading was adopted for 20 mm maximum particle size using the parallel curve technique, HMSO, 1978. The gradation adopted for this material is shown in the Table 2 where, Fuller's equation was used to obtain maximum dry density, Chandra et al, 2000.

Table 2: Gradation of Material B for further tests

Sieve Size (mm)	Percent Passing	
	Grading as per parallel technique	Grading as per Fuller's equation
20	100	100
10	67	71.5
2.36	32	38
0.075	9	8

MDD and OMC were also determined for material B where, values of 21.8kN/m3and 6% were achieved for both respectively.

1.2. CBR Tests

CBR tests were conducted as per ASTM D1883 (ASTM 1999) on the selected materials in order to check the suitability of these materials for subbase course in pavement systems.

1.3. Static Triaxial Test

Loads applied to a pavement in service are transient and it is doubtful whether any drainage takes place during the loading cycle therefore, quick undrained test should govern the design of pavements, Yoder, 1978. So, unconsolidated undrained static triaxial tests were conducted as per ASTM D2850 (ASTM 2003) on all selected materials to study their stress-strain behaviour and to determine the modulus of elasticity (E-value) as an indicator of the material strength and subsequently, its suitability for subbase course. E-value was considered as the initial tangent of the stress-strain curve. For this concern, cylindrical specimens of size 100 mm diameter and 200 mm height were prepared. The choice of confining pressure was very difficult as there is no field evidence available so; the tests were conducted at four different confining pressures of 50, 80, 100 and 140 kPa.

1.4. Repeated Triaxial Test

A pavement system is normally subjected to a series of stress applications and release in the form of pressure pulses. Such a loading is called a repeated or cyclic load. Simultaneously, road materials do not behave elastically in the sense that a comparatively well-defined elastic range is followed by permanent deformation and failure, Huang, 2004. Therefore, it is important in measuring the elastic behaviour, deformation and fatigue properties of road materials that the method of test should model as closely as possible the actual loading conditions within the pavement as well as the environment in which it will operate. In this aspect, cyclic triaxial tests provide the most reliable method for determining the modulus of pavement materials and also for studying the deformation characteristics of such materials under repeated stress. So, the cyclic triaxial tests were conducted as per ASTM D5311 on all selected materials. Therefore, samples were prepared in the same manner described earlier for static triaxial tests. On the other hand, the repeated compressive deviator stresses were applied at three different confining pressures 50,80 and 140 kPa. The deviator stress to be applied for a particular confining pressure was taken 50% or less of failure stress in static triaxial tests so, a vertical stress of 175 kPa was selected which leads to three different deviator stresses. The frequency of load repetitions in all the tests was 70 cycles per minute. The repeated loads were applied up to 10,000 cycles of load application and behavior of various parameter such as resilient modulus, permanent strain, resilient strain were observed at different load repetitions. Axial deformations were measured by the linear variable differential transducer (LVDT) at different number of load repetition. For estimating recoverable deformation, specimens were kept free of deviator stress till the needle of LVDT recorder was stabilized. This recoverable deformation can simply be found by the difference of reading of recorder with applied deviator stress and on removal of it when needle of recorder stabilized.

2. RESULTS AND ANALYSIS

2.1. CBR Values

The results obtained from CBR test are given in Table 3. The CBR test results obtained are the average of three tests conducted on each material.

Table 3: CBR Values of the Investigated Materials

Material	A	B	C
CBR (%)	38.25	53.46	44.87

It is clear that the CBR of all selected local materials satisfies the requirements of AASHTO for subbase course which required a typical value of CBR ranging from 28 to 51%. Moreover, valley bed materials (material B) have the highest CBR value and crusher dust (material C) has the lowest CBR value.

2.2. Stress-Strain Behavior

The results of static triaxial tests were plotted in a typical form of stress-strain curves for all tested materials under different confining pressures. These curves were used to determine the deviator stress and strain at failure and also to determine the modulus of elasticity of soil. The deviator stresses and axial stains at failures for all tested materials are shown in Figures 1 and 2 respectively.

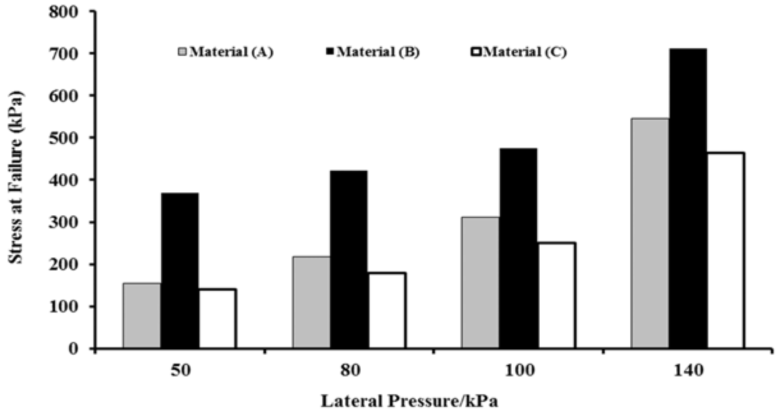

Figure 1: Stresses at Failure for All Tested Materials under Different Lateral Pressures

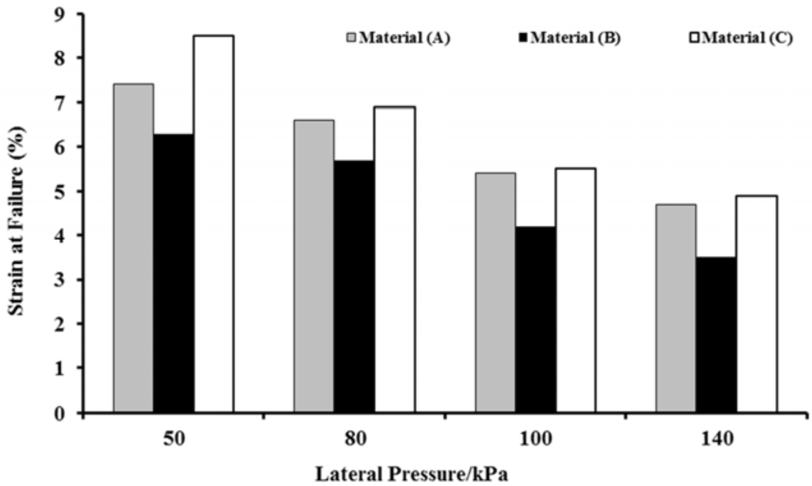

Figure 2: Strains at Failure for All Tested Materials under Different Lateral Pressures

It is clear that failure stress increases with the increase of the confining pressure while, the strain at failure goes on decreasing as the confining pressure increasing. This means that as the confining pressure increases, the lateral buckling decreases. Material B has gained the high strength since, it got the maximum failure stresses. On the other hand, the modulus of elasticity (E-value) of each material under prevailing conditions was calculated corresponding to the initial tangent of the stress-strain curve. It was observed that the modulus of elasticity of all materials increases with the increase in confining pressure which is quite expected. It was observed that, the valley bed materials (material B) attains the highest E-values, moreover, the coarse sand (material C) attains E-values higher than those of aggregates crushers dust (material A). A linear correlation between the confining pressure (δ_3) and the modulus of elasticity was tried for all materials. Equations 1 to 3 show these relationships.

$E(material\ A) = 0.77*\delta_3+7.37$ (1)

$E\ (material\ B)=0.47*\delta_3+56.26$ (2)

$E(material\ C) = 0.63*\delta_3+23.45$ (3)

Where: E in (MPa) and (δ3) in kPa

2.3. Resilient Strain Analysis

The conducted repeated triaxial tests on all materials showed that the resilient strain increases with the increase of the number of load cycles and deviator stresses while, it decreased with the increase of confining pressure as shown in Figures 3 to 5. This behavior is quite reasonable and supported by literature also. As may be seen, resilient strain was almost constant up to 100 cycles for all materials and then increases gradually. In addition, material B attains the highest resilient response of all tested materials.

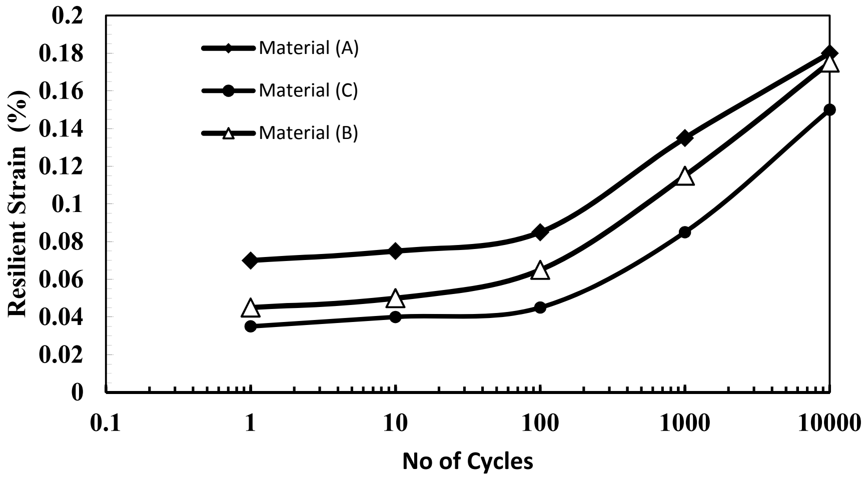

Figure 3: Resilient Strain at Deviator Stress of 125 kPa (Lateral Pressure=50kPa)

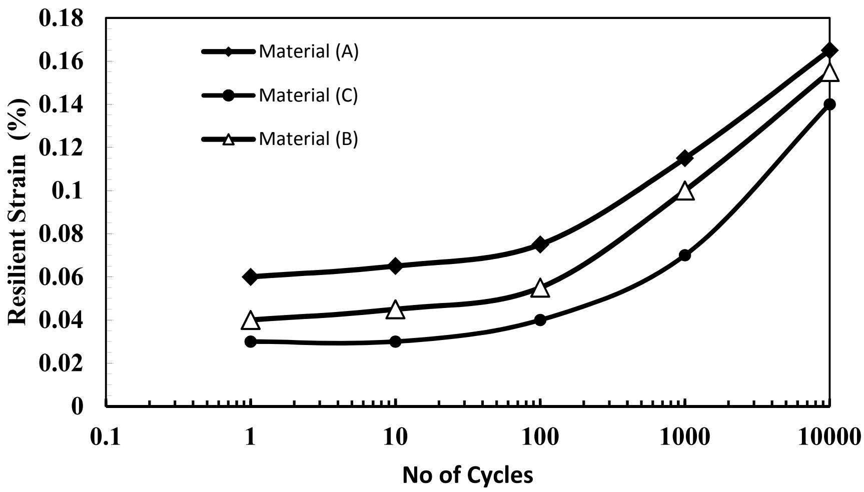

Figure 4: Resilient Strain at Deviator Stress of 95 kPa (Lateral Pressure=80kPa)

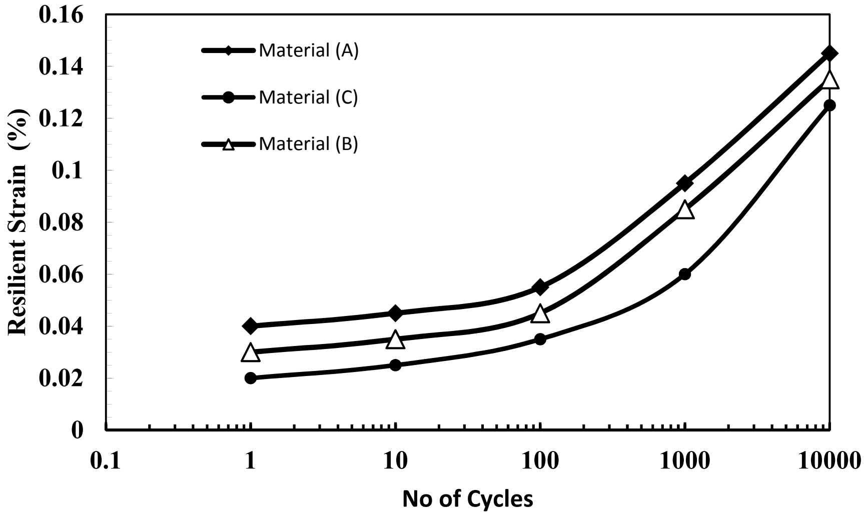

Figure 5: Resilient Strain at Deviator Stress of 35 kPa (Lateral Pressure=140kPa)

2.4. Permanent Strain Analysis

The observed permanent strains of all tested materials were found to be related directly to the deviator stress and inversely to the confining pressure. The shown curves through figures 6 to 8 indicate that the increase in permanent strain is very marginal up to 100 load cycles. In few cases, this continues till 1000 load cycles. This is followed by a sudden increase in the permanent strain values after 1000 load cycles for most of the cases in all materials. But, the general trend is that the permanent strain increases with the number of load cycles. That is quite expected as the growth of permanent strain under repeated loading is a gradual process during which each load application contributes a small increment to the accumulation of strain.

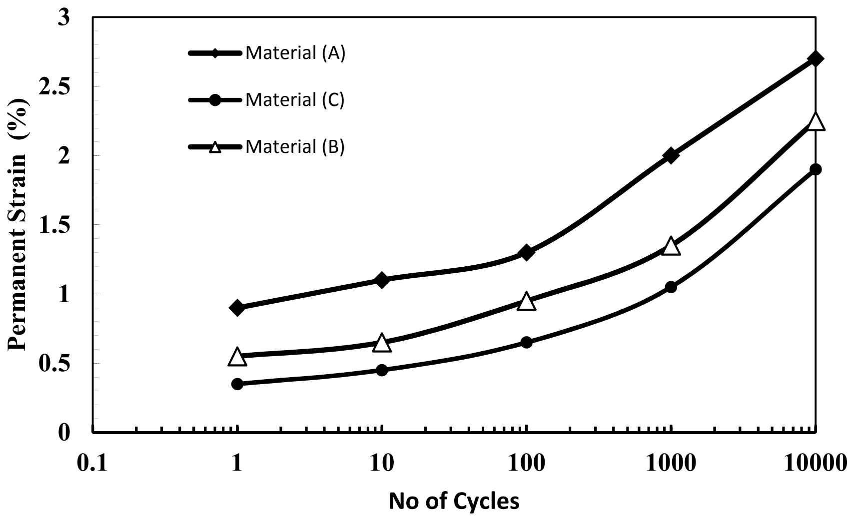

Figure 6: Permanent Strain at Deviator Stress of 125 kPa (Lateral Pressure=50kPa)

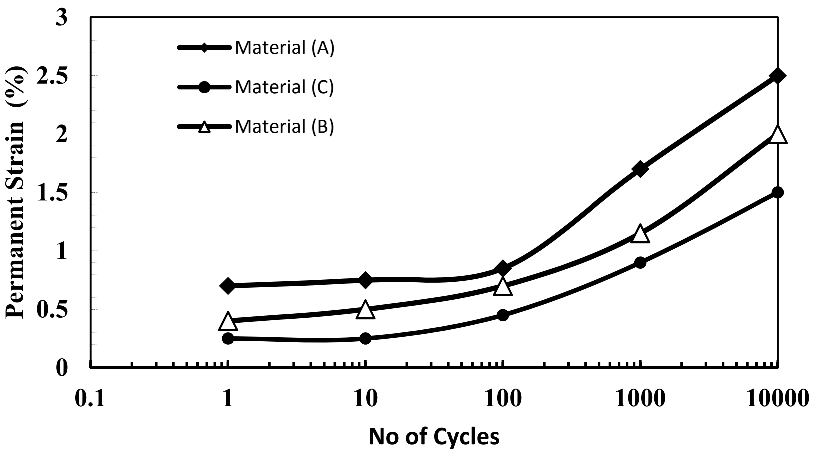

Figure 7: Permanent Strain at Deviator Stress of 80 kPa (Lateral Pressure=95kPa)

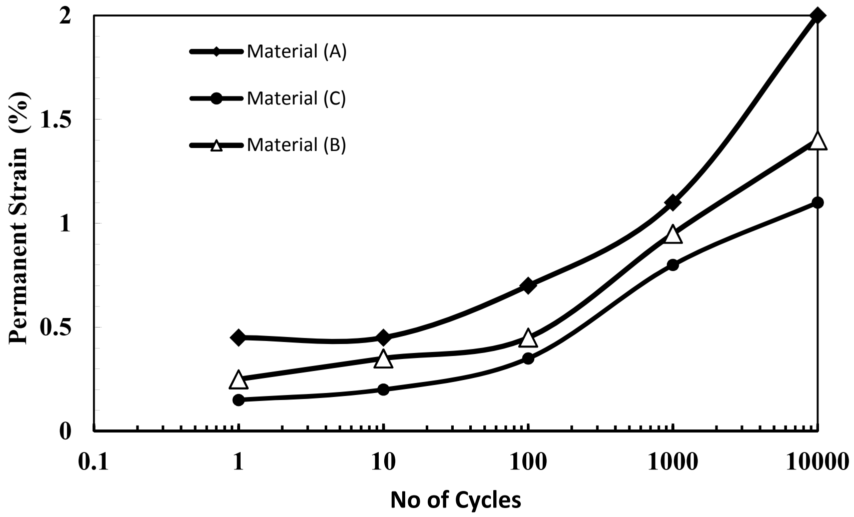

Figure 8: Permanent Strain at Deviator Stress of 35 kPa (Lateral Pressure=140kPa)

2.5. Resilient Modulus Analysis

The resilient modulus is an important input parameter for mechanistic design or AASHTO design procedure (the most widely used method all over the world). Therefore, it is important to enable the pavement designers to select suitable values of resilient moduli of pavement layers. Since, flexible pavements are always subjected to moving loads, the elastic modulus of pavement materials is usually estimated from the repeated triaxial tests using Equation 4.

$$M_r = \frac{\sigma_d}{\varepsilon_r} \qquad (4)$$

Where: σ_d is the deviator stress and εr is the recoverable (resilient) axial strain and Mr is the resilient modulus.

Many researchers like, Lekarp et al, 2000 have suggested K-θ model for granular and cohesion less materials and the same has been deduced for all tested materials. The simple model is of the form as given in Equation 5.

$$Mr = K_1 * \theta^{K_2} \quad (5)$$

where, θ is the bulk stress, sum of principle stresses (MPa) and K1 and K2 are material constants that are dependent on the physical properties of the material. The power equations fitted through the data points provided values of K1 and K2 for different test conditions are shown in Table 4.

Table 4: CBR Values of the Investigated Materials

	Material (A)	Material (B)	Material (C)
K_1	38.25	53.46	44.87
K_2	0.085	0.45	0.076

3. CONCLUSIONS

The suitability of three local materials available in Taif City, KSA namely, aggregates' crushers' dust, materials collected for the vallies' beds and coarse sand for using as subbase materials, was evaluated. It was found that all materials satisfy the requirements of subbase materials in terms of strength parameters and behavior under static and dynamic loads. It was also found that the materials collected from the bed of vallies exhibited the highest strength parameters like, CBR, E and Mr values in comparison with the other two materials. Moreover, it has shown better behavior under repeated loads like, lower values of permanent deformation which indicates high resistance to rutting formation.

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