Effects of Aggregate Gradation on Resilient Modulus and CBR in Unbound Granular Materials

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Abstract

Resilient modulus and California Bearing Ratio (CBR) in unbound granular materials are the key technical characteristics of layers in a flexible pavement design. Among the factors affecting these two parameters, the aggregate gradation is the most important. Using particle size distribution curve developed by AASHTO, together with other considerations mentioned in the related regulations have yielded desirable results in many cases. However, many roads loaded by heavy vehicles, for which all technical instructions of standard regulations were observed, have undergone deformations caused by subsidence of layers. According to the related technical documents, one hypothesis could be the proximity of aggregate gradation to the boundary areas. Therefore, the aim of this study was to determine the effect of changes in the scope of aggregation in the border areas on strength parameters. For this purpose, effects of aggregate grading variation on two types of aggregates, i.e. limestone and quartzite (as determined by AASHTO) were investigated using specific gravity, CBR, and resilient modulus tests. The results showed that, in the gradation boundaries determined by AASHTO, the difference between specific gravity values was insignificant. In the CBR and resilient modulus tests, however, there was a significant difference between test results in the upper and lower limits of gradation. In addition, gradation variation had a lower impact on resistance parameters in quartzite aggregate compared to limestone aggregate. Therefore, under special utilization conditions, materials with highest values of technical specifications should be used, since even materials whose technical specifications are in the standard range may not behave as expected in real world situations.

Keywords: Unbound granular materials, particle size distribution curve, specific gravity, CBR, resilient modulus

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1. Introduction

The effect of resilient modulus and CBR on layer thickness, deformation reduction, and pavement fatigue life is clear to pavement designers [AASHTO®, 1993]. In granular materials, the base and sub-base courses, resilient modulus and CBR are subject to factors such as gradation, percentage of flat and elongated aggregate, sand equivalent, Los Angeles abrasion test, weight loss with sodium sulfate, percentage of breakage, specific gravity, and Atterberg limits [Barksdale, 1991; AASHTO, 2014]. Gradation seems to play the most significant role among the aforementioned factors. In pavement surface layers, gradation plays an effective role in load tolerance, in addition to load distribution on a broader basis [AASHTO, 1993; AASHTO, 1990; AASHTO, 1993].

To select aggregate gradation, AASHTO has presented a range, rather than a single number, for the allowed percentage of material passing through each sieve [AASHTO, 1993]. It is usually said that the best choice is the average of the presented range. Most constructed pavements function well when gradation is in the allowed range and other aforementioned technical specifications are observed [AASHTO, 1990; AASHTO, 1993]. Reports by Sang and Kooh Company show that in some constructed roads, a subsidence caused by deformation of base and sub-base courses was seen on surface layers despite limestone aggregates with appropriate technical properties were used [Sang and Kooh Company, 2015]. One considerable factor in technical documents of these roads is grading curves. Gradation of the consumed aggregate was within AASHTO gradation range; however, the amounts were close to the boundaries set by AASHTO. One remarkable point in these types of projects is the passage of heavy vehicles in large numbers during utilization [Sang and Kooh Company, 2015].

CBR is a function of gradation, material, degree of moisture, soil specific weight, and the method through which CBR was conducted. The CBR of coarse-grained soil is higher than that of fine-grained soil, and more density results in higher CBR. Moisture has a negative effect, particularly on fine-grained soil. CBR test is an old conventional criterion to measure soil strength in road building. This test follows AASHTO T193 and is conducted under saturation conditions. CBR is defined as the maximum ratio of pressure required to penetrate the soil with a standard circular piston to pressure required to achieve an equal penetration on a standard crushed rock material, at 2.54 mm and 5.08 mm penetrations (6.9MPa and 10.3MPa respectively) multiplied by 100 [AASHTO, 2014; Huang, 2004].

The resilient modulus clearly describes the inelastic behavior of granular materials against loads. This rich description helps to design pavements based on real course function under load. Resilience is the capability of a material to absorb energy during elastic transformation and to emit it during unloading. Considering this feature and its consistency with real aggregate function, attempts have been made during recent decades to replace resilient modulus with CBR. This test is conducted according to AASHTO T307. A typical triaxial chamber is suitable for resilient modulus testing of base and subbase materials [Huang, 2004; AASHTO, 2003; Lavin, 2003].

According to the above, one hypothesis regarding the subsidence observed is the proximity of the chosen gradation to the boundaries of AASHTO's gradation range. In order to confirm or reject this hypothesis, an experimental study was required. The aim of the present study was to determine the effect of changes in aggregation in the allowed range on resistance and strength of the end material. Therefore, the main research questions are: "What effects does gradation variation within its allowed range exert on the strength and resistance of the mixture?", "how much does gradation variation within its allowed range affect the strength and resistance of the mixture??", "Is the research hypothesis confirmed?"; "On which resistance parameter does gradation variation have the most effect?" and finally "where in the gradation range is the highest degree of resistance observed? "

In this study, specific gravity, CBR, and resilient modulus tests were administered in accordance with AASHTO standards to estimate resistance and strength of the aggregates. These tests were conducted on aggregate base and sub-base courses, upper limit (UL) gradation and lower limit (LL) gradation, middle grading (MG) gradation, and Fuller-Thompson gradation (Fuller, W. and Thompson, S. equation or FandT equation). In order to obtain reliable results, limestone aggregates were obtained from Sang and Kooh Company, while quartzite aggregates were obtained from another mine. It should be mentioned that, six laboratory samples were prepared for each test.

2. Literature Review

One of the effective parameters in the resiliency of unbound granular materials is their specific gravity [AASHTO, 1990; AASHTO, 1993]. Fuller-Thompson (FandT) relationship (Equation 1) is the result of studies in this area, revealing the effect of gradation on specific gravity. FandT equation gives particle size distribution between smallest and largest sieve sizes so that all pores in aggregates are filled through appropriate gradation distribution. This will result in minimal porosity [Fuller and Thompson, 1907]:

$$P=\left(\frac{d}{D}\right)^{n} \qquad (1)$$

Where:

P=the percentage aggregate passing through any sieve size d
n=for maximum particle density n=0.5 according to F and T
d= sieve size being considered
D= maximum aggregate size
Other studies have shown that gradation not only affects the resistance of unbound granular materials, but also plays a significant role in the resilience of asphalt layers. Pan and Tutumbluer studied the effect of size and shape of coarse grained materials on functioning of flexible pavements. The results indicated that size and shape of coarse-grained materials have a considerable effect on pavement performance [Pan and Tutumluer, 2005].
Ekblad evaluated different models of determining resilient modulus. The effect of changes in exponent n in F and T equation was also considered in these evaluations. The results clearly demonstrated the importance of gradation and particle size distribution [Ekblad, 2008].
Golalipour et al. studied the effect of gradation changes on rutting in asphalt pavement. In addition to emphasizing the importance of gradation, their study revealed that creep stiffness is highest in the upper limit of gradation and permanent deformation is highest in the lower limit of gradation [Golalipour et al. 2012].
Bilodeau, Plamondon and Dore developed a model to estimate resilient modulus in granular materials by combining the effects of particle size distribution, material supply source, and frictional properties. In addition, they showed the role of gradation in increasing resilient modulus, that in turn results in increased pavement lifetime [Bilodeau, Plamondon and Dore, 2016].
Hamidi, Azini, and Masoudi showed the impact of gradation on the shear strength-dilation behavior of well graded sand-gravel mixtures. They studied shear strength behavior against specific gravity variations and surcharge pressure, and concluded that surcharge pressure increases shear strength and decreases relative density [Hamidi, Azini and Masoudi, 2012].

Kim et al. showed that resilient modulus is a parameters that clearly explains inelastic resilient behavior, and hence, it is widely used in advanced design methods to design and determine layer thickness [Kim D. and Kim J. R., 2007]. Resilient modulus is determined by dividing deviatoric stress by resilient strain [Huang, 2004]:

$$\mathrm{M_r}=\frac{\sigma_d}{\varepsilon_r} \qquad (2)$$

According to equation (2), the increase in resilient modulus is due to increased stress or decreased strain. In cases where stress is constant, increased aggregate quality leads to decreased strain, which in turn results in increased resilient modulus. Based on equation (3), increased resilient modulus, which is a result of decreased strain, leads to an increase in the number (Nf) of allowed loads. As a result, pavement utilization life increases without major deformations [Kim D. and Kim J. R., 2007; Barksdale, 1991; Uthus, 2007]:

$$\mathrm{N_f}=\left(\frac{\varepsilon_r}{240}\right)^{-3.29}\times 10^{6} \qquad (3)$$

Xiao and Tutumluer showed the impact of gradation and packing characteristics on stability of granular materials. They evaluated aggregate gradation and shape/morphological properties on pavement granular layer packing characteristics and load-carrying performance with a validated model (3d), using a particle image-aided discrete element method (DEM). According to the results, they suggested that the gravel-to-sand (G/S) ratio must be about 1.5 to the achieved densest packing [Xiao and Tutumluer, 2016].

Xiao et al. studied gradation effects on mechanical properties of aggregate base and sub-base granular materials in Minnesota. They used laboratory resilient modulus and shear strength. The result showed that the most significant correlations were between gravel-to-sand ratio and aggregate shear strength properties and the gravel-to-sand ratio could be used to optimize aggregate gradations for improved base and sub-base performance primarily influenced by shear strength [Xiao et al. 2012].

Gu et al. developed an efficient method for estimating resilient modulus of unbound aggregates. They developed prediction models and used laboratory experiments and multiple regression analysis conducted on 20 different base course materials. The laboratory experiments consisted of repeated load triaxial test and tests to measure performance-related base course properties. Using methylene blue value, percent fines content, gradation of particle sizes, and shape, angularity, and texture of aggregates, they designed prediction models with higher R-squared values [Gu et al. 2014].

Lekarp and Isacsson investigated the effect of changes in maximum particle size of graded aggregates on triaxial test results. They used three unbound granular materials in different grading scales. The results indicated that reduced grading scale had a significant effect on both the resilient and permanent strain responses, and the structural response observed depended on maximum grain size. They concluded that the effect was complex and inconsistent when different materials were compared; thus, they suggested triaxial testing of granular materials be performed at natural grading [Lekarp and Isacsson, 2001].

Ghabchi et al. carried out laboratory and field studies on the effects of gradation and source properties on stability and drainability of aggregate bases. The tests were resilient modulus (MR), falling weight deflectometre modulus (MFWD) and coefficient of permeability (k), which were performed on aggregates from three different sources. The results showed that MR and MFWD and lower k values in the laboratory and in the field experiments increased as the density of gradations increased. In addition, MFWD increased, while field k decreased over time, possibly because of traffic-induced compaction [Ghabchi et al. 2013].

Bilodeau, Dore and Pierre ran laboratory tests on three aggregate sources to study frost susceptibility that quantified with the segregation potential (SP). They investigated gradation influence on frost susceptibility of base granular materials. The SP values obtained for all sources were strongly related to the uniformity of grain size distribution and fines specific surface [Bilodeau, Dore and Pierre, 2008].

Cunninghama, Evansb and Tayebalib studied the effect of different gradations of aggregate base course (ABC) on material performance using five different gradations. The results indicated that coarser gradations gave better strength and resilience values; the coarsest composites became too difficult to work with in real conditions and they lacked the stability of well-graded ones. Another finding of the study was that, if the amount of fines in the specimens exceeded 8–12% by mass, the fines governed the behavior of the material [Cunningham, Evansb and Tayebalib, 2013].

Jiang, Wong and Ren used a California bearing ratio numerical test on graded crushed rocks using particle flow modeling. They selected a loading rate of 1.0–3.0 mm/min, a piston diameter of 5 cm, a specimen height of 15 cm and a specimen diameter of 15 cm for the CBR numerical test. The numerical results revealed that CBR values increased with the friction coefficient at the contact and shear modulus of rocks, while the influence of Poisson's ratio on CBR values was insignificant. The correlation between CBR numerical results and experimental results suggests that numerical simulation of CBR values could help assess the mechanical properties of graded crushed rocks and optimize the grading design [Jiang, Wong and Ren, 2015]. Yildirim and Gunaydin studied the application of different methods (simple–multiple analysis and artificial neural networks) for estimating California bearing ratio (CBR) from sieve analysis, Atterberg limits, maximum dry unit weight and optimum moisture content of the soils. The results showed strong correlations (R2 = 0.80–0.95) between the parameters and correlation equations obtained. The researchers recommended that the proposed correlations could be used for a preliminary design where there is financial and time limitations [Yildirim and Gunaydin, 2011].

3. Gradation and Materials

3.1 Selection of Gradation

Many studies have been conducted to find the appropriate gradation curves for base and sub-base aggregates. The proven practical results of such studies have been published in formal regulations. One of the most authentic and comprehensive of these regulations is AASHTO Guide for Design of Pavement Structures [AASHTO®, 1993], which specifies a range for the percentage of material that passes through each sieve. The optimal gradation choice is usually said to be the average of the range; F and T equation can be used to achieve maximum specific gravity [Fuller and Thompson, 1907]. Gradation IV from AASHTO–T27 and F and T gradation are presented in Table (1) and Figs (1) and (2).

The validity of F and T equation has been confirmed in various field and laboratory studies [TRB, 2013; Huang, 2004]. Therefore, in this study Fuller and Thompson gradation was used for comparing experimental results with control samples in gradation of base aggregates. This means that the results of experiments in the upper and lower limits of gradations were compared using Fuller and Thompson gradation.
In gradation of subbase aggregates, fuller and Thompson percentages of materials passing through sieves 3/8’’, #4, #10, #40 and #200 fell outside of the gradation curve; therefore, the results of experiments and control sample were compared based on average gradation. Minimum values of other technical specifications required to ensure durability and stability of the mixture were determined according to AASHTO (Table (2).

3.2 Selection of aggregates

In order to test the hypothesis, limestone aggregates required for the experiments had to be obtained from the same mine from which Sang and Kooh Company obtains the required materials for road building. In addition, to better evaluate test results of limestone aggregates, quartzite aggregates were also tested. Table (3) shows the technical specifications of these aggregates. It is clear that the selected aggregates meet the minimum technical requirements presented in Table (2).

Table 1. Gradation of base course and subbase course*

Sieve Size	Base Course (% passing)			Subbase Course (% passing)		
	IV	MG	F andT	IV	MG	F andT
1.5”	100	100	100	100	100	100
1.0”	70-100	85	81.6	90-100	95	81.6

¾”	60-90	75	71.2	-	-	-
3/8”	45-75	60	50.3	55-80	67.5	50.3
#4	30-60	45	35.6	40-60	50	35.6
#10	20-50	35	23.1	28-48	38	23.1
#40	10-30	20	10.6	14-28	21	10.6
#200	2-8	5	4.5	5-12	8.5	4.5

*(IV from AASHTO–T27, MG (Middle Grading), F andT equation

Table 2. Minimum technical specifications of base and subbase aggregates according to AASHTO standard [AASHTO®, 1993]

Test*	Base Course	Subbase Course
PI	Max. 4	Max. 6
LL	Max. 25	Max. 25
SE	Min. 40	Min. 30
Los Angeles abrasion test	Max.45	Max.50
CBR	Min. 80	Min. 30
Weight loss with sodium sulfate	Max.12	-
Percentage of breakage (Remaining on sieve 4.75mm)	Min. 75	-
Percentage of grains Flat and Elongated (Remaining on sieve 9.5mm)	Max.15	-

*(PI=Plasticity Index (AASHTO T90), LL=Liquid Limit(AASHTO T89), SE=Sand Equivalent (AASHTO T176), Los Angeles abrasion Test (AASHTO T96), CBR=California Bearing Ratio (AASHTO T193), Weight loss with sodium sulfate(AASHTO T104), Percentage of breakage (ASTM D5821), Percentage of grains Flat and Elongated (ASTM D4791)

Table 3. Technical specifications of aggregates selected for base and sub-base courses

Test	Base Course		Sub-base Course	
	Limestone	quartzite	Limestone	quartzite
PI	NP	NP	3	1
LL	Indeterminable	Indeterminable	8	3
SE (Lower limit)	64.2%	81.7%	55.6%	75.9%
SE (Fuller and Thompson for base	58.9%	73.4%	51.2%	63.1%

and Middle Grading for sub-base)				
SE (Upper limit)	50.3%	67.3%	44.5%	58.5%
Los Angeles Rattler Test	27%	14.2%	27.5%	13.9%
Weight loss with sodium sulfate	6.8%	8.2%	-	-
Percentage of breakage (Remaining on sieve 4.75mm)	100%	100%	-	-
Percentage of grains Flat and Elongated (Remaining on sieve 9.5mm)	3.2%	6.8%	-	-

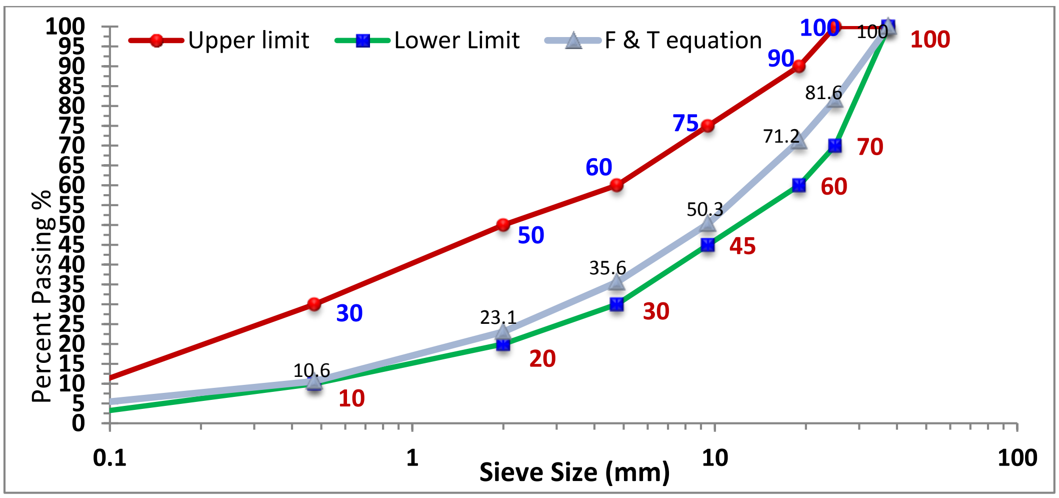


Figure1. Diagrams of base gradation for upper limit, lower limit, and F and T values

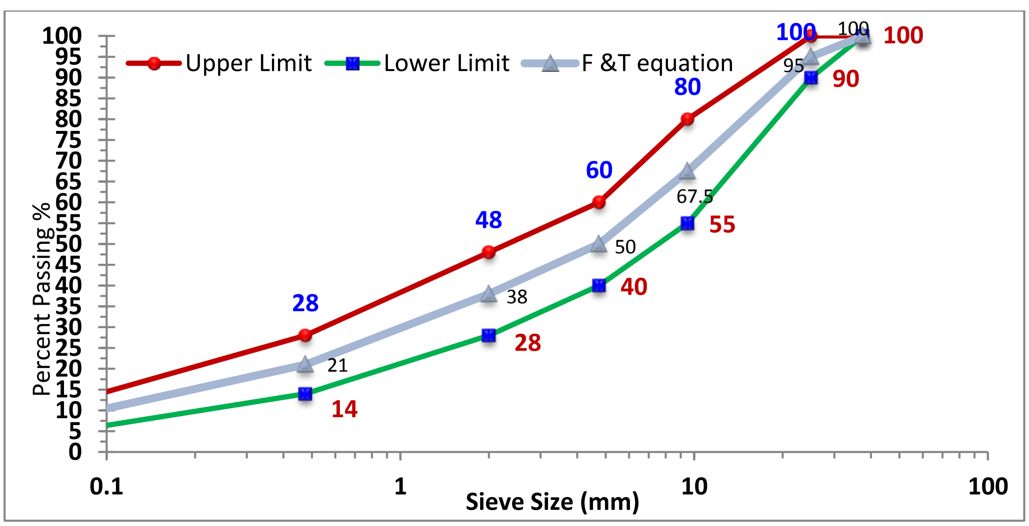


Figure 2. Diagrams of subbase gradation for upper limit, lower limit and middle grading values

4. Test and Results

4.1 Compacted Test

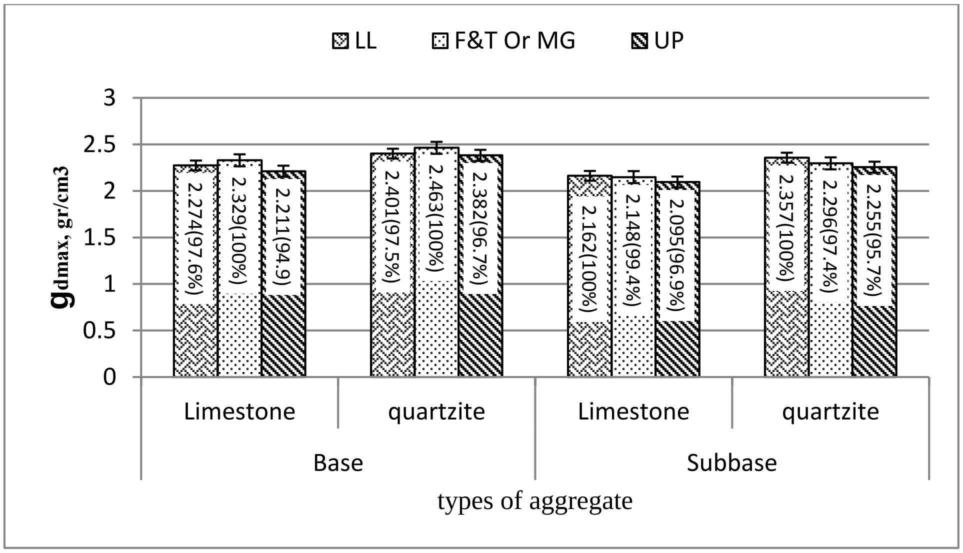
For the selected aggregates, high specific gravity means more appropriate gradation, since more aggregates can be contained in a certain volume. An increase in specific gravity results in increased strength of materials. In this study, density and specific gravity tests were carried out according to AASHTO T180–D [AASHTO, 2014; Huang, 2004]. Table (4) and Fig. (3) show the results of the aforementioned tests. Fig. (3) shows the results of specific gravity test on two types of aggregate base and sub-base courses.

As Table (1) shows, there is no significant difference between the gravities obtained for different gradations. The specific gravity values were measured in the middle, upper and lower limit gradations based on F and T equation. This lack of a significant difference could be due to the fact that density percentage was close to 100% for different gradations. As aggregate gradation moves toward well-graded coarse grains, specific gravity must increase; this was clearly shown in this experiment. In addition, other amounts passing through standard sieves ensure uniform gradation. Table (4) presents the results of specific gravity and optimal moisture tests on two types of aggregate base and sub-base courses.

Table 4. Result of specific gravity and optimal moisture tests on two types of aggregate base and sub-base courses

Subject*		$\gamma_{d_{\max}}$ **(gr/cm3)**		w_{opt} **%**	
		Limestone	quartzite	Limestone	quartzite
Base	LL	2.274	2.401	9.4	8.1
	Fan dT	2.329	2.463	10.1	8.9
	UP	2.211	2.382	11.2	9.5
Subbase	LL	2.162	2.357	11.1	9.2
	MG	2.148	2.296	12.3	8.6
	UP	2.095	2.255	13.2	8.0

*(LL (Lower limit), F and (Fuller and Thompson equation), UP (Upper limit), MG (Middle Grading))

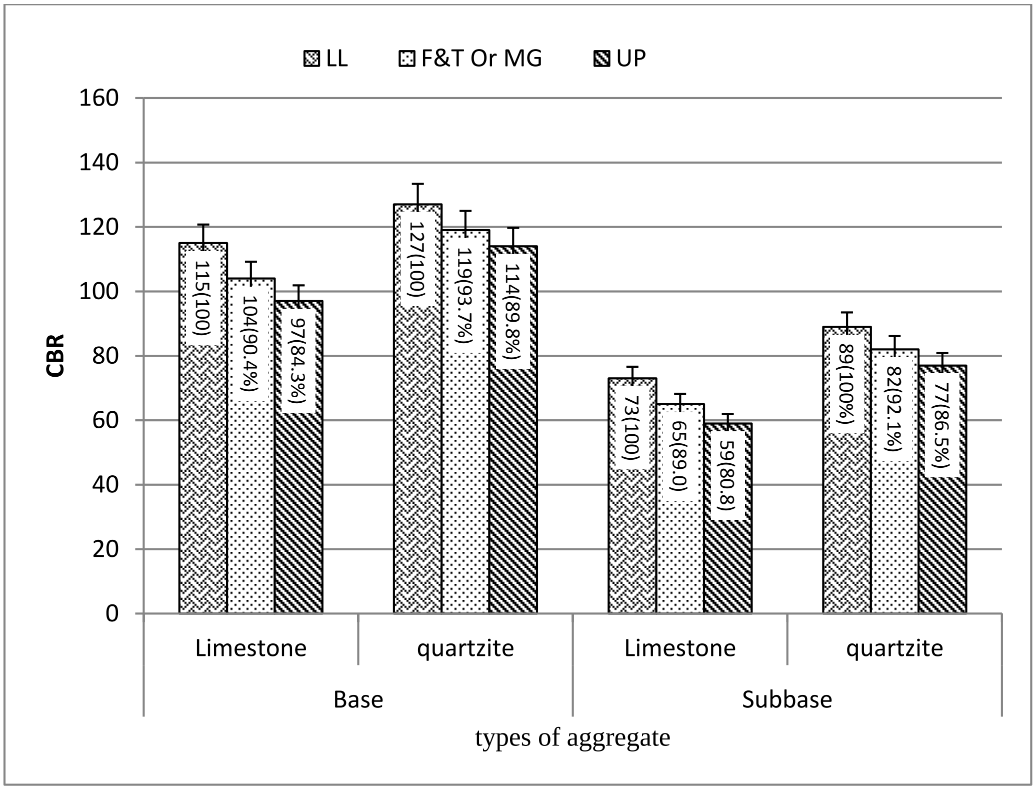


(LL (Lower limit), F andT (Fuller and Thompson equation for base), UP (Upper limit), MG (Middle Grading for subbase))

Figure 3. Result of specific gravity test on two types of aggregate base and sub-base courses

4.2 CBR Test for Aggregate Base and Sub-base courses

To test the materials, three different specimens were prepared and compacted in five layers with 55 blows to each layer. The compacting tool was a 4.89kg hammer. The specimens were allowed to take on water by soaking. The specimens were placed on the penetration test machine and used the load on the piston with penetration rate 1.25 mm/min. The load readings at penetrations of 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 7.5, 10 and 12.5 mm were recorded and the results of stress versus penetration depth were plotted to determine the CBR for each specimen. Figure (4) shows the results of experiments conducted on gradations of Table (1).



(LL (Lower limit), F andT (Fuller and Thompson equation for base), UP (Upper limit), MG (Middle Grading for subbase))

Figure 4. Results of CBR test on two types of aggregate base and sub-base courses

4.3 Resilient Modulus for Aggregate Base and Sub-base courses

According to AASHTO T 307, unbound granular base and subbase materials are Type I materials with a diameter of 6 inches for test specimens. In this test, a repeated axial cyclic stress with a load duration of 0.1 s and a cycle duration of 1.0 s was applied. The test began by applying a minimum of 1000 repetitions and using a haversine shaped load pulse. One of the most important variables in this test is the stress state that affects the modulus of granular materials. In this study, 15 stress states were used - three stress states (13.8, 27.6, 41.4 kPa) for confining pressure and five Stress states (12.4, 24.8, 37.3, 49.7, 62.0 kPa) for deviator stress. Tests were run in consolidated drained state (CD Test). The resilient modulus (M_r) is the ratio of maximum axial repeated deviator stress to maximum recoverable axial strain of the specimen. Tables (5), (6) and Fig. (5) display the values of resilient modulus [AASHTO, 2003].

The structural layer coefficient was used to determine the thickness of the layer according to AASHTO. The coefficient was calculated using resilient modulus. A change in resilient modulus could cause significant changes in the layer thickness; therefore, it is necessary to pay special attention to changes of resilient modulus. Tables (5), (6) and Fig. (5) display structural layer coefficients obtained from equations (4), (5), as well as the results of tests conducted on gradations of Table (1) [AASHTO®, 1993]:

$$\text{For Base}: a_2 = 0.249 Log_{10}\left(\frac{M_r}{0.007}\right) - 0.977 \quad (4)$$

$$\text{For Subbase}: a_3 = 0.227 Log_{10}\left(\frac{M_r}{0.007}\right) - 0.839 \quad (5)$$

Where M_r is resilient modulus in terms of MPa.

4.4 Data Analysis

To check the normality of distribution of data, Kolmogorov-Smirnov test was used. P-values were also calculated. As is shown in Table (7), since P-value (sig) is smaller than the significance level of 0.05, the null hypothesis is rejected.

Table 5. Results of resilient modulus test on limestone aggregates of base and subbase courses

Subject*		Resilient Modulus (MPa)	structural coefficient	Difference with structural coefficient $_{ax}$	Difference %
	LL	276	0.167	0	0
Base	FandT	253	0.158	0.009	5.4
	UP	233	0.149	0.018	10.8
	LL	157	0.149	0	0
Subbase	MG	138	0.136	0.013	8.7
	UP	122	0.124	0.025	16.8

*(LL (Lower limit), F andT (Fuller and Thompson equation), UP (Upper limit), MG (Middle Grading))

Table 6. Results of resilient modulus test on quartzite aggregates of base and subbase courses

Subject*		Resilient Modulus (MPa)	structural coefficient	Difference with structural coefficient$_{max}$	Difference %
	LL	328	0.186	0	0
Base	FandT	310	0.180	0.006	3.2
	UP	297	0.175	0.011	5.9
	LL	246	0.193	0	0
Subbase	MG	229	0.186	0.007	3.6
	UP	213	0.179	0.014	7.2

*(LL (Lower limit), F andT (Fuller and Thompson equation), UP (Upper limit), MG(Middle Grading))

(LL (Lower limit), F andT (Fuller and Thompson equation for base), UP (Upper limit), MG (Middle Grading for subbase))

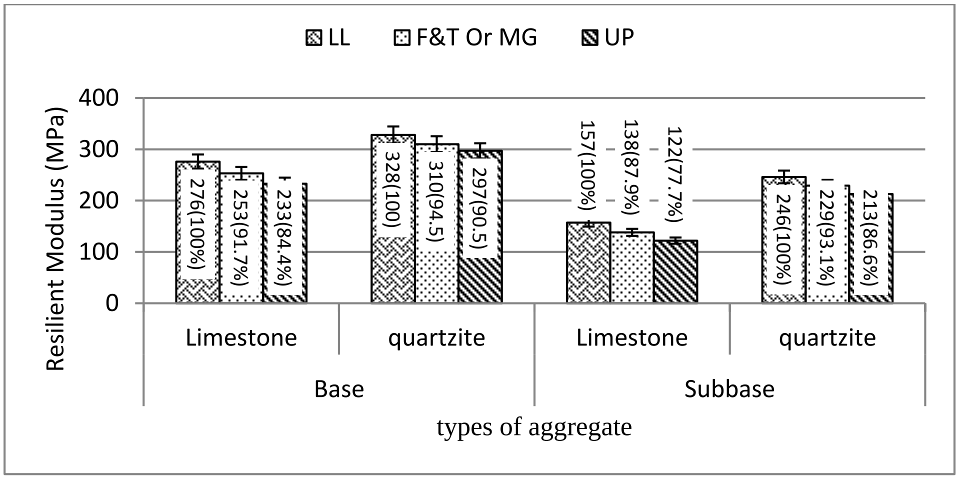


Figure 5. Results of resilient modulus test on two types of aggregate base and sub-base courses

Table 7. Results of data analysis with SPSS software

			g Base limestone	g Base quartzite	g Subbase limestone	g Subbase quartzite	CBR Base limestone	CBR Base quartzite	CBR Subbase limestone	CBR Subbase quartzite	Resilient Modulus Base limestone	Resilient Modulus Base quartzite	Resilient Modulus subbase limestone	Resilient Modulus subbase quartzite
N			9	9	9	9	9	9	9	9	9	9	9	9
Normal Parameters	Mean		2.27133	2.41533	2.13500	2.30267	105.33	120.00	65.67	82.67	254.00	311.67	139.00	229.33
	Std. Deviation		.132343	.122830	.229868	.165346	8.761	7.176	6.538	6.519	20.670	16.355	16.741	17.051
Most Extreme Differences	Absolute		.213	.172	.215	.194	.132	.117	.156	.142	.147	.126	.150	.178
	Positive		.213	.172	.215	.194	.132	.116	.156	.142	.147	.114	.150	.178
	Negative		-.150	-.145	-.146	-.147	-.110	-.117	-.126	-.140	-.126	-.126	-.107	-.142
Kolmogorov-Smirnov Z			.638	.516	.645	.582	.396	.352	.468	.426	.441	.378	.450	.535
Asymp. Sig. (2-tailed)			.810	.953	.799	.888	.998	1.000	.981	.993	.990	.999	.987	.937
Monte Carlo Sig. (2-tailed)	Sig.		.737[c]	.916[c]	.725[c]	.830[c]	.990[c]	.998[c]	.956[c]	.981[c]	.973[c]	.995[c]	.969[c]	.895[c]
	95% Confidence Interval	LowerBound	.728	.911	.716	.823	.988	.997	.952	.979	.970	.993	.966	.889
		UpperBound	.745	.922	.734	.838	.992	.999	.960	.984	.976	.996	.972	.901

5. Discussion

According to field observations and data of roads that are built with limestone aggregates, it seems that the deformations were caused by gradation of base and sub-base layers. To test the research hypothesis, the bearing capacity of base and sub-base courses was examined using specific gravity, CBR and resilient modulus. In addition, the effect of changes of resilient modulus on the structural coefficient and thickness of the layers were investigated. The selected materials were the same as those used in the construction of roads in the country. To increase the reliability of the results, experiments were also carried out on quartzite aggregates. The following can be concluded from results of tests conducted on base and sub-base materials:

1. For materials and gradation in Tables (1) and (3), a comparison of specific gravity values presented in Table (4) and Figure (3) shows that maximum gradation changes of aggregate base and sub-base courses in the standard range is 5.1%. Therefore, specific gravity changes are not considerable and have no significant effect on pavement performance. This means that damages to pavement are not due to gradation changes. This is reasonable since there was no difference between real world and experimental materials or their gradation or compaction percentage.

2. Figure (4) shows CBR test results for gradations presented in Table (1). A comparison between CBR results reveals that the difference between the lower and upper limits of gradation for limestone and quartzite aggregates ranges between 15.7% and 10.2% in the base and 19.2% and 13.5% in the subbase, respectively. The differences seem considerable and it might be argued that the detected defects stem from the use of upper limit gradation. Therefore, the hypothesis is confirmed. In this case, aggregate gradation moves toward non-uniform gradation. These results suggest that where the road is under heavy loading, the effects of gradation variation are more clearly noticeable. The above results might be related to the sensitivity of unbound granular materials to gradations and technical specifications. In addition, strength decrease in limestone aggregates is more significant than strength decrease in quartzite aggregates, which might be due to smaller general and friction stiffness of limestone.

3. Tables (5), (6) and Figure (5) show the results of resilient modulus. The differences between lower limit and upper limit of gradation for limestone and quartzite aggregates are 15.6% and 9.4% in the base course and 22.3% and 13.4% in the sub-base course, respectively. This result shows that gradation change is accompanied by resilient modulus change. Since resilient modulus is a more realistic demonstration of material behavior against loading, the 22.3% changes could be a better justification for pavement damages. The difference might be the result of non-uniformity of gradation in UP gradation limit. Again, these results might be related to the sensitivity of unbound granular materials to gradations and technical specifications. In addition, strength decrease in limestone aggregates is more significant than strength decrease in quartzite aggregates, which might be due to smaller general and friction stiffness of limestone.

4. According to Tables (5), (6) and Figure (5), the differences between minimum and maximum values of structural layer coefficients for limestone and quartzite aggregates are 10.8% and 5.9%, respectively, in the base course, and 16.8% and 7.2%, respectively, in the sub-base course. The nature of this coefficient necessitates that layer thickness be increased by the aforementioned percentages in order to compensate for the decrease in resilient modulus. This was not observed in pavement design, which will lead to decreased strength and increased damage.

5. The results presented in Tables (5), (6) and Figure (5) show that gradation variation in quartzite aggregates has less effect on strength parameters compared to limestone aggregates. Since quartzite aggregates are harder than limestone aggregates, it can be concluded that aggregate strength is more sensitive to gradation changes in weaker aggregates.

6. According to Figures (4) and (5), and Tables (5) and (6), gradation changes can affect aggregate strength and durability, particularity in the sub-base course. A comparison of the upper and lower limits of gradation indicates that this effect is considerable. Hence, the hypothesis is confirmed. Moreover, gradation changes greatly affect resilient modulus and can be effective in reducing load distribution over a wider area.

6. Conclusions

The results indicated that strength is highest in the lower limit of gradation. Therefore, the use of well-graded coarse-grained aggregates could significantly increase the strength. In special cases, the increase in strength is very crucial and materials with highest values of technical specifications must be used. Therefore, in circumstances where the road is under special utilization conditions, the aggregate base course and sub-base course need to have top technical specifications. Special utilization conditions include repeated heavy loading, extremely weak bedding, or harsh weather conditions. In case of weak aggregates, it is possible to improve material strength by increasing the thickness. In this way, weaker materials would be more sensitive to resistance against load

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