An Investigation on the Effects of Aggregates Properties on the Performance of Unbound Aggregate Base Layer

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Abstract

Unbound aggregates are the most common type of materials used for construction of the base layer of flexible pavements in Iran and worldwide. Due to the lack of binder, the performance of the compacted unbound aggregates is highly dependent on the properties of the gravel (particles remaining on sieve 4.75mm, sand (particles passing sieve 4.75mm and remaining on sieve 0.75mm), and fines (particles passing sieve No. 200) fraction of the mixture. As the bearing capacity and permeability are the main properties influencing the performance of the base layer; in this research, the effects of the angularity of gravel and sand, and the content and plasticity of fines on the bearing capacity and permeability of the compacted unbound aggregates have been investigated. The bearing capacity of the materials in dry and saturated conditions has been measured using CBR test, and the permeability has been measured using constant head permeability test method. The results showed that the CBR of the compacted mixture decreases with decreasing the percentage of angular sand and gravel in the mixtures. The strength of the mixture was shown to be optimum at a certain amount of fine content and decreases with increasing the plasticity of the fine fraction. It was found that the saturated CBR of the mixtures with non-plastic fines is higher than that of the dry CBR. It was also found that the angularity of the sand is more effective than the angularity of gravel on the CBR. The permeability of the mixtures is shown to increase with decreasing the angularity of the gravel and sand fraction in the mixture.

Keywords: Base, unbound aggregate, permeability, CBR, angularity, plasticity

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1. Introduction

Pavement materials are subjected to repeated loading over the service life of pavement. In an appropriately designed pavement, the stress level in pavement materials exceeds the elastic limit [Soleiman and Shalaby, 2015]. The occurrence of failure in pavement is a gradual process, and not a rapid collapse [Sharp, 1985]. Gradual accumulation of permanent deformation in unbound and asphaltic layers, and fatigue cracking of bound layers after a sufficiently large number of load applications results in pavement failure. The performance of the structural layers of pavement is vital to avoid premature failure of pavement. The base layer of a flexible pavement is the main structural component which distributes the wheel load to the underlying layers, support the upper asphaltic layers and protect the subbase and subgrade from the detrimental effects of moisture and freezing [Xiao, Tutumluer and Siekmeir, 2011; Tutumluer, 1995]. In most cases, as the subgrade assures a sufficient bearing capacity, the rutting phenomenon takes place mainly in the granular base and subbase layers causing pavement depression and progressive fatigue cracking of bituminous layers [Lekarp and Dawson, 1998; Wekrmeister, Dawson and Wellner, 2005].

Unbound aggregate materials are commonly used as highway pavement base due to their satisfactory mechanical strength and permeability [Cetin et al., 2014]. Millions tons of this material are used every year for highway construction. Unbound aggregates are a mixture of particles in different sizes, with a dense gradation, which are compacted to a desired level without any binder and used in pavement layers. These materials are made of graded crushed hard rocks, processed natural aggregates or secondary waste materials, such as steel slag, recycled asphalt and concrete aggregate etc. The performance of the unbound aggregates as base layer is highly dependent on the properties of the constituent particles, including the coarse (particles larger than 4.75mm, sand (particles between 4.75 and 0.075mm) and fines (particles finer than 0.075mm) fractions.

Under repeated loading, unbound granular materials show resilient and permanent deformation. The former is related to the stiffness and the latter is related to the resistance against permanent deformation. Stiffness and resistance against permanent deformation are the main properties of unbound granular materials associated with the pavement performance and usually considered in mechanistic-empirical pavement design methods. Stiffness of unbound layers affects the fatigue cracking of overlying asphalt layers, whereas, the accumulated permanent deformations under repeated loading leads to the failure of pavement structure due to rutting. Adequate thickness of each layer must be provided in such a way that the pavement structure does not experience failure due to fatigue cracking and rutting over the design period [Cerni et al. 2012].

The stiffness is an indication of the ability to distribute the imposed load to the underlying layers and support the upper asphaltic surfacing layers. Similar to the soils, the stiffness of unbound aggregates is presented by resilient modulus, which varies nonlinearly with stress state [Huang, 2004]. Resilient modulus, defined as the ratio of the deviatoric stress to the resilient strain, has a strong influence on pavements layer thicknesses, response, performance and cost [AASHTO, 1993; Huang, 2004]. As a matter of fact, the stress–strain relationship of unbound aggregates is non linear and reveals a complex elasto-plastic behavior, which involves resilient and permanent strain under each wheel load application. However, under normal wheel loading conditions and for adequate pavement design, the strains tend to become almost completely recoverable after a large number of load repetitions and can be considered entirely resilient [Lekarp, Isacsson and Dawson, 2000]. The stiffness of unbound materials is affected by many factors including the properties of the constituent particles. Lekarp, Isacsson and Dawson [2000] reported that the stiffness of unbound aggregates depends on the moisture content, dry density, gradation and plasticity index. Mishra [2012] stated that the shape of particles, angularity and fine content directly influence the mechanical properties of unbound aggregates including stiffness. Bilodeau and Dore [2012] investigated the effects of gradation and angularity of aggregate particles on the resilient modulus of compacted unbound aggregates. They found that, for partially crushed aggregates, the resilient modulus is mainly affected by the angularity of fines. On the other hand, in the crushed aggregates, the resilient modulus is mainly affected by he crushed coarse particles. The resilient modulus of unbound aggregates is measured by standard test method using repeated load triaxial testing. However, due to the complexity of the repeated load triaxial tests, most agencies don't routinely measure the resilient modulus using the triaxial testing, but, rather estimate it by empirical equations correlating the physical properties or simpler tests results, such as California Bearing Ratio (CBR), to the resilient modulus. There are some correlation equations between the resilient modulus and CBR [Powell et al., 1984; Heuklon and Klomp, 1962]. However, in the CBR test, the resistance against penetration is measured, which is an indirect measure of shear strength, while the resilient modulus is a measure of elastic properties. Therefore, the resilient modulus is not a simple function of CBR [Brown, Loach and Reilly, 1987]. But, nevertheless, partially due to its simplicity and well-acceptance among pavement engineers, the CBR test is widely used for evaluating pavement materials and correlation to resilient modulus [Brown, Loach and Reilly, 1987].

The main mode of failure in unbound granular materials is the permanent deformation. Unbound aggregates are elastic-plastic materials, which undergoes shakedown process under repeated loading [Johnson, 1985]. If the elastic limit is not exceeded, which is the case under light loads or thick pavements, an elastic–plastic material undergoes purely elastic behavior with no plastic deformation. When the elastic limit is initially exceeded, the material experiences initial plastic deformation which produces residual stresses. In subsequent load applications, the behavior of the material is dependent on the combined action of the applied load and the residual stresses produced by previous load applications. After a certain number of load repetitions, the residual stresses build up to a value that leads to a steady state with entirely elastic deformation under subsequent load applications (shakedown limit). When the shakedown limit is exceeded, the material experiences incremental plastic deformation under repeated loading. For repeated loads above the shakedown limit, an elastic– plastic material may undergo two deformation patterns: cyclic plasticity or progressive increase in plastic deformation (ratcheting) [Barber and Ciavarella, 2000]. Depending on the pattern, the failure mode will be different. In the first deformation pattern, the failure is governed by low-cycle fatigue, the failure mode in the second pattern is by exhaustion of ductility or static plastic collapse [Barber and Ciavarella, 2000]. Several studies have used the shakedown concept to characterize the behavior of unbound aggregates under repeated loading [Garcia-Rojo and Herrmann, 2005; Nazzal, Mohammad and Austin, 2011; Tao et al. 2010; Werkmeister, Dawson and Wellner, 2001, 2005]. After the elastic limit, the deformation behavior of unbound aggregates can be classified into three categories: plastic shakedown (A), plastic creep (B), and incremental collapse(C). The permanent deformation behavior within these three categories, as shown in Figure 1, is explained as follows [Werkmeister, Dawson and Wellner, 2004]:

- Plastic shakedown (range A): the response of unbound granular materials is plastic for a finite number of load applications, with a rapid decrease in permanent strain rate; afterward the response becomes purely elastic and no further permanent deformation occurs.
- In range B (plastic creep), unbound granular material experiences high permanent strain rate in the early loading cycles which decreases later to low or constant strain rate. After a high number of loading cycles, the material may experience progressive increase in permanent strain and advances toward an incremental collapse (range C).
- In range C (Incremental collapse), the material experiences progressively increasing increments of permanent strain with each load cycle, leading to failure after a relatively low number of loading cycles. The desirable behavior of a well-designed unbound aggregate is to be within the plastic shakedown or the plastic creep categories with an acceptable total permanent deformation [Sharp, 1985; Werkmeister, Dawson and Wellner, 2004].

The stiffness and resistance against permanent deformation is affected by the moisture content, which in turn, affected by the permeability of the mixture. The higher the permeability of the aggregates mixture, the less the materials are exposed to the moisture. The permeability of the unbound aggregates in base layer is an indication of the ability of that layer to drain the water infiltrated from the upper layers. Inadequate permeability of unbound base layer results in remaining water in that layer for a longer time, leading to decrease of stiffness and resistance against permanent deformation, or find way to the underlying layers which may cause their weakening against the imposed loads. The permeability of the unbound aggregates depends on the physical properties of the materials and the properties of the water [Das, 2008].

In this research, with the objective of improving the understanding of the bearing capacity and permeability of the unbound aggregates used in base layer of flexible pavements, the effects of some properties of the gravel, sand and fines fractions of unbound aggregates on the permeability and CBR are experimentally investigated.

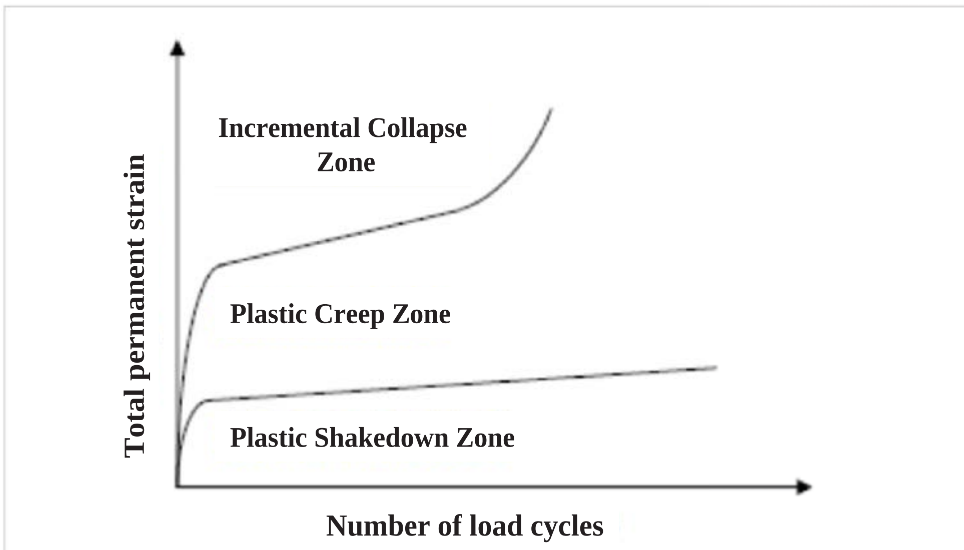

Figure 1. Permanent deformation behavior of unbound granular materials according to shakedown concept [Werkmiester et al. 2004]

2. Materials

The materials used in this research for making the test specimens include gravel (particles passing sieve 25mm and remaining on sieve 4.75mm), sand (particles passing sieve No. 4 and remaining on sieve No. 200) and fines (particles passing sieve No. 200). Two different types of angular and rounded gravel and sand were used in this research. The crushed gravel and sand particles were collected from an asphalt plant aggregate stockpiles which had been produced by crushing and processing river bed materials. Table 1 shows the properties of the aggregates from asphalt plant. The rounded materials were obtained directly from sieving the river bed materials without crushing. The source of both the crushed and rounded particles was siliceous. Except the angularity, the rest of properties of the rounded gravel and sand are the same as those shown in Table 1. Four different types of fines with different plasticity indices were also used in this research. The limestone filler of asphalt plant was used as non-plastic fines and three different kaolinites with the plasticity indices of 7, 14 and 20 were obtained from a porcelain production company.

For the mixtures used in this research, the gradation number IV in code 234 [Iranian asphalt pavement code, 2011] for pavement materials was used, and the materials between two successive sieves were taken in an amount, which the gradation of the mixture lies in the range specified by the code. Figure 2 shows the gradation of the mixtures used in this research and the limits specified by code.

Table 1. Gravel and sand properties

Properties (tests) / Materials	Sand Equivalent% ASTM-D2419	Los Angeles Abrasion Test ASTM-C131		Plasticity Index AASTM-D4318	Angularity in two sides % ASTM-D5821	Moisture Absorption %	Density ASTM-C127,128 ,D854	Flakiness BS 812	Loss in Magnesium Sulfate Solution % ASTM-C88
Coarse aggregate	-	21		-	100	1.6	2.607	12	1
Fine aggregate	53	-		N.P	-	2	2.593	-	1
Filler	-	-		N.P	-	-	2.72	-	-

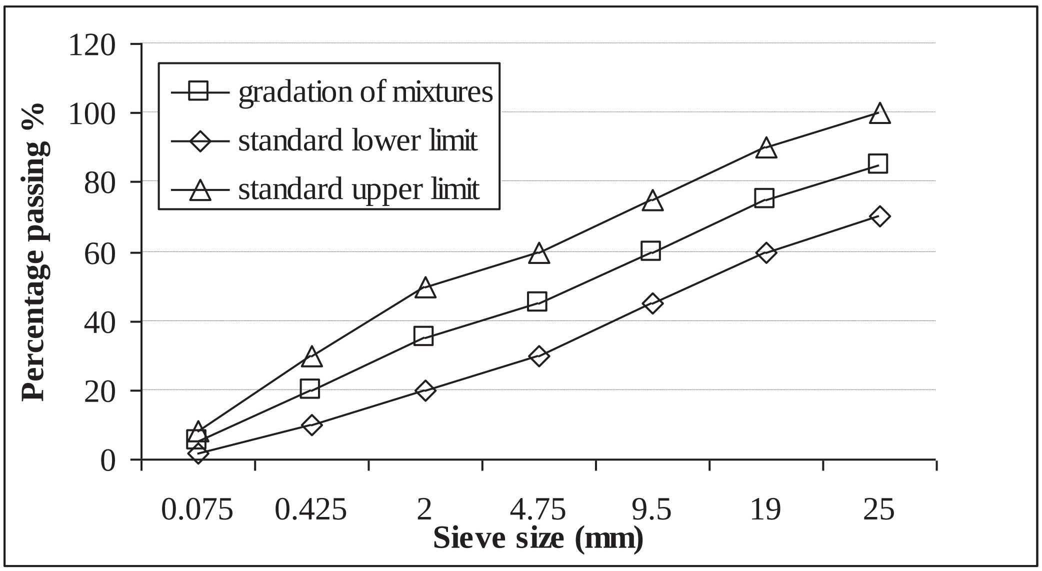

Figure 2. Gradation of the mixtures used in this research

3. Experimental Works

3.1 Tests Program

16 different mixtures were investigated in this research. The mixture with angular gravel and sand, and 5% of non-plastic fines was selected as the control. The effects of four variables, including the angularity of gravel and sand, the amount of fines and the plasticity index of the fines, on the dry and saturated CBR, and the coefficient of permeability have been investigated. With the same fines content and plasticity index as those of the control mixture, 25, 50, 75 and 100% of the angular gravel and sand were replaced with smooth surface natural gravel and sand. With the same angularity of sand and gravel and the plasticity index of the fines as those of the control mixture, four different mixtures with 0.5, 2.7, 7.2 and 9.5% of fines content were made. And, finally, with the same angularity of sand and gravel, and fines content, as those of the control mixture, 3 different mixtures with the plasticity indices of 7, 14 and 20 were made.

For making the mixtures with different percentages of angular gravel and sand, the required amount of the angular sand or gravel between two successive sieves were replaced with the rounded sand or gravel, between the same successive sieves. For making the mixtures with different percentages of fines, the difference between the weights of the fines with the control mixture was compensated by the weight of sands.

3.2 Compaction Tests

In order to obtain the optimum moisture content and the maximum dry density of the mixtures, to be used for making the specimens for the CBR and permeability tests, modified C method of compaction test, according to ASTM D1557, was conducted. In this method, the specimens are compacted by a 4.5kg hammer, in 5 different layers, each by 56 blows. For each mixture, 5kg of the materials was moisten with different moisture contents and compacted. Then, after measuring the volume of compacted specimen, and knowing the weight and moisture content, the dry density of the compacted specimens was calculated. Then, the variation of the dry density against moisture content was plotted and the maximum dry density and the optimum moisture content were obtained for each mixture. Figure 3 shows, as an example, the variation of the dry density with moisture content for the mixtures with different percentages of angular sand. As can be seen, the maximum dry density decreases, and the optimum moisture content increases with decreasing the angular sand content in the mixture. Using the same results, the maximum dry density and the optimum moisture content of the mixtures were obtained, which are shown in Table 2. In this table A1 to A4 denote the mixtures with 75, 50, 25 and 0% of angular sand, respectively, B1 to B4 denote the mixtures with 75, 50, 25 and 0% of angular gravel, respectively, C1 to C4 denote the mixtures with 0.5, 2.7, 7.2 and 9.7% of non-plastic fines, respectively, and D1 to D4 denote the mixtures with 5% of fines with plasticity index of 0, 7, 14 and 20, respectively. It should be noted that the mixture D1 is the control mixture, with 100% of angular sand and gravel and 5% of non-plastic fines. As can be seen, the optimum moisture content of the mixtures increases with increasing the fines content, the plasticity of fines and the percentage of angular sand in the mixture. The increase of the moisture content with the variables is attributed to the increase of specific surface with increasing fines content, plasticity index and angularity. It can also be seen that the maximum dry density of the mixtures increases with increasing the percentage of angular sand and the plasticity index of fines, and decreasing the fines content in the mixture. As the particles larger than 4.75 mm have not been used in the compaction test, the optimum moisture content and the maximum dry density of the mixtures D1 to D4 is the same as the control mixture.

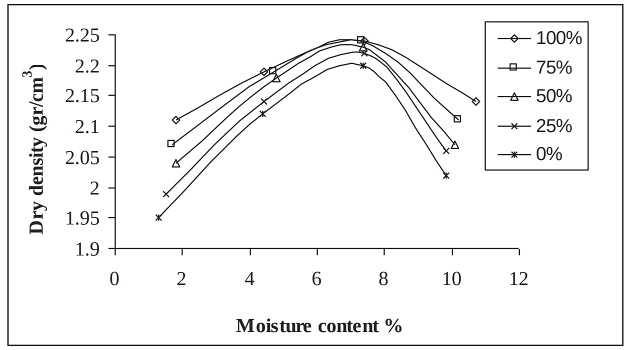

Figure 3. Variation of the compacted dry density with the moisture content for the mixtures containing different percentages of angular sand

3.3 CBR Tests

California Bearing Ratio (CBR) is the most common method for determination of the relative strength and bearing capacity of subgrade soil and unbound aggregates in highway and airport pavements. It was introduced in 1929 by Jim Porter working as Soils Engineer for the state of California [Brown, 1987]. CBR is measured as the ratio of the pressure required to penetrate a standard piston at a standard rate of 0.05in./min to a depth of 0.1 in. into the tested soil to the pressure required to penetrate the same piston at the same rate to the same depth into a standard crushed materials. Although CBR test does not simulate the dynamic loading of the materials in highway pavement; however, due to the simplicity and low cost, it is common to be used as an indication of the bearing capacity of pavement materials.

CBR tests on saturated and dry specimens were conducted according to ASTM D1883-07 standard method. The test was conducted on cylindrical specimen of the mixtures with a diameter of 15.2cm and approximate height of 13cm compacted to the maximum dry density at the optimum moisture content. For doing the test at saturated condition, the specimens were submerged in water tank for 4 days after placing a filter paper on the top surface and 4.5kg of burden weights. The test was conducted by loading the specimens by penetrating the piston at a rate of 0.05in./min and measuring the pressure required for penetrating the piston to a depth of 0.1in (Figure 4).

Table 2. The maximum dry density and the optimum moisture content of the mixtures

	A		B		C		D	
	ω_{opt} %	$\gamma_{d\max}$ (gr/cm^3)	ω_{opt} %	$\gamma_{d\max}$ (gr/cm^3)	ω_{opt} %	$\gamma_{d\max}$ (gr/cm^3)	ω_{opt} %	$\gamma_{d\max}$ (gr/cm^3)
1	7.32	2.24	7.4	2.2	7.35	2.24	7.4	2.2
2	7.36	2.23	7.4	2.2	7.38	2.23	7.42	2.21
3	7.4	2.22	7.4	2.2	7.42	2.22	7.48	2.23
4	7.34	2.2	7.4	2.2	7.47	2.21	7.55	2.24

Figure 4. CBR test

3.4 Permeability Tests

In order to determine the coefficient of permeability of the mixtures, constant head permeability test was conducted according to ASTM D2434-68 standard method. This test was conducted on the cylindrical specimens, 12cm in diameter, of the mixtures, compacted to the maximum dry density at the optimum moisture content. After compaction of the materials in the mold, and placing a paper filter and a porous stone on the top surface the cap of the mold was fastened. Then, the mold was placed in water tank for 4 days to saturate the specimen. It was then connected to the test set up under a head of 2 meter and the discharge valve to the water container was opened until a constant rate of discharge was achieved. Then, the volume of discharged water after 14 minutes was measured and the coefficient of permeability was measured using Equation 1. Figure 5 shows schematically the constant head permeability test.

$$K = \frac{QL}{AH} \tag{1}$$

Where; k is the coefficient of permeability in cm/sec, Q is the rate of discharge in cm^3/sec, L is the length of specimen in cm, and H is the constant head in cm.

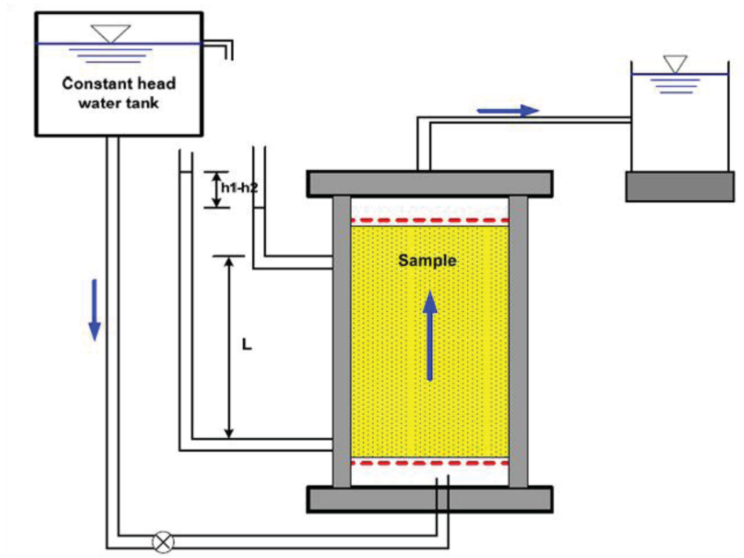

Figure 5. Schematic of constant head permeability test

4. Test Results and Discussion

Figure 6 shows the variation of the dry and saturated CBR of the unbound aggregates with the percentage of fines passing sieve No. 200. It should be noted that, in this mixes, the plasticity index of fines is 0, and the angularity of sand and gravel is 100%. As can be seen, for all the mixtures, the CBR in saturated condition is higher than that in the dry condition, which seems to be due to the shear strength enhancement due to the surface tension of the water in voids [Carrier, 2003; Chapuis, 2004]. It can also be seen that, the CBR, both in dry and saturated conditions, have a maximum value at 5% of fines content, which is for the control mixture. This amount of fines content is the required amount to fill the voids between the mix of gravel and sand particles, beyond which their contact is lost and the shear strength is decreased, and, for the fines content lower than the optimum content, the friction component of the strength is decreased, resulting in a lower CBR.

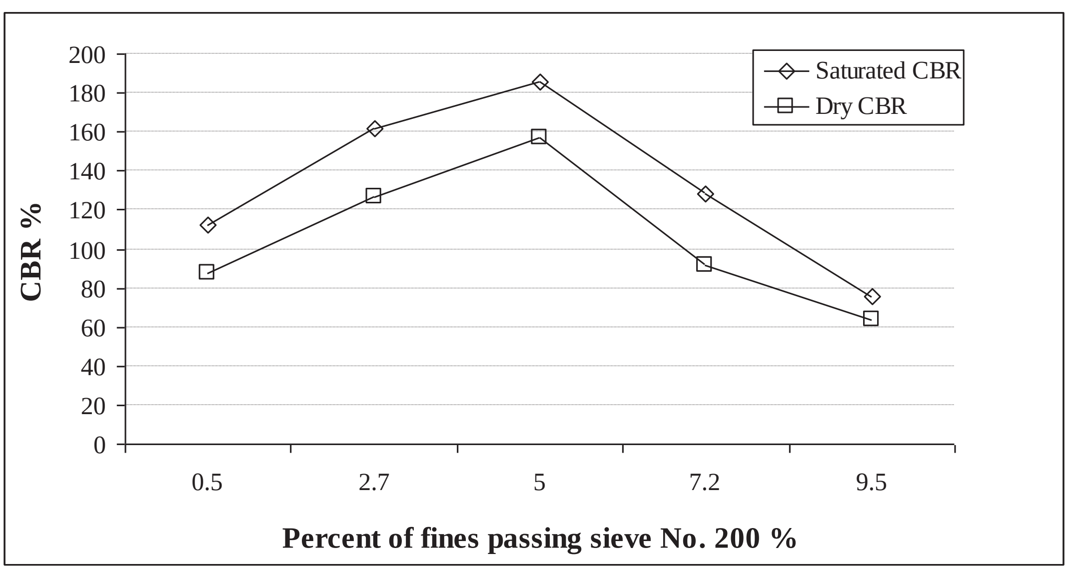

Figure 6. Variation of the CBR with the fines content

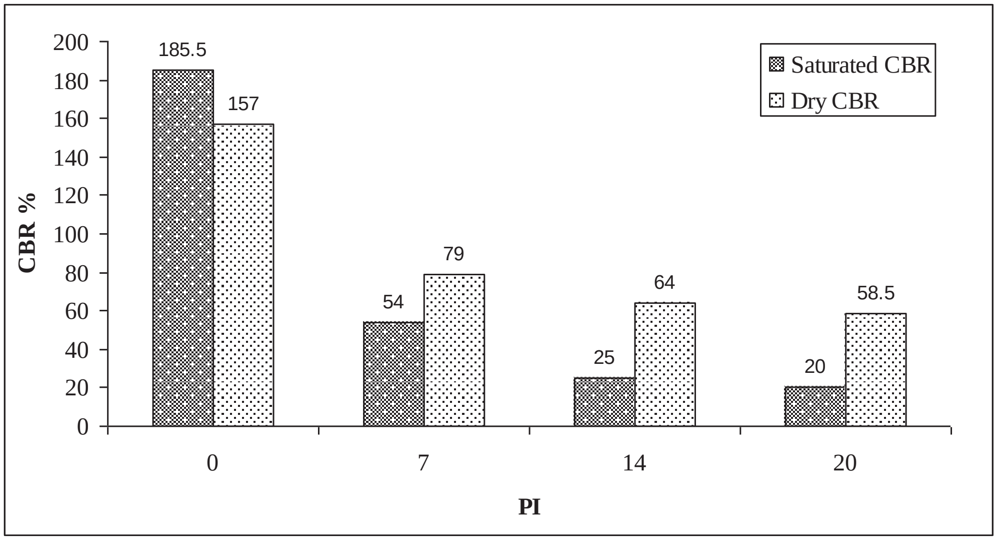

Figure 7. Variation of dry and saturated CBR with the plasticity index of fines

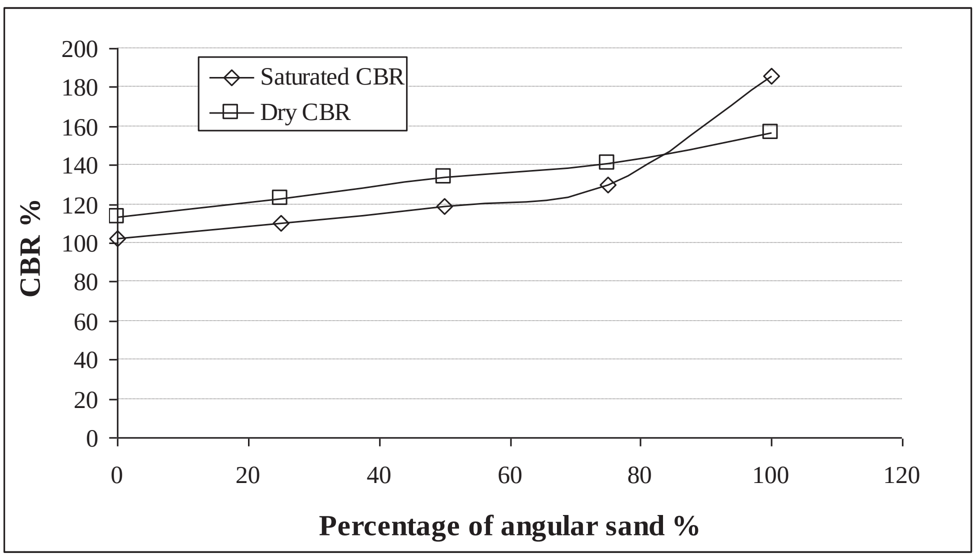

Figure 8. Variation of saturated and dry CBR with the percentage of angular sand in the mixture

Figure 9 shows the variation of the dry and saturated CBR of the unbound aggregates with the percentage of angular gravel in the mixtures. As can be seen, similar to the effect of sand angularity, the dry and saturated CBR of the mixtures increase with increasing the percentage of angular gravel in the mixture. These results indicate that, to obtain a stiffer base layer with a higher resistance against permanent deformation, angular crushed coarse aggregates be used instead of rounded natural gravel with smooth surfaces. The Iranian code limits the minimum angularity of coarse fraction of base aggregates to be 75%.

In order to compare the effect of gravel and sand angularity on the strength of compacted unbound aggregates, the variation of the saturated CBR with the percentage of angular sand and gravel is shown in Figure 10. It can be seen that the saturated CBR is more sensitive to the angularity of sand fraction. Although sand and gravel constitute 40% and 55% of the mixtures, respectively; however, sand has a higher specific area and surfaces in contact. Therefore, using a sand with a lower percentage of angular particles results in a higher reduction in the friction component of shear strength and CBR.

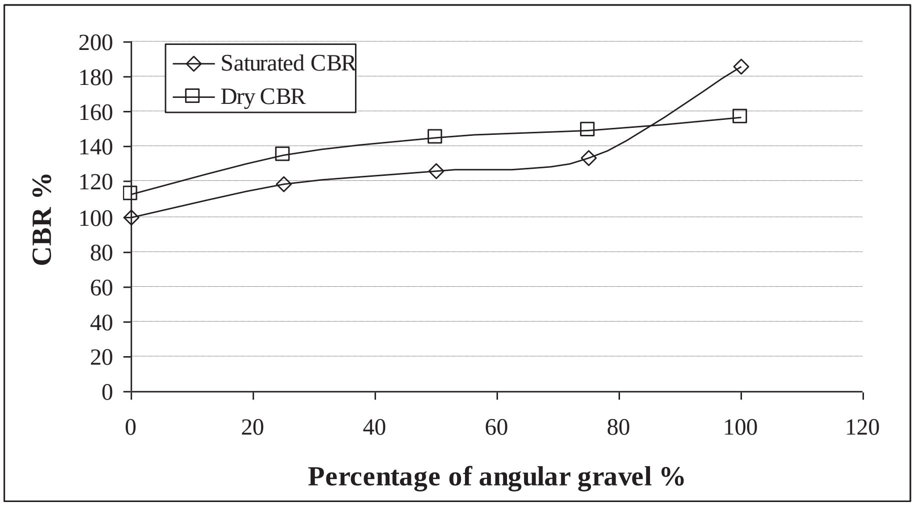

Figure 9. Variation of the dry and saturated CBR with the percentage of angular gravel particles

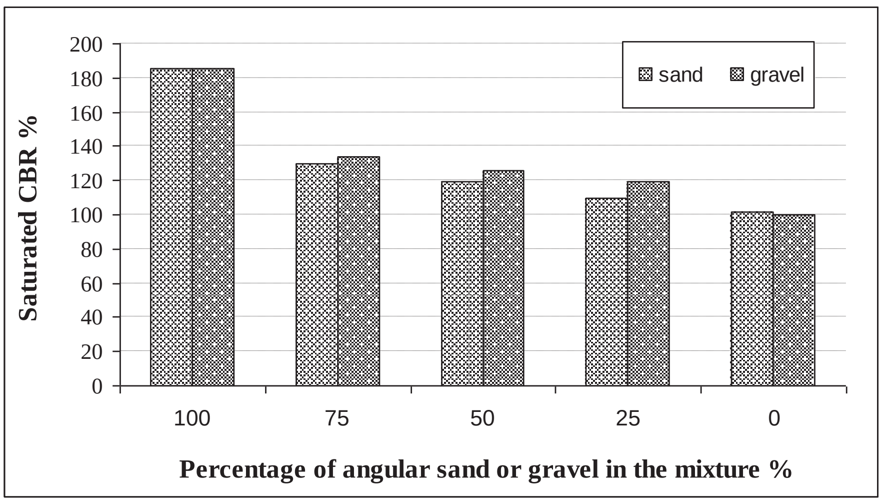

Figure 10. Variation of the saturated CBR of the mixtures with the percentage of angular gravel and sand particles

Figure 11 shows the variation of the coefficient of permeability of the unbound aggregate mixture with the percentage of the particles finer than 0.075mm in the mixture. As can be seen, up to a certain level of fine content, the permeability of the mixture decreases with increasing the fine content, after which, the trend reverses. With the increase of finer particles the voids between the coarser particles are filled which leads to reduction of permeability. This trend continues up to a certain amount of fines, after which the increase of fine particles prevents appropriate densification resulting in increase of voids content in the mixture and permeability. In order to discharge the infiltrated water from the base layer, a higher coefficient of permeability is desirable, which can be achieved by using a lower fine content in the mixture. However, as described in previous sections, the reduction of fines in the mixture results in a reduction in the strength, which is not desirable. Therefore, in the Iranian code for the gradation of unbound aggregate base, the percentage of the particles finer than 0.075mm is limited in the range of 2 to 8%, which does not provide a significant coefficient of permeability.

Figure 12 shows the variation of the coefficient of permeability of the mixture with the plasticity index of fines. As can be seen, the permeability of the unbound aggregates decreases with increasing the plasticity index. It should be noted that in these mixtures, the percentage of the particles finer than 0.075mm is the same value of 5%, and 100% of

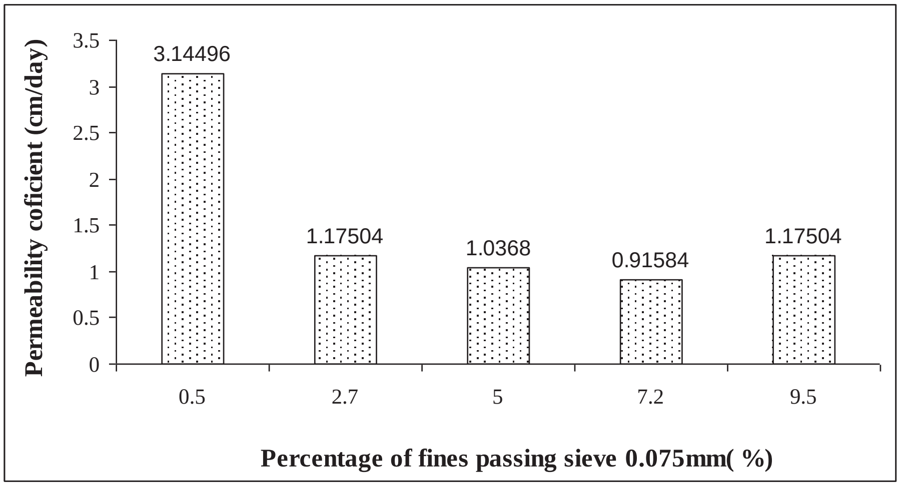

Figure 11. Variation of the coefficient of permeability with percentage of particles finer than 0.075mm

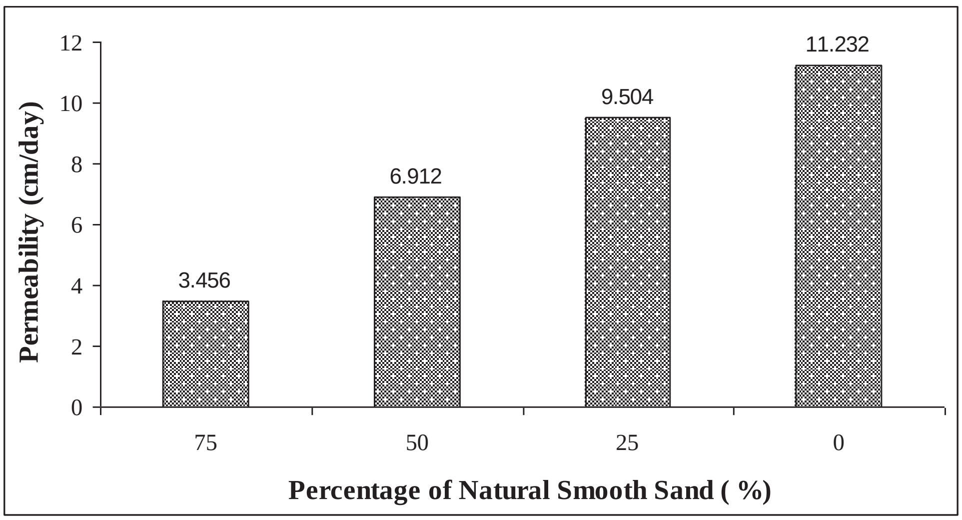

Figure 12. Variation of the coefficient of permeability with the plasticity index of fine particles

the coarse fraction are angular particles. The decrease of the permeability with increasing plasticity index is attributed to its effect on the voids ratio, which is directly related to the permeability. Therefore, in order to have a higher permeability without detrimental effect on the strength, the use of non-plastic fines is recommended.

Figure 13 shows the variation of the coefficient of permeability with the percentage of angular sand particles in the mixture. As can be seen, the permeability of the unbound aggregates increases with increasing the percentage of natural sand with smooth surface, in the mixture. This is attributed to the decrease of surface area of the particles in contact, which results in the increase of void ratio and permeability. Although the increase of permeability is desirable; however, as the increase of smooth surface sand particles adversely affects the strength, it is not recommended to use such composition.

Figure 14 shows the variation of the coefficient of permeability with the percentage of angular gravel particles in the mixture. As can be seen, similar to the effect of smooth surface natural sand, the permeability of the mixture increases with increasing smooth surface gravel particles in the mixture. similarly, it can be described by the reduction of contact surface area between particles with the increase of smooth surface particles, resulting in the increase of voids ratio and permeability. However, the comparison of the Figure 12 and 13 reveals that, the effect of the angularity of sand particles on the permeability is much higher than that of the gravel. For example, the mixture with 100% of its sand of natural

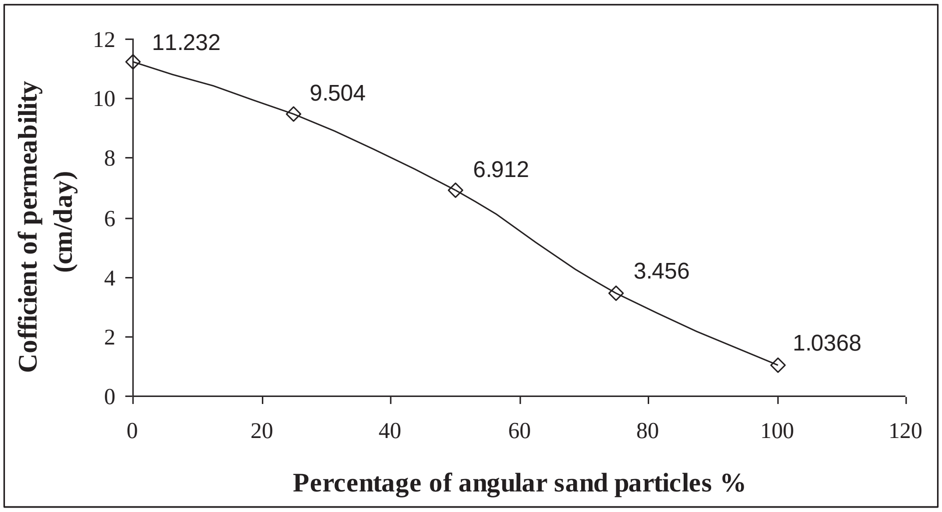

Figure 13. Variation of the permeability with the percentage of angular sand in the mixture

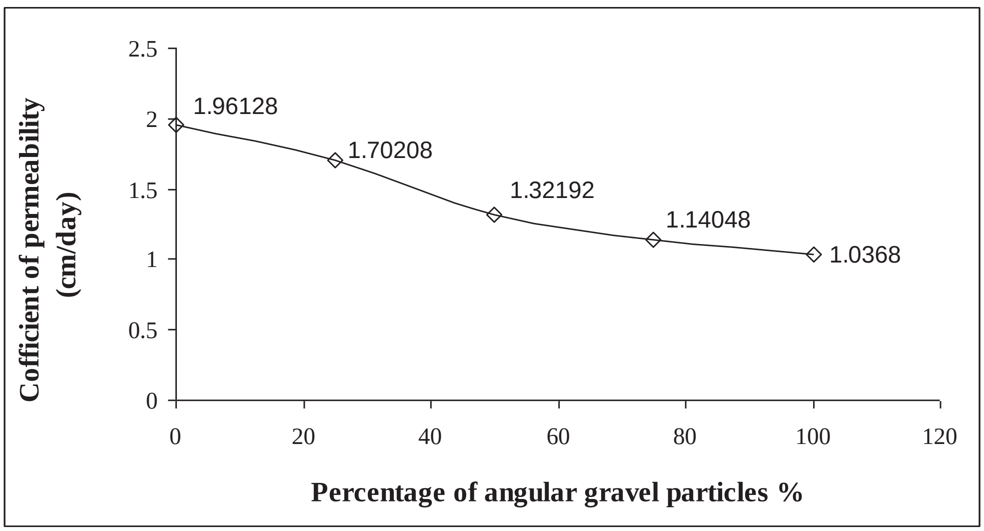

Figure 14. Variation of the permeability with the percentage of angular gravel in the mixture

materials with smooth surface is approximately 6 times higher than that of the mixture with 100% of smooth surface gravel particles. This is because of the higher specific surface area of the sand compared with that of gravel. It should be noted that gravel and particles constitute 55% and 40% of the mixture, respectively.

5. Conclusions

The effects of the percentage of angular sand and gravel, the fines content and the plasticity index of fines on the dry and saturated CBR, and the coefficient of permeability have been investigated. Brief results are represented as follows:

- The maximum dry and saturated CBR can be achieved at a certain level of fines content, which is associated with the amount of fines for which the maximum density can be achieved.
- The dry and saturated CBR decreases with decreasing the percentage of angular sand and gravel, and increasing the plasticity index of the fines.
- For the mixtures with non-plastic fines and 100% of angular sand and gravel the saturated

CBR was found to be higher than the dry CBR. This finding needs more investigation.

- Due to the higher specific area, the effect of the sand angularity on the strength is higher than that of the angularity of gravel. Therefore, it is recommended that requirement be set in codes to limit the minimum angularity for sand.
- The coefficient of permeability of unbound aggregates decreases with increasing the fines up to a level, beyond which the trend reverse.
- The permeability of the unbound aggregates decreases with increasing the plasticity index of the fines, and the percentage of the angular sand and gravel in the mixture.
- The coefficient of permeability of the unbound aggregates is more sensitive to the angularity of sand than that of gravel.

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