LABORATORY PERFORMANCE OBSERVATION OF CEMENT GROUTED BITUMINOUS MIX USING DIFFERENT GRADES OF BINDER

A Dissertation submitted

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Submitted by

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ABSTRACT

Cement Grouted Bituminous macadam contains high void bituminous mix incorporating proper stone on stone contact for interconnectivity between voids, which eventually leads to 25% to 35% air voids in the bituminous skeleton. The bituminous mix will assure the flexibility, whereas, stiffness will be assured by grout, thus making it a semi-flexible layer. In this study, high void bituminous mix having volume of voids in between 30% to 35 %was prepared with different grades of paving bitumen such as VG 30, VG 40 and PMB 40 and likewise with emulsion at optimum content and then filled with grout (for full depth penetration) to make it a composite wearing course. The grout used was a high-strength good flow able grout. The grout was allowed to flow under the action of gravity to fill all the interconnected voids. The mechanical properties of the cement grouted composite course with different grades of paving bitumen and emulsion was incorporated by performing tests such as Marshall Stability, Compressive strength, indirect tensile strength and resilient modulus (ITSM). Whereas, performance was evaluated by performing Dynamic fracture energy, moisture induced susceptibility (MIST), and Abrasion. It was observed that CGBM prepared with PMB 40 bitumen possess remarkable properties as compared to other paving bitumen like VG 30 and VG 40. The results inferred from the mechanical and performance characteristics tests in case of Emulsion mix CGBM signifies the dominance of grout material in the composite mix and the findings from performance characteristics of Emulsion mix CGBM was not conclusive. It was also inferred from the results that this semi-flexible layer possesses superior properties as compared to conventional bituminous concrete (BC) course. Pavement design was done to check the suitability of CGBM as a wearing course in comparison with BC using IITPAVE. Pavement thickness for moderate traffic was estimated after structural analysis with IITPAVE. From the properties and specifications of CGBM, it can be expected that, incorporating this as a wearing course or overlay course as a replacement of BC in urban roads for moderate traffic can be great asset for sustainable road development.

DECLARATION

The author hereby declares that this thesis entitled **"Laboratory Performance Observation of Cement Grouted Bituminous Mix Using Different Grades of Binder"** in whole or part has not been used to obtain any degree in this, or any other institute, Except where references have been given in text, it is entirely the authors own work. The authors Confirms that the library may lend or copy this thesis upon request for academic purposes.

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CERTIFICATE

This is to certify that, the thesis, **"Laboratory Performance Observation of Cement Grouted Bituminous Mix Using Different Grades of Binder"** being submitted by **Pritam Bhowmik (801723017)** in partial fulfillment for the award of degree of Master of Engineering in Infrastructure Engineering at **Thapar Institute of engineering and Technology, Patiala** is a bonafide work carried out by him under our thorough guidance and supervision and that no part of the thesis has been submitted for the award of any other degree.

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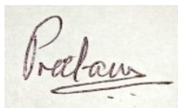

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF PHOTOS

LIST OF ABBREVIATIONS

CGBM	Cement Grouted Bituminous Mix
BC	Bituminous Concrete
VG	Viscosity Grade
PMB	Polymer Modified Bitumen
ITS	Indirect Tensile Strength
MR	Resilient Modulus
MIST	Moisture Induced Sensitivity Test
TOT	Texas Overlay Tester
OPC	Ordinary Portland Cement
SP	Super Plasticizer
WC	White Cement
SF	Silica Fume
VIM	Voids in Mix
VMA	Voids in Mineral Aggregates
DBM	Dense Bituminous Macadam
AIV	Aggregate Impact Value
SSD	Saturated Surface Dry
FI	Flakiness Index
EI	Elongation Index
NMAS	Nominal Maximum Aggregate Size
SD	Surface Dressing
CRI	Crack resistance Index
MSA	Million Standard Axles
VDF	Vehicle Damage Factor
LDF	Lane Distribution Factor
WMM	Wet Mix Macadam
GSB	Granular Sub Base

CHAPTER 1

1. INTRODUCTION

1.1 BACKGROUND

Pavements are commonly categorized as flexible/bituminous or rigid/concrete pavements depending on the type of the materials used in the structural layers. Bituminous pavements deteriorate generally due to permanent deformation, fatigue cracking and moisture damage particularly in locations with heavy traffic volumes and heavy vehicular loading. On the other side, concrete pavements, which have very high resistance to permanent deformation, require more construction periods due to the longer time essential for curing of concrete and increased construction cost. Flexible pavements, the failures of which are rutting, cracking and moisture damage, implement almost 95 per cent of the road projects executed in India. These kinds of failures are depicted in Photo 1.1 (a), (b), (c) respectively.

a) Rutting type of field failure

b) Fatigue cracking failure

b) Moisture Induced Damage (Stripping) at Field

Photo 1.1. Different types of Failures for Bituminous Pavements

In an attempt to curb these primary failure modes of bituminous pavements, the idea of cement grouted bituminous mixes has been conceptualized. The practice of grouting the high void bituminous layers with cementitious materials started in the early 1950s. The early idea was mainly to reduce the damage caused to bituminous pavements due to spillage of oils (Van De Ven and Molenaar, 2004). The first development of the semi-flexible process was carried out in the 1950's, in France, as a protection of asphalt concrete surface course against the attack of waste oils and fuels. The French construction company Jean Lefebvre Enterprises, as a cost-effective alternative to Portland cement concrete further developed this process, known as Salviacim. After the *Salviacim* process became successful, its usage spread throughout various countries including Great Britain, South Africa, Japan, Australia and Saudi Arabia. Since then, similar products have been used with different designations, according to the location.

The CGBM layer is normally described as a high void bituminous concrete mixture, having 25% to 35% void content (by volume of Mix design), which is filled with a cement grout. It is apparent that the high void bituminous mix be designed properly to have sufficient air voids with proper inter-connectivity in them and to select a grout, which could fully penetrate into the voids through the interconnectivity of the voids. The high-void bituminous mixture and cement grout are produced and placed separately. The cement grout is generally composed of cement, sand, fly ash, micro silica, super-plasticizer and water. Once after the bituminous mixture reaching to ambient temperature, the cement grout material is poured onto the high void bituminous mix. Most of the void spaces in the bituminous mix are occupied with cement grout by the gravity force. The newly grouted surface should be squished to increase the skid resistance. The CGBM layer normally reaches full strength in 28 days, in most cases it may be opened to traffic in 24 hours. The CGBM material has been used in various applications, airport and vehicular pavements, industrial floorings, gasoline stations, city plazas and malls, railway stations, and port areas.

1.2 NEED OF THE PRESENT STUDY

Urban roads, which are primarily consists of bituminous concrete (BC), often gets damaged due to Moisture induced due to stagnant water during monsoon by the pore pressure developed by movement of the wheels, Which leads to moisture induced damage such as stripping, leading to failure in the form of Pot holes. So, to resist such damage CGBM is a boon. Moreover, the bituminous mix will give the flexibility and cement grout will give the stiffness required thus reduced the possibility of providing joints to prevent thermal expansion unlike that of rigid pavements.

CGBM also possess superior performance as compared to flexible pavements in terms of rutting, fatigue and moisture induced damages. Furthermore, according to literature the design life of CGBM is more than that of flexible pavements, which will reduce the maintenance cost and thus optimize the life cycle cost. Use of polymer modified binders in place of viscosity grade binders, to improve the properties. Generally, PMB mixtures appear to have high stiffness and possess higher fracture strength of the mix especially at high temperatures. Generally, there stability is also good (resistance to shear flow) as compared to viscosity grading bitumen. Polymer modified bitumen forms a thick uniform mixture over the aggregates, which will prevent the composite grout mix from segregation. Polymer modified bitumen also increases the resistance against chemicals, thus wider its application area in industrial floors.

In addition to paving bitumen, Emulsion based high void bituminous mixes has also been tried to prepare CGBM mixes, which will widen the horizon of CGBM's applicability.

1.3 SCOPE OF THE STUDY

The scope of the study is limited to the following: -

(a) To choose the suitable gradation for open graded High Void Bituminous mix skeleton to result into a void content in between 25% to 35 %.

(b) To determine the optimum binder content of the mix for proper coating and flexibility of the mix and also to get the proper interconnectivity between voids without clogging the voids and to determine the void content at maximum bulk density (by fixing the number of flows required) of the mix for full depth penetration of the grout.

(c) To fix the optimum water content of the grout slurry for full depth penetration at desirable strength of the grout.

(d) To study and compare the effect of varying Binder grades like PMB 40, VG 30, VG 40 and emulsion and varying binder content on volumetric properties of the high void bituminous mix.

(e) To study the Mechanical Parameters like Marshall Stability, Resilient Modulus, Indirect Tensile Strength (ITS) and compressive strength of CGBM with different grades of binder.

(f) To study the performance parameters such as dynamic fracture energy, Moisture Induced Stress, Cantabro (Abrasion) loss of CGBM prepared with varying grades of binder.

(g) To carry out the design of pavements and structural analysis incorporating Cement grouted composite wearing course as a surfacing layer for urban areas having moderate traffic.

(h) To check the suitability of the cement grouted porous bituminous mix with bituminous concrete as a surfacing layer.

1.4 METHODOLOGY

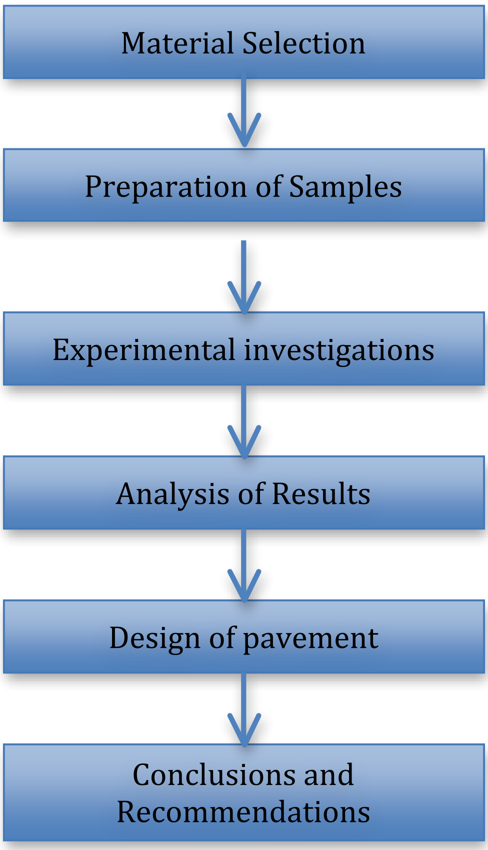


Fig 1.1: Shows the methodology structure of the study

For the experimental investigation and keeping in mind the scope of the study, High Void Bituminous mix skeleton was made with different grades of bitumen such as VG 30, VG 40 and PMB 40 and tailor-made emulsion in the MS category at desirable void content for full depth grout penetration. Several trials were adopted for the grout slurry at varying water contents were carried out to check full flow ability at optimum strength of the grout. The grout available was a commercial grout, which has high strength and flow ability.

The mechanical properties of the composite course were obtained in the form of Marshall stability, Indirect tensile Resilient Modulus, Indirect Tensile strength test at varying temperatures, whereas, the performance parameters were evaluated in the form of Dynamic Fracture Energy, Abrasion resistance test and Moisture Induced Stress test (MIST). The pavement structural analysis and design has been done with the help of these mechanical properties.

CHAPTER 2

2. LITERATURE REVIEW

2.1 GENERAL

It was well established in the literatures that a high void mix must have sufficient voids to allow the grout slurry to flow easily. Anderton (2000) has suggested that, for full depth penetration, the volume of void require is about 25 % to 35 %.

2.2 EFFECT OF AGGREGATE GRADATION

Husain et al. (2014) has conducted a research study on the effect of various aggregate gradations on the physical properties of Cement grouted bituminous mix. Two types of cementitious materials including ordinary Portland cement (OPC) and silica fume (SF) which will help in obtaining the early strength of the grout, with the adequate amount of super plasticizer to improve the workability of the slurry were used to prepare grout mixture and three aggregate gradations designated as G1, G2, G3 ranging from the most porous to the least High Void Bituminous mix skeleton such as 33.4 %, 32.5 % and 28.8 % respectively, were selected in this study to investigate and compare the physical properties and strength such as Volumetric properties, Durability and Compressive strength of the CGBM from the selected three aggregate gradations. Studies carried out by Hassan et al. (2002) showed similar values compared to the selected aggregate gradations in this particular study. The results demonstrated that as the aggregate gradation gets coarser, the increment in volume of voids is higher and the filling process of fluid grout becomes easier and the abrasion resistance was found to be inversely proportional to the changes in compressive strength of CGBM at both early and later ages.

In another study, Setyawan (2005) has compared the effect of three different aggregate gradations denoted as A, B and C on the void content of High Void Bituminous mix mixes. The maximum aggregate size used in gradation A was 14 mm while in gradation B and C it was 10 mm. The fines content in A, B and C were 8%, 15% and 25% respectively. Using these aggregate gradations, Setyawan (2005) prepared compacted High Void Bituminous mix mixes with same bitumen content. He recommended that porosity of the High Void Bituminous mix was the most important property that depicts the ease with which grout slurry would flow through the voids. A higher value of porosity implied that grout penetration would take place more easily.

He observed that using gradations B and C decreased the porosity of the mix by 1% and 3% respectively as compared to gradation A. Pie et al. (2016), has identified that both marshal stability and density got decreased with the increase in porosity of the mix.

2.3 EFFECT OF BINDER PROPERTIES AND GRADES OF BINDER

Previous studies show that, the criteria for choosing the optimum bitumen content for High Void Bituminous mix is that the aggregates should be properly coated without the binder being drained down as per ASTM criteria, so that there should not be any excess of binder which will clog the voids.

This phenomenon of effect of Binder content on High Void Bituminous mix skeleton has been well established by Ary Setyawan et al. (2005). His study clearly inferred that void content reduces with the increase in binder content. Moreover, increase in the binder content of modified binder reduces the maximum specific gravity and bulk density because of the thick coating. He has also investigated the effect of SBS polymer modified bitumen on the mechanical properties of High Void Bituminous mix. He inferred that use of modified binder improves the abrasion resistance and also increases the stiffness. He had also observed that use of cellulose fibers to prevent drain down decreases the void content but it had significantly increased the tensile strength, on the other hand abrasion resistance decreases, reason being use of cellulose fibers made the mix less workable.

In another study, C.P Plug et. al (2006), had incorporated grouted composite mix by using PMB 40 binder, he had inferred that increase in the binder content increases the indirect tensile strength, toughness of the grouted composite.

2.4 EFFECT OF CEMENTITIOUS GROUT ON THE COMPOSITE COURSE

The cementitious grout should have optimum flow ability for full depth flow of grout through the High Void Bituminous mix skeleton. The full depth flow ability depends upon the composition of the grout and water content. Marshall Flow cone test shall be conducted to determine the optimum flow at desired compressive strength. Much higher strength growth may develop thermal stresses in the composite course making the mix more rigid. Koting et al. (2014) has identified two optimum grout compositions consisting of OPC and white cement and OPC and Silica fume (SF). Pure OPC samples were also made as control samples for comparison. All the trials were made at Water content of 0.3 with two Super Plasticizer (SP) dosages, 1.5% and

2% by mass of cement. For the first grout mix design study, the authors replaced OPC by White Cement starting from 5% to 100% at each of the SP dosages. For the second type of grout composition the OPC was replaced with SF by 5% and 10% at each of the SP dosages. Then the flow ability and compressive strength of the grouts were measured. The lowest flow ability was achieved at 2% SP dosage and 5% replacement of OPC for both WC and SF. The flow times were found to increase for further replacement. With WC the highest compressive strength was achieved at 2% SP, 80% OPC and 20% WC. With SF the compressive strength did not vary much for the dosages used in this study. The authors observed that at the same replacement level compressive strength of hardened grout increased with increase in SP dosage. This was attributed to the fact that addition of SP led to a better dispersion of cement particles, which helped in hydration. Overall from this study the authors recommended that for superior grout performances, SF might replace OPC up to 10% and WC may replace OPC up to 20%.

Pei et al. (2016) investigated an experimental study on composition design and performance validation of high-performance cement paste (HPCP) as a grouting material for semi-flexible pavement. They mixed three different types and dosages of additives with HPCP. Polycarboxylate Super plasticizer, expansion admixture and air-entraining agents were used as additives. These additives reduce the water susceptibility of the slurry, which led to high initial strength of the grout as compared to simple OPC paste. The findings of this study revealed that each additive had a distinct influence on the fluidity, strength, and drying Shrinkage of HPCP, and HPCP showed good workability with the compound addition of the three additives.

Hasan et al. (2002), he had formulated cementitious grout by considering five combinations including OPC, silica fume and fly ash. The grout were designed o improve the strength, maintaining the high workability with reduced water/grout ratio. The composition is shown in Table 2.1 below,

Table 2.1

Composition of grout (Hasan et al. 2002)

Slurry	Composition	W/G ratio
OPC	100 % OPC	0.28
SF	OPC: SF (95:5)	0.28
FA/SF	OPC: FA: SF (65:30:5)	0.28
SBR	OPC: SF: SBR (75:4:21)	0.25

Grout designed with SF showed highest compressive strength of 28 days of about 115 MPa, similarly OPC also had the same compressive strength as that of SF mix, whereas Fly ash/Silica Fume showed lowest compressive strength of about 85 MPa. On the other hand use of SBS/SBS-Sand reduced the compressive strength to 60 MPa. Porosity, Permeability and Shrinkage properties were also tested, while porosity values remains almost similar for all the cementitious binder, SF and FA reduces the permeability and also reduces the shrinkage strain due to the refinement of the pore structure of the grout mixes.

Raju et al. established a relationship between void ratios of dry aggregates (Which is measured by measuring the density of aggregates in Dry Rodded technique) to the volume of cement grout penetrated into the porous mix. It was seen that with the increase in the voids in the dry aggregates, the volume of cement grout penetrated also increased. This means that higher void content in dry aggregates, i.e. more the open gradation more will be the porosity of the mix.

2.5 GROUTED COMPOSITE COURSE

2.5.1 Effect of Aggregate Gradation on properties of CGBM

Aggregate Gradation has a primary effect on the properties of the cement grouted composite course, which ensures full depth flow of the grout. In a study Oliveira et al. (2006) has investigated the effect of aggregate size and gradations. He has considered a standard mix with aggregate gradation of 10mm NMAS and compared with a mix prepared with 14 mm, 20 mm NMAS gradations and graded 20 mm – 6.3 mm aggregates. However, he has kept the binder constant to a percentage of 4.1 % for all the mixes. He has also determined the fatigue life. It was inferred that, change in the aggregate gradation had a very less effect in the stiffness modulus. Similarly, use of broader gradation had a very little impact on the stiffness modulus values. Furthermore, all the mixtures exhibited similar fatigue life except that of 20 mm single sized aggregates, which may be attributed to the fact that of the smaller ratio between the depth of the beam and the nominal maximum size of aggregates.

Hussain et al. (2010), has investigated the effects of three aggregate gradations (G1, G2, and G3) starting from lower limit, middle limit to upper limit on the volumetric and mechanical properties of composite wearing course. It was identified that, with the increase in aggregate gradation (G1, G2 and G3) from lower limit to upper limit, voids in mix increases (9.93 % to 11.60 %) and bulk density decreases. Aggregate gradation

G1 gave the maximum value of stiffness modulus as well as compressive strength. Bulk density, compressive strength and stiffness modulus are related inversely to VIM (voids in mix). Lower value of VIM means, more volume of grout is penetrated, which makes the composite mix stiffer and improved the properties. Thus, the author concluded that coarser gradation would lead to more flow of grout, which will make the mix stiffer.

In another study carried out by Ding et al. (2011), it was inferred that CGBM samples more homogenous gradation exhibits better strength and durability, whereas, in case of consecutive gradation, the pore structure formed were more complicated. The author had concluded that use of homogenous gradation will lead to more interconnectivity between voids and proper stone on contact in the porous mix, which will allow the grout slurry to penetrate to the full depth of the mix.

2.5.2 Effect of Binder content and Grade on the properties of CGBM

Ary Setyawan et al. (2005), investigation was concerned with the design and evaluation of CGBM, by incorporating during range of mixtures by using different types of bitumen and bitumen content. The author has used three-binder grades namely 100-Pen straight run bitumen, 50-Pen straight run bitumen, and SBS Modified bitumen at different binder content of 4 %, 4.5 % and 5 %. It was seen that increase in binder content reduces the porosity of the High Void Bituminous mix. Moreover, over 4% binder content binder drain down was evident. For rest of the study he has considered 4% as standard optimum binder content with different grades of binder to determine the volumetric properties and Mechanical properties of the composite course. The results indicated that grades of binder have a significant impact on the properties of composite course. 50-Pen fiber reinforced specimen gave the highest value of ITS and stiffness modulus followed by 50-Pen straight run binder. Similarly, compressive strength for 50-penetration binder mix is higher as compared to SBS modified binder mix. SBS modified bituminous mix gave less stiffness modulus values as well lesser ITS value as compared to 50-pen and 100-pen bituminous mixes. On the other hand, SBS modified bituminous mix showed excellent resistance to abrasion (Cantabro loss almost half to that of straight run penetration grade bitumen) because of the enhanced elastic properties as compared to the penetration grade binders.

Oliveira et al. (2006) studied the effect of different grades and content of bitumen on cement grouted composite mix. The author in his study basically investigated the effect of harder grade bitumen and different content of bitumen on fatigue life, thermal cracking and stiffness modulus of the composite course. The authors made a standard mix with 200-pen binder at 4.1 % binder content and the properties were compared with two other variations in the mix, one with the 50-pen binder at 4.1 % binder content and other were prepared at 1.5% and 3% binder content with the same grade of binder as that of standard mix. The author inferred that harder grade binder (50-pen) and reduced binder content increases the stiffness modulus, which might be evident from the fact that reduction in the percent binder volume is directly, related to that much volume being filled by grout and so the VMA reduces accordingly. However with the use of harder grade and low binder content at same grade of 200-pen, resistance to thermal cracking reduced considerably. The samples got thermal cracks at much lower strains as compared to the standard mixture, which depicts that reduction in the binder content or use of harder grade binder makes the composite mix more brittle. The fatigue life did not show much variation and for all the grouted macadam tested, results lied on a single fatigue characteristic.

S.E Zoorob et al. (2002) have investigated an interesting domain in cement grouted composite surface. He redesigned the hot mix High Void Bituminous mix in cold mix cold laid High Void Bituminous mix skeleton using bitumen emulsion, which will eventually reduce the initial cost of grouted macadam. A range of tests was carried out like compressive strength, Indirect Tensile Stiffness Modulus, Dynamic creep and shrinkage test and the results were compared with hot mixed grouted macadam. It was reckoned that hot laid grouted macadam gave higher compressive strength as compared to cold laid grouted macadam, although it was still adequate for highway application. For stiffness modulus, cold mix cold laid grouted macadam has higher stiffness than hot mix laid macadam at higher test temperatures. The reason being the binder coating and reduces binder film thickness in cold mix macadam were not uniform, apparently resulted in more aggregate to aggregate contact points, where the grout stiffness will dominate, whereas, in hot laid macadam binder coating is quite uniform and therefore will influence the stiffness of the mix to a somewhat greater extent.

Although cold mix grouted macadam showed higher creep stiffness as compared to hot mix grouted macadam at higher test temperature of 60°C, but the shrinkage behavior remained same for both hot mix and cold mix grouted macadam.

2.5.3 Performance of CGBM with respect to conventional BC

G. Bharath et al. (2019) has investigated the laboratory and field evaluation of CGBM. The author had evaluated properties such as Indirect Tensile strength (ITS), Resilient Modulus and Dynamic modulus at different test temperatures. For performance evaluation rutting, Fracture energy, Moisture Resistance, and Durability were also evaluated for the mixes. Both these mechanical properties as well performance properties were compared with conventional BC layer. The author had also laid a test section of CGBM on site for its field performance evaluation. It was evaluated that, CGBM has ITS value of almost 2-2.5 times and resilient modulus value 3-3.5 times higher than that of BC. Moreover, the temperature susceptibility of CGBM mix is very less as compared to BC when tested at different test temperatures. ITS and resilient modulus value got decreased with the increase in temperature, which depicts the flexibility of the CGBM mix.

The resilient modulus of BC was almost decreased by 82% when the test temperature changed from 25°c to 45°C, whereas, in case of CGBM the reduction is only 45 %, which shows its less temperature susceptibility. CGBM samples showed compressive strength, approximately 6.5 and 2.75 times higher values as that of BC mix. Dynamic modulus values of CGBM were found to be substantially higher than that of BC samples. In case of rutting phenomenon CGBM samples under rut test showed much lesser rut depth as compared to BC samples. Full depth grouting was observed from the cores collected from field samples. Modulus values and indirect tensile strength (ITS) values of these cores were observed to be much within limits.

Joel R. M. Oliveira et al. (2016) carried out several laboratory tests for a better understanding of fatigue performance of grouted macadam vis-à-vis Dense Bituminous Mix. The conventional mix taken was DBM 50 and two point and four point bending tests were performed to understand the fatigue performance. The fatigue performance of the grouted macadam was different from that of DBM samples. Therefore in his study author performed the fatigue test beyond the failure criteria (50 % initial stiffness reduction) until the stiffness modulus had been reduced to 10 % of its initial value. It was seen that grouted macadam continues performing well, after the

50 % reduction in the initial stiffness, where as in DBM after 50 % stiffness reduction, there developed a second point of inflection, as shown if Figure 2.1 and 2.2 below.

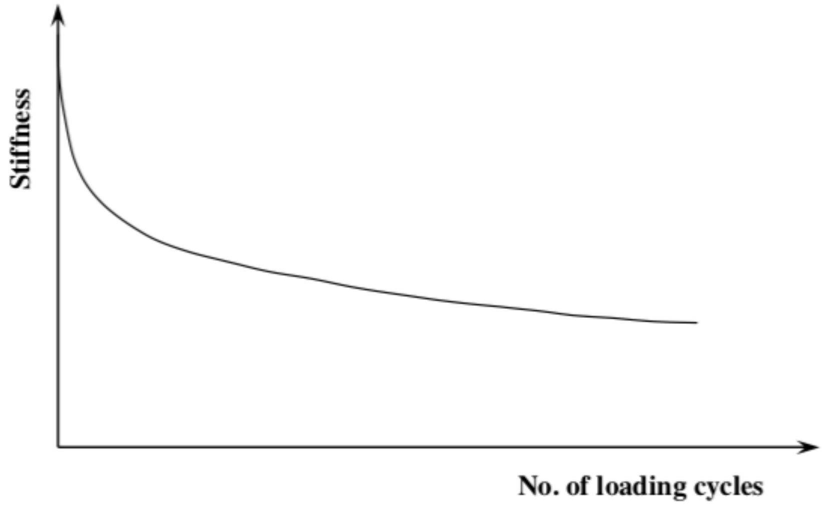


Figure 2.1: Fatigue test on Grouted macadam (Joel R. M. Oliveira et al. 2016).

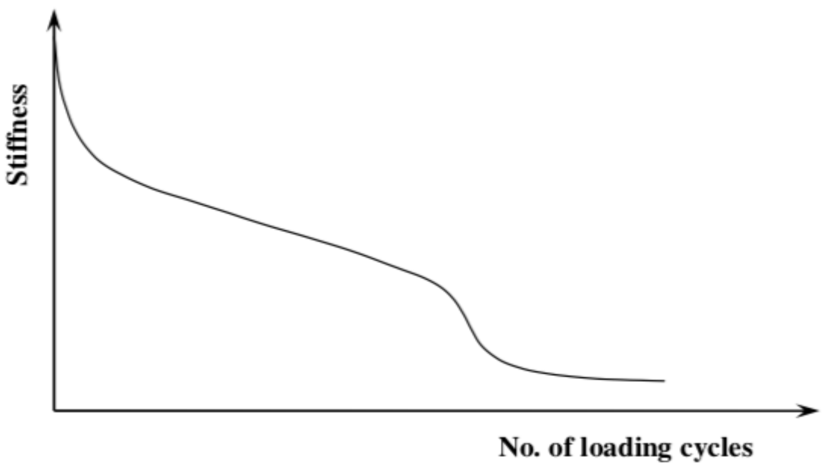


Figure 2.2: Fatigue test on DBM (Joel R. M. Oliveira et al. 2016).

The author had also inferred that, the crack propagation in grouted macadam is all together different from DBM, in grouted macadam, the cracks propagate through bitumen films surrounding the aggregate and only through the grouts at weak spots (where the voids are very closely spaced and grout volume was less in between). Whereas in DBM, the crack propagated through the mastic particles (bitumen mixed will finer aggregate particles), which filled the spaces between larger aggregates as shown in Figure 2.3 and 2.4 below.

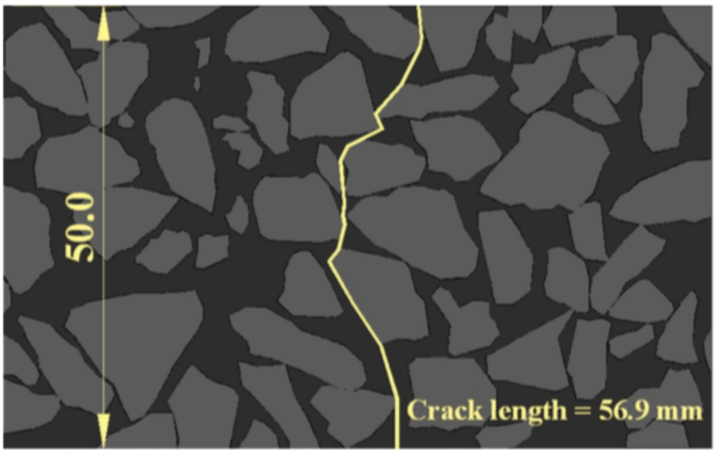


Fig. 2.3: Crack propagation in DBM (Joel R. M. Oliveira et al. 2016)

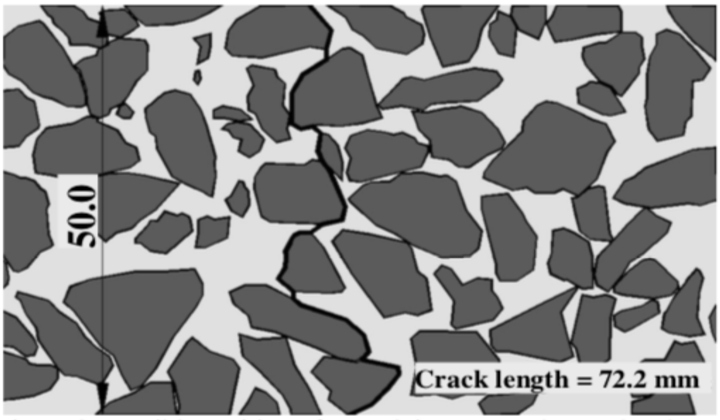


Fig. 2.4: Crack propagation in Grouted Macadam (Joel R. M. Oliveira et al. 2016)

Hou at al. (2015) investigated the rut resistance, durability, and low temperature fracture strength of grouted macadam with conventional asphalt AC-16. For the High Void Bituminous mix the binder content was chosen to be 3.8 %, which gave void content of 30 %. Moisture stability and rut resistance of grouted macadam were found to be much superior as compared to conventional asphalt mix. However, low temperature (-10^{o}c) flexural strength of grouted macadam was found to be less than conventional asphalt, which depicts the rigidity/brittleness of the grouted macadam. The fatigue life was also observed to be longer than conventional AC-16 mix.

CHAPTER 3

3. MATERIALS

3.1 GENERAL

As already discussed in the previous chapters, properties of materials plays a crucial role in the design mix of grouted composite macadam, therefore it is of utmost importance to study the properties of individual materials required to develop the composition for cement grouted composite macadam. In this chapters properties of various materials such as Aggregates, Binders and Grout are thoroughly presented, which will have a significant impact in the aggregate gradation, binder content and optimum water/grout content for the development of High Void Bituminous mix skeleton and thereby grouted composite macadam.

3.2 AGGREGATES

Aggregates which were used in the preparation of High Void Bituminous mix as shown in Photo 3.1 below were tested for their physical properties such as aggregate specific gravity, water absorption, flakiness and elongation index and aggregate impact value so as to meet the aggregate requirements as per ASTM, Bureau of Indian Standards or MORTH specifications.

Photo. 3.1: Aggregates used for the study

3.2.1 Aggregate Impact value

Aggregate impact value gives the measure of the resistance of aggregates to sudden shock/impact. This test was performed on conformation with IS 2386, Part IV. In this test method, a standard sample of aggregates with sizes ranging from 12.5 mm to 10.0 mm is subjected to loading in the form of 15 blows from a 100 mm diameter hammer as shown in Photo 3.2. The sample aggregates break due to the impact loading. A 2.36 mm sieve is then used to separate the fines from the impact-loaded sample. The percentage of fraction passing the 2.36 mm sieve (W2) relative to initial weight (W1), gives the impact value of the aggregates. A lower value indicates tougher aggregates and vice-versa. The test results are given in Table 3.1.

Photo. 3.2: Aggregate Impact test apparatus

Table 3.1

Aggregate Impact Value

Observations	Trial 1	Trial 2
	Weight, gm	Weight, gm
Initial weight of total aggregate (W1), gm	360.0	365.6
Weight of aggregate passing 2.36 mm sieve (W2), gm	42.8.4	39.82
AIV Value (W2/W1), %	11.8%	10.89%
Average Value	11.34%	

3.2.2 Specific Gravity & Water Absorption

The specific gravity (bulk and apparent) and water absorption of coarse aggregates was determined using ASTM C127 which requires weighing both dry and saturated surface dry (SSD) coarse aggregate in air and weighing saturated aggregate under water as shown in Photo 3.3 and 3.4. The specific gravity (bulk and apparent) and water absorption of fine aggregates was determined using ASTM C128 which requires use of a pycnometer as shown in Photo 3.5. The test results for specific gravity (bulk and apparent) and water absorption of each size fraction of coarse and fine aggregates are given below in Table 3.2.

The specific gravity and Water absorption of coarse aggregates are given by following equations:

$$Bulkspecificgravity(GSB) = \frac{A}{B-C}$$

$$Apparentspecificgravity(GSA) = \frac{A}{A-C}$$

$$WaterAbsorption(\%) = \left[\frac{B-A}{B}\right]100 \tag{3.1}$$

A = mass of oven-dry test sample in air, g,

B = mass of saturated-surface-dry test sample in air, g, and

C = apparent mass of saturated test sample in water, g.

Photo. 3.3: Water bath for 24 hours

Photo 3.4: SSD condition

The specific gravity of fine aggregates was calculated using following equations:

$$Bulk specific gravity (GSB) = \frac{A}{B + S - C}$$

$$Apparent specific gravity (GSA) = \frac{A}{B + A - C}$$

$$Water absorption, \% = \left[\frac{S - A}{A}\right] X100$$

(3.2)

A = mass of oven dry specimen, g

B = mass of pycnometer filled with water

C = mass of pycnometer filled with specimen

S = mass of saturated surface-dry specimen

Photo 3.5: Specific gravity and W/A measurement for Fine Aggregates

Table 3.2

Specific Gravity and Water Absorption

Aggregate specimen		Bulk Specific Gravity (Gsb)		Apparent Specific Gravity (Gsa)		Water Absorption, %	
Passing/Retained IS sieve, mm	Trial No.	Trials	Average Value	Trials	Average Value	Trials	Average Value
19.0/13.2	1	2.79	2.79	2.82	2.83	0.2	0.3
	2	2.8		2.83		0.3	
13.2/9.5	1	2.64	2.69	2.67	2.81	0.4	0.5
	2	2.75		2.94		0.5	
12.5/9.5	1	2.76	2.77	3.01	3.00	0.2	0.2
	2	2.78		3.00		0.2	
9.5/4.75	1	2.84	2.84	3.02	3.02	0.2	0.2
	2	2.85		3.01		0.3	

9.5/6.3	1	3.02	3.00	3.04	3.02	0.3	0.2
	2	3.01		3.03		0.2	
6.3/2.36	1	2.74	2.66	3.02	2.86	0.2	0.6
	2	2.58		2.93		1.3	
4.75/2.36	1	2.97	2.94	3.02	3.27	0.4	0.4
	2	2.91		3.72		0.2	
2.36/0.075	1	3.07	3.16	3.32	3.38	2.4	2.0
	2	3.21		3.42		1.9	

The combined specific gravity and water absorption of the aggregates are given below in Table 3.3:

Table 3.3

Combined Specific gravity and Water Absorption

Properties	
Combined Specific Gravity	2.85
Water Absorption	0.55

3.2.3 Flakiness and Elongation Index

Elongated and flaky particles have a large surface area relative to its small volume; hence it decreases the workability and performance of mix. The particle is considered as elongated if its length is more than 1.8 times the mean sieve size of the size fraction to which the particle belongs. Similarly, the particle is considered as flaky if its thickness is less than 0.6 times the mean sieve size of the size fraction. Combined FI & EI must be less than 35% as per IS: 2386 Part-I and MoRT&H specifications, as shown in the Table 3.4. Photo 3.6 shows standard length and thickness gauge.

Photo 3.6: Standard thickness and length gauge

Table 3.4:

Flakiness and Elongation Index

Trial 1		Trial 2	
Flakiness Index	12.386	Flakiness Index	12.880
Elongation Index	21.540	Elongation Index	22.012
Combined FI & EI	33.926	Combined FI & EI	34.892

3.3 BINDER

Different grades of binder were being used for this study. As discussed previously in the literature review, grades of binder have notable effect on the properties of High Void Bituminous mix skeleton; therefore, it is very much important to determine the properties of binder. VG 30, VG 40, PMB 40 and Bitumen emulsion of SS grade were used for the study as shown in the below Photo 3.7.

a) Bitumen Emulsion (SS) b) PMB40, VG40, VG30 (from left)

Photo 3.7: Binders used for the study

The mixing and compaction temperatures for VG 30 were established at 152^{o}C - 157^{o}C and 138^{o}C – 145^{o}C respectively and for VG 40, the mixing and laying temperatures were established at 165^{o}C – 168^{o}C and 155^{o}C – 160^{o}C respectively corresponding to viscosity ranges of 0.17 + 0.02 Pa.s and 0.28 + 0.03 Pa.s respectively as shown in Figure 3.1. In case of polymer modified binder, the mixing and laying temperature were considered in between 150^{o}C -170^{o}C and 140^{o}C -155^{o}C respectively as per IRC SP: 53-2010.

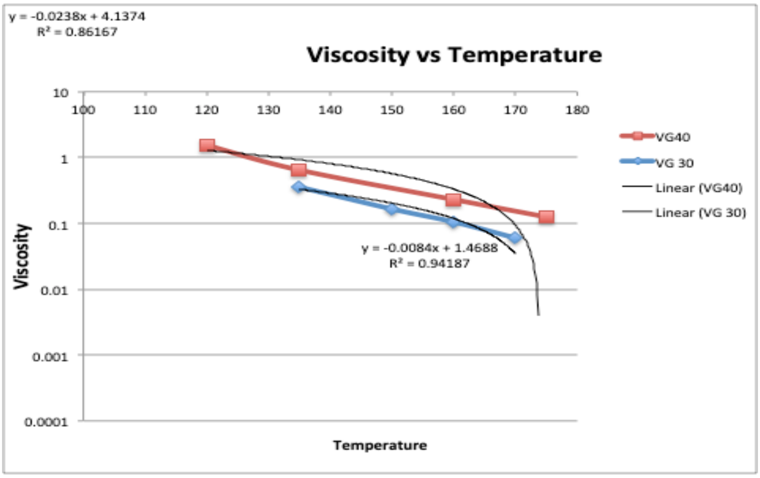


Fig. 3.1: Graph showing relation between Viscosity and Temperature

Viscosity at 135°C for VG 30 and VG 40 and viscosity at 150°C for PMB 40 were performed with the help of Brookfield viscometer as per ASTM: D4402, for other properties such as specific gravity, softening point, penetration and ductility, tests have been performed on the binders as per IS: 1202, IS: 1203, IS: 1205 and IS: 1208 respectively as shown in the Table 3.5 below,

Table 3.5

Binder Properties

Property	VG 30	VG 40	PMB 40	Test Method
Specific Gravity, 27 °C	1.01	1.013	1.032	IS: 1202
Penetration, 25 °C,100g, 5s (dmm)	65	50	39	IS: 1203
Softening Point (°C)	50	52	70	IS: 1205
Ductility, 27 °C (cm)	Above 100	Above 100	-	IS: 1208
Elastic Recovery, 15°C (cm)	-	3709	75	IS: 1208
Viscosity at 60 °C (Poise)	2894	3709	-	IS: 1206 (Part 2)
Viscosity at 135 °C (cst)	575	645	-	IS: 1206 (Part 3)
Viscosity at 150°C (Poise)	-	-	7.5	IS: 1206 (Part 2)

3.3.1 Properties of Emulsion

Emulsion used for the study was tailored made cationic emulsion and falls under SS category. Several tests such as residue by evaporation (IS: 8887-2004), Viscosity by Saybolt Furol Viscometer at 25°C (IS: 3117-2004), Penetration and Ductility on residue were performed according to IS: 1203 and IS: 1208 respectively.

3.3.1.1 Residue by Evaporation

This test provides the idea of amount of bitumen present in the emulsion also gives the water content in the emulsion. 50gms of cationic emulsion sample is first heated in the oven for 2 hours at constant temperature of 163±2.8°C as shown in the Photo 3.8 below. At the end of 2 hour, the residue of beaker is stirred well and further heated for 1 hour at 163±2.8°C in oven. The weight remaining in the beaker in percentages of the residue is determined after cooling the beaker at room temperature. This test was performed in accordance IS: 8887-2004.

Photo 3.8: Residue test on Emulsion

3.3.1.2 Saybolt Furol Viscosity

The viscosity of bitumen emulsion was measured at 25°C by using Saybolt Furol viscometer as per IS: 3117-2004. In this test, efflux time in seconds was noted for 60 ml of the emulsion at 25°C temperature to flow through an orifice of a specific size. The higher the viscosity of the bitumen more time will be required for a quantity to flow out. Water bath shall be maintained to maintain the proper test temperature during the test.

*Photo 3.9:*SayboltFurol Viscosity test

All the properties of bitumen emulsion used for the study are tabulated in the Table 3.6,

Table 3.6

Physical properties of Emulsion

Property	Value Obtained	Test Method	Value Specified
Viscosity SayBolt Furol (Viscometer Seconds at 25°C)	33	IS: 3117-2004	30-150
Tests on Residue			
Residue by Evaporation, % Min	66	IS: 8887-2004	60
Penetration, 25°C, 200g, 5s (d mm)	77	IS: 1203	60-220
Ductility, 25°C (cm)	72	IS: 1208	Min. 50

3.4 CEMENTITIOUS GROUT MATERIAL

Grout is a cementitious material (Photo3.10), which primarily consists of cement, fly ash, sand (fine sand), and admixture (super plasticizer). The selection of grout material depends upon the type of medium to be grouted and the purpose of grouting. As per the scope of this project, medium to be grouted is the porous bituminous mix. The highly porous bituminous mix may contain open type, semi open type or closed type of voids; depending upon the interconnectivity of these voids. The requirement of rigidity in the composite material is justified by the quality and water content of grout material, once it sets into the voids of bituminous mix. Grout material (Photo 3.10)

Photo 3.10: Grout material used for the study

used in this project was industrially developed by joint venture of CSIR-CRRI and M/s International Combustion Limited. The suitability of the grout material was checked with respect to two main decisive criterions. The first one being consistency of grout slurry showing high flow ability such that full depth of grout penetration into the bituminous mix is achieved, which was tested with Marsh Flow Cone test; and the second being strength (compressive strength) of the grout material which was measured on grout cubes of 50 cm^2 face area. These two primary requirements of grout were checked as per the following tests.

Workability of the grout slurry is measured using Marsh Flow Cone. Flow ability of grout is defined as the time taken for one liter volume of grout to flow through the Marsh cone via circular opening at bottom of diameter 10 mm. Flow ability of grout material being checked using Marsh Flow Cone is shown in Photo 3.11. Water was added by weight of the grout. It was observed, that the grout needs to be thoroughly mixed with water as shown in Photo 3.12 and 3.13, in order to achieve best consistency in terms of low Marsh Flow Value; as Marsh Flow Time was found to be dependent on mixing efficiency (mixing speed and mixing time). It was achieved that 20 minutes of mixing time is the optimum time of mixing which was shared by the company from where grout was procured and samples were made at labarotary to check the validity. It was observed that after 20 minutes of mixing there was no change in the flow time at a pre determined water content. At first grout shall be manually mix for 5 minutes after adding half of optimum water content and once the lump breaks, after that another half of the water content shall be added and mixed for another 2-3 minutes before placing it in a mechanical mixer as shown in Photo 3.12 and 3.13 respectively. Once achieving the optimum mixing in terms of mix time, the grout flowabilty in terms of Marsh Flow Time is shown in Figure 3.2.

Photo 3.11 Measurement of marsh flow time for grout slurry progress

Table 3.7

Flowability of Grout with respect to water content

Water Content (%)	Flow (Seconds)
16	103
18	73
20	46
22	31
24	17
26	15

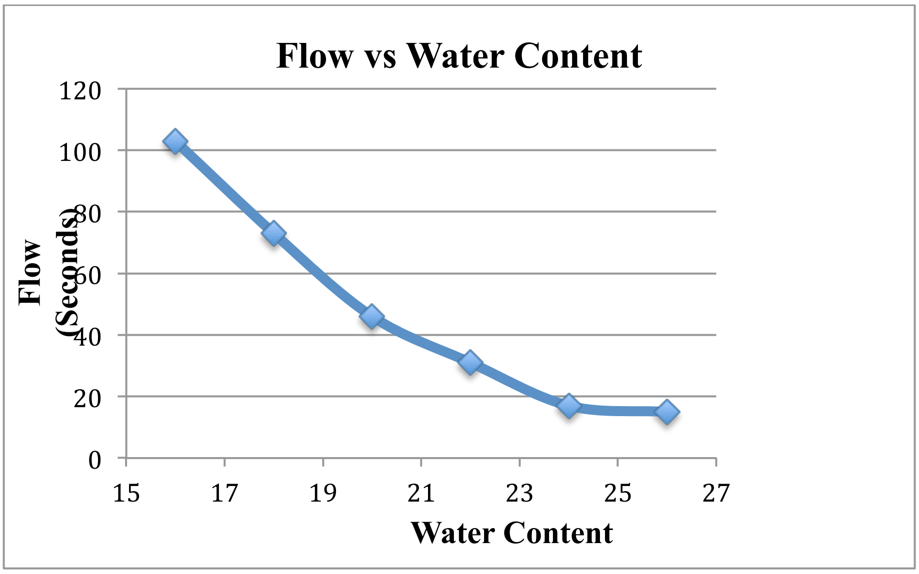


Fig. 3.2. Variation of grout flowability w.r.t. Water content

Photo 3.12. Manual Mixing of Grout

Photo 3.13. Mechanical mixing of Grout slurry

For strength evaluation, standard grout cubes having 50-cm^2 area (Figure 3.14), were casted and tested for their compressive strength. Compressive strength of the grout as determined on cubes after 7 days of curing with respect to varying water content shown graphically in Fig. 3.3 and in Table 3.8.

Photo 3.14. Casting of Cubes for Strength test

Table 3.8

Strength of Grout at varied water content.

Water Content (%)	Grout Strength (Mpa)
18	126
20	98
22	75
24	59

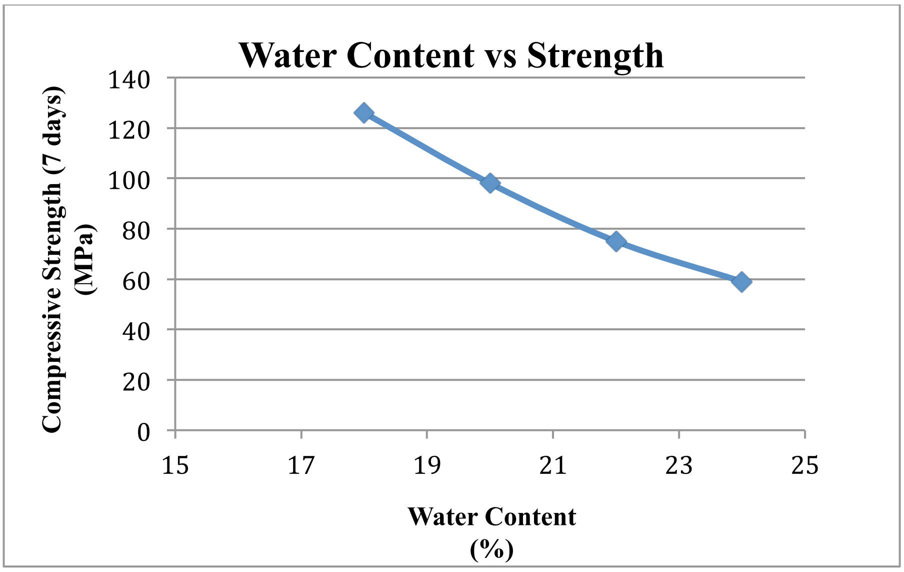


Fig. 3.3. Variation of grout compressive strength with water content

The above detailed inferences clearly depict that flow ability of the grout material increases with increase in water content with subsequent decrease in the grout compressive strength. However according to literature higher strength of grout will increase the rigidity and decrease the flexibility of the mix and might also rise to temperature stresses. Minimum grout strength required should be of 30 Mpa (J.Oliveira, N.H. Thom and S. Zoorob, 2006). This combined generalized behavior of grout is shown graphically in Fig. 3.4.

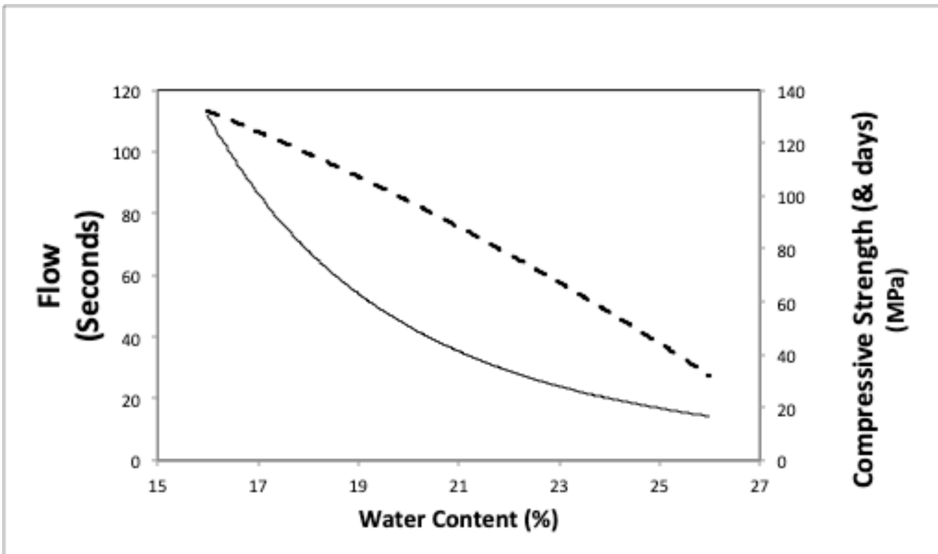


Fig: 3.4. Variation of grout flow ability and compressive strength with respect to mixing water content.

CHAPTER 4

4. MIX DESIGN OF HIGH VOID BITUMINOUS MIX

4.1 SUITABLE AGGREGATE GRADATION

Two different aggregate gradations were considered to choose from them, the most suitable gradation for the high voids bituminous mixes. These aggregate gradations are described in Table 4.1. Out of these, one is the gradation suggested by MoRT&H for 13.2 nominal maximum aggregate (NMAS) size for surface dressing while the other gradation is Open Graded Friction Course as per ASTM D 7064.These two aggregate gradations were considered as alternate gradations likely to result in sufficient air void content in bituminous mix. The grain size analysis of the two open gradations and the one for conventional dense graded bituminous mix is given in Table 4.2. The graphical representation of the two aggregate gradations considered for high voids bituminous mix design is shown in Fig. 4.1.

Table 4.1

Aggregate gradations considered in this study

Gradation Detail
Mid-point gradation of OGFC with NMAS of 12.5 mm.
Mid-point gradation of Surface Dressing with NMAS of 13.2 mm

Table 4.2

Aggregate size distribution for different gradations

Sieve Size	OGFC	CGBM Gr-I	BC
19.0	100.00	100.00	100
13.2	-	85-100	90-100
12.5	85-100	-	-
9.5	35-60	0-40	70-88
6.3	-	0-7	-
4.75	10-25	-	53-71
2.36	5-10	0-2	42-58
1.18	-		34-48
0.600	-		26-38
0.300	-		18-28
0.150	-		20
0.075	2-4	0-1.5	10

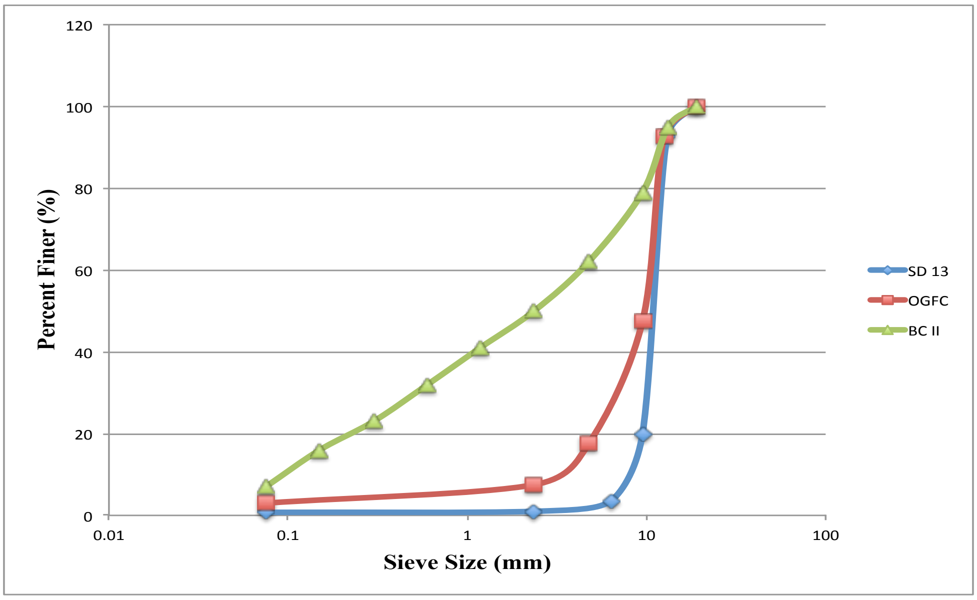


Fig. 4.1. Different aggregate gradations used for the study

Out of these two trial gradations CGBM GR-I was found to be suitable for construction of high voids bituminous mixes; the reason for such consideration is discussed later. So, the gradation CGBM GR-I was adopted for cement grouted bituminous mixes was typically chosen to have a complete detailed comparative assessment of CGBM over conventional BC.

4.2 COMBINED BULK SPECIFIC GRAVITY OF AGGREGATES

The combined bulk specific gravity was calculated according to ASTM C127 as shown in Equation 4-1. Combined bulk specific gravity is an important parameter to determine the stone on stone contact and proper interconnectivity between the voids. The combined bulk specific gravity of both the gradations are shown in Table 4-3.

$$G = \frac{1}{\frac{P_1}{100G_1} + \frac{P_2}{100G_2} + \cdots \frac{P_n}{100G_n}} \tag{4.1}$$

Where

G	Average relative density (specific gravity)
G1,G2,...Gn	Appropriate average relative density (specific gravity) values for each size fraction
P1, P2,..Pn	Mass percentages of each size fraction present in the original sample.

Table 4.3

Combined bulk specific gravities of aggregates used for different gradations

Gradations	Bulk Specific Gravity
CGBM GR-I	2.85
OGFC	3.05

4.3 DETERMINATION OF OPTIMUM BINDER CONTENT (OBC)

Their drain down test determined the Optimum Binder Content of both high void bituminous mixes in accordance to *ASTM D6390*.

4.3.1 Drain Down Test (ASTM D6390)

According to the ASTM test method, a sample of the asphalt mixture to be tested is prepared in the laboratory. The sample is placed in a wire basket, which is positioned on a plate or other suitable container of known mass as shown in Photo 4.1. The sample, basket and plate are placed in a forced draft oven for 1 h at a pre-selected temperature (160^{o}C). At the end of 1 h, the basket containing the sample is removed from the oven along with the plate or container and the mass of the plate or container containing the drained material, if any, is determined. The amount of drain down is then calculated and the amount should not exceed 0.3% of the total weight of the sample.

Photo 4.1. Drain down Test Basket

For both gradations, bituminous mixtures were prepared at different binder content in increment of 0.5 % with VG 30 binder, which is considered to be the standard reference grade for this study. In Figure 4.3, it is observed that the binder drain down loss increases with increase in binder content. Drain down loss criteria of 0.3% (maximum) for high void bituminous mix is considered for selecting the optimum binder content. This restriction of 0.3% is referred from other well-established codal provisions for high voids bituminous mixes like Stone Mastic Asphalt, Open Graded Premix Carpet, etc. Particularly for CGBM GR-I gradation, the binder content required for limiting the drain down to 0.3 % was obtained as 2.43 percent by weight of the mix as shown in Fig.4.2. For OGFC gradation, the binder content to limit the drain down at 3% is 4.7% as shown in Fig. 4.3. The design binder content (OBC) for BC grade II mix was selected to be 5.4 % by weight of the mix corresponding to 4% air voids.

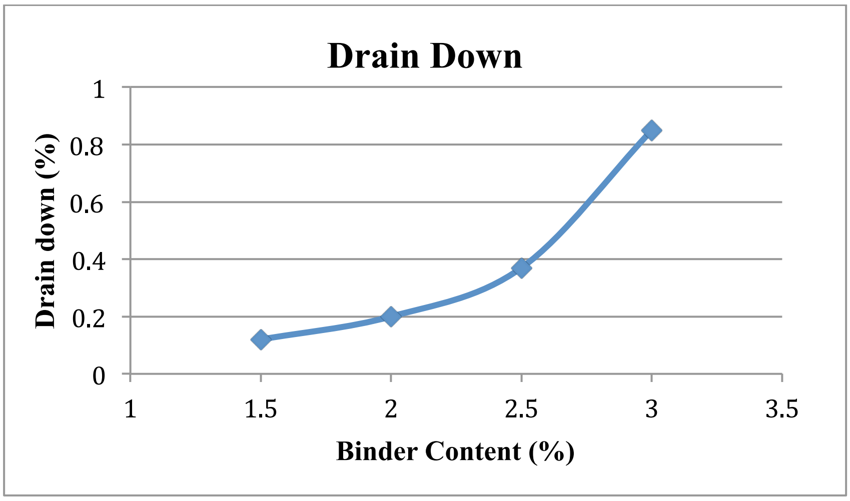


Fig. 4.2. Test Results for Drain Down performed on CGBM Gr-I with VG 30

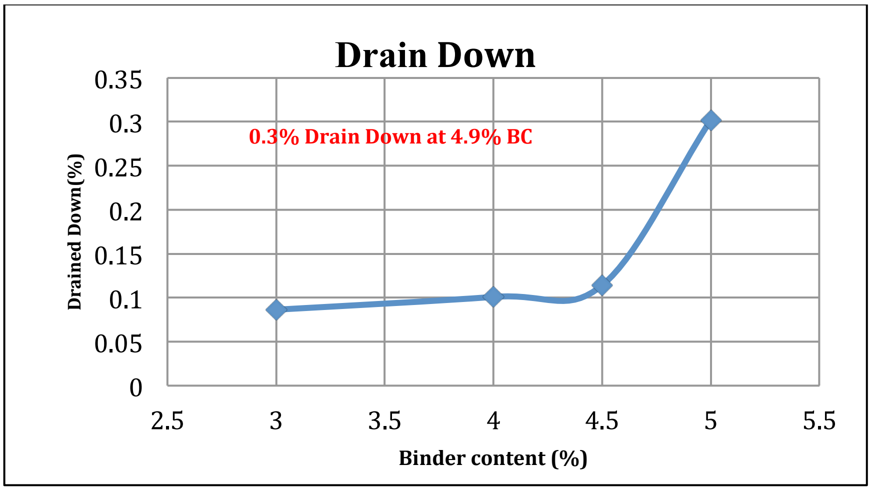


Fig.4.3. Test Results for Drain Down performed on OGFC with VG30

To evaluate the effect of different grades of binder like VG 30, VG 40, PMB 40 and emulsion, optimum binder content was also determined for them. Drain down tests of high porous bituminous mix with VG 40, PMB 40 was carried out as per the procedure mentioned above. Since, criteria of determining the optimum emulsion content is different from that of the hot mix High Void Bituminous mix skeleton, therefore, it is mentioned separately. In CGBM, to develop a high void bituminous mix, primary requirement is, there should be proper bitumen coating on the aggregates. So optimum binder content shall be that much which will serve the purpose of adequate coating of the aggregates. If the binder content is much beyond optimum, there might be a chance of non-interconnectivity between the voids and subsequently the void content will be less and grout might not penetrate to full depth.

Based on reference to optimum binder content for VG 30, accessing the full coating of aggregates by visual inspection and evaluating the volumetric properties of the mix at requisite binder content, optimum binder content for VG 40 and PMB 40 were contemplated to 2.5 %. Volume properties with the mix at 2.5 % binder content are discussed in next clause.

Table 4.4

Optimum Binder Content for different Aggregate Gradations

Binder Grade	Gradation Type	Optimum Binder Content % (Weight of Mix)
VG 30	OFGC	4.7
	CGBM GR-I	2.43
VG 40	CGBM GR-I	2.5
PMB 40	CGBM GR-I	2.5

4.3.2 Optimum Emulsion Content

The optimum emulsion content for emulsion mix High Void Bituminous mix skeleton was carried out in accordance to Asphalt Institute MS-19 Manual. As per the manual, job aggregates and emulsion are mixed to establish optimum emulsion content on the basis of residue runoff (Photo 4.2). Mixtures were prepared with varying emulsion content started at 3% with 1% increment. The emulsion content at residue runoff of 10 grams is the optimum emulsion content. Along with residue runoff, the percentage of coating, which is also a basic criterion to determine the optimal, was evaluated through visual inspection (Photo 4.3) of the mix spread in a paper-lined tray.

4.3.2.1 Procedure

- 2 kilograms of job aggregates of desired gradation (CGBM GR-I) were taken. The graded aggregates are then thoroughly mixed with 2% water by weight of dry aggregates and then covered with clean cloth for around 15 minutes.
- Initial amount of 3% emulsion were added and manually mixed minutes using a trowel. Thereafter, workability of the mix and the amount of coating was checked. Emulsion contents in 1.0% increments are further added to determine the optimum content.
- Immediately after preparation of the mixture, the mixture was transferred to 2.36 mm sieve placed over a tray of known mass. The mix was then allowed to drain in the pan/tray for 30 minutes.
- After that lift the sieve and measure the mass of the tray containing the emulsion drain down/runoff (Photo 4.2). Subtract the empty weight of the tray to determine the exact amount of runoff.
- The tray was then placed in a draft oven at 110^{o}C and dried to a constant mass and weighed to get the residue runoff.
- A graph was plotted between subsequent Emulsion content and Residue runoff, optimum emulsion content is that content at which residue runoff shall be 10 grams.
- The Optimal Emulsion Content determined for our study was 5% against residue runoff of 10 grams as per MS-19 Asphalt Institute (Fig. 4.4).

Photo 4.2: Emulsion Drain down test

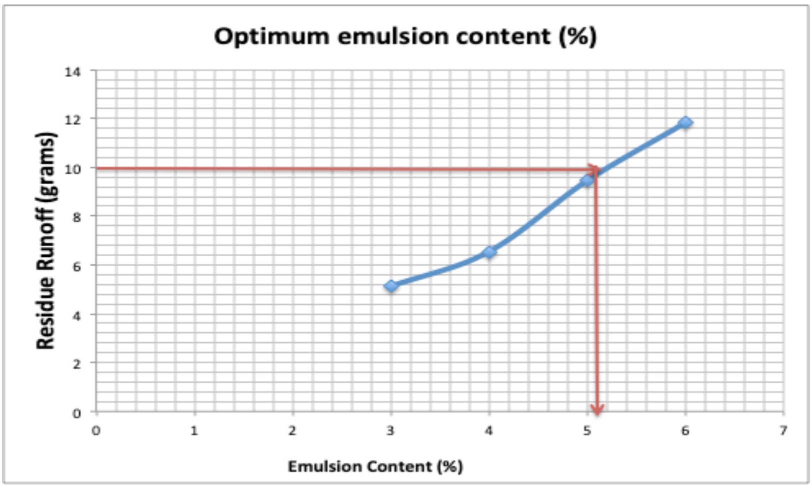


Fig.4.4. Selection of Optimum Emulsion content

Photo 4.3. Level of coating of aggregates at Optimum Emulsion Content

4.4 COMPACTION EFFORT

There are no established criteria for selection of number of blows for the compaction of high void bituminous mix. Marshal compactor with incremental variation of compaction effort was followed for the high void bituminous mixes. Literatures have suggested 50 gyrations or in-between 10 to 75 blows.

4.4.1 Preparing Hot High Void Bituminous Mix

4.4.1.1 Using Marshall Compactor for Hot mix High Void Bituminous mix skeleton

Cylindrical specimens of 101 mm diameter (4 inch) were prepared using a 100 mm diameter split mould for considered aggregate gradations. The Marshall compactor is shown in Figure 4.2. For each aggregate gradation, cylindrical samples were compacted at various blows (20 to 70) of Marshall Hammer with increment of 10 blows applied on single face of the specimen. One of the typical graphs obtained while optimizing the compaction effort i.e. number of Marshall Hammer blows for bituminous mix of CGBM GR-I gradation in shown in Figure 4.5. Figure 4.6 shows variation of compactive effort with number of blows for OGFC gradation. The number of Marshall Hammer blows optimized for both gradations of CGBM can be inferred from Table 4.5.

Total material considered for preparation of one Marshall Sample was taken as 900g, consisting of both aggregates and binder for compacted specimen of 100 mm diameter. As it was observed that, in 900g of mix, height of Marshall Samples was in between 55 to 63 mm at full density. The heated aggregate and bitumen heated at their requisite temperatures as discussed in clause 3.2 were thoroughly mixed until complete coating of aggregates was achieved. The mix was transferred to the mould, which was then compacted using a Marshall hammer. The intact compacted samples were then air-cooled for 1 day before extracting from the Marshall mould as shown in Photo 4.3 and 4.4.

Photo 4.4. Marshall Compactor

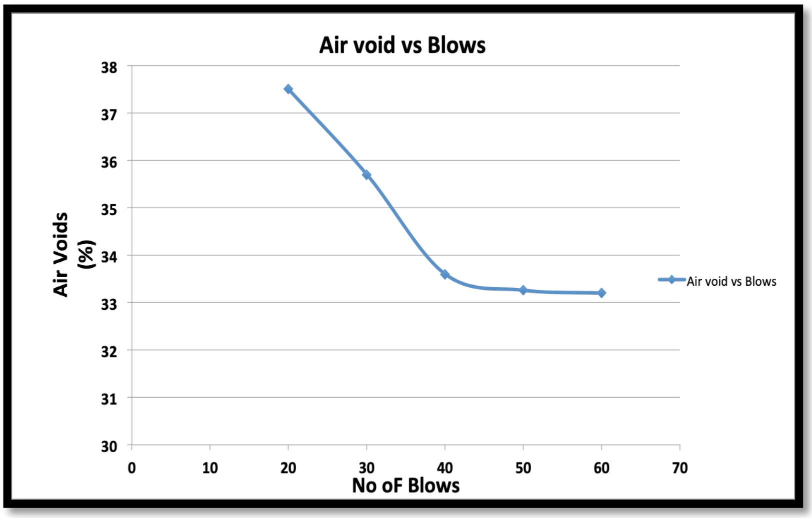


Fig.4.5. Compaction curve in reference to OGFC mix

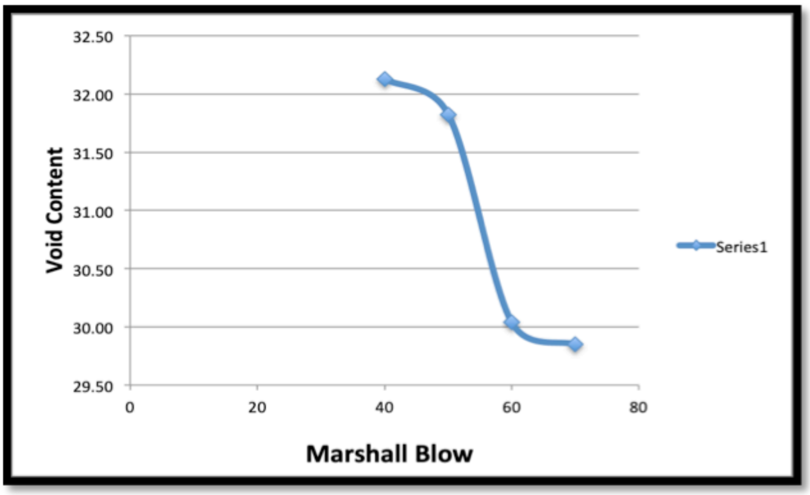


Fig.4.6 Compaction curve in reference to CGBM GR-I mix

Photo 4.5. Compacted Mix of OGFC Gradation

Photo 4.6 Compacted Samples of CGBM GR-I Gradation

Based upon the nature of the study, high Void Bituminous mix of CGBM GR-I gradation was prepared with different grades of binder like VG 30, VG 40 and PMB 40. Since compactive effort depends primarily on the gradation chosen, it was seen that for CGBM GR-I mix prepared with VG 40 and PMB 40, maximum density got achieved at 40 blows. After 40 blows, as seen from the Figure 4.7, the curve remains straight, as the maximum density is achieved. Moreover, it was observed that after 40 blows, higher blows leads to aggregate crushing, considering the above factors 40 blows was optimized for CGBM GR-I gradation to achieve maximum density of the compacted porous skeleton prepared with VG 30, VG 40 and PMB 40.

4.4.1.2 Using Modified Marshall compactor

Modified Marshall Samples of 150 mm diameter and 94.5 mm height (to the ratio of 1.5 as that of standard Marshall Sample) of CGBM GR-I mix with VG 30, VG 40 and PMB 40 were prepared for the requirement of Dynamic Fracture Energy Test as shown in Photo 4.5. Total 3038 grams of mix was taken consisting of aggregates and binder and based on the ratio of compactive effort of standard and Modified Marshall sample, prepared mix was compacted at 60 blows (considering a ratio of 1.5 to the standard) with a modified compaction hammer to achieve the desired maximum density.

Photo 4-7. Modified High Void Bituminous mix Skelton of CGBM GR-I gradation

4.4.1.3 Compaction by Gyratory Compactor

Standard samples of 100 mm diameter and 180 mm height as per ASTM C39 were prepared with the help of gyratory compaction (Photo 4-8) for compressive strength test. Based on the height and density, the amount of sample taken was 2545 grams of both aggregates and binder mix prepared with VG 30, VG 40 and PMB 40. The samples were compacted on the basis of fixing the height of the sample and varying the number of gyrations (Photo 4.9). The average gyrations required producing a sample of height 180 mm is shown in the Table 4.5 below.

Table 4.5

Gyrations required for high void mix with different grades of Binder

Specimen		Avg. Dia.	Avg. Ht.	No. Of Gyrations
		mm	mm	
PMB	1	100	170	92
	2	100	167	94
VG 40	1	100	175	90
	2	100	173	92
VG 30	1	100	178	104
	2	100	178	96

Photo 4.8. Gyratory Compactor

Photo 4.9. High Void Bituminous Skeleton for compression test compacted using Gyratory Compactor

4.4.2 Preparation Of Emulsion Based High Void Bituminous Mix

To prepare a compacted high Void Bituminous mix cold mix with emulsion as a binder, 900 grams of mix consisting of aggregates as per requisite aggregate gradation of CGBM GR-I and tailored made SS2 emulsion were taken to prepare cylindrical marshal specimen of 101 mm diameter and 63 mm height.

4.4.2.1 Preparation of loose mixes

At first, aggregates are mixed with water (1%) approximately to the water absorption of the aggregates, so that the aggregates got charged and they do not soak the water from emulsion, if so, emulsion will break much earlier and will reduce the workability of the mix. Perhaps, this will lead to difficulty in compaction. Since the gradation considered is open graded, there is no need to find the optimum water content, as addition of further water will lead to delay the process of breaking and may lead to runoff of emulsion. After adding of water, the mix was kept idle for 5 minutes so that the water gets absorbed. After that optimum emulsion content at 5 % of the mix was added and thoroughly mixed for 2 minutes to get uniformity. The mix is then transferred to a tray as shown in photo 4.8, and allowed for breaking of emulsion (turning brown to black coating) before compaction.

To have proper insights to the mechanism of breaking in open case of open graded mix, the loose mix was kept under several conditions to scrutinize the breaking phenomenon. Several inferences have been established. In Trial 1, the loose mix was kept in oven at 40°C and at intervals the percentage of breaking was noted down. It was seen that within 20 minutes, the entire loose mix turned black from brown, and the mix became very stiff and difficult to compact. This is because of the open grading; the surface area of the mix in contact with the air has increased, as a result, water gets evaporated much faster as compared to close graded mix. So due to the increased exposed area to the atmosphere and constant conditioning at 40°C in forced oven, the emulsion got broken much earlier, became stiff and non workable. Thus for Open Graded mix, it is not suitable to condition the loose mix at 40°C for breaking of emulsion unlike that of close graded mix. Primarily, this will also not simulate the field condition in case of plant mix.

In another trial of the laboratory study, the loose mix were spread in a tray and kept in ambient temperature. The mix was checked at regular interval of 15 minutes to determine the intensity of breaking. After 20 minutes, the upper layer of mix turned black. The mix was then agitated and kept idle for further observation. It was observed visually that at 50 minutes from mixing, there were almost 70%-80% breaking of bitumen emulsion in the mix (conversion from brown to black in color) (Photo 4.10). On the other hand if we keep the mix in open sunlight for 10 minutes to 15 minutes, then after 40 minutes the mix turned completely into black, which depicts the breaking of emulsion. Therefore, according to laboratory study, it is suggested to keep the mix in sunlight for 10-15 minutes for fast breaking and then compacted at 70%-80% breaking of emulsion.

a) Fresh mix　　b) 70%-80% Breaking　　C) Complete Breaking

Photo 4.10. Breaking Phenomenon of Emulsion

4.4.2.2 Evaluation of optimum compaction

To determine the compactive effort required to get the maximum density of the highly porous cold mix skeleton, the mix was compacted with Marshall Compacter from 20 blows to 50 blows at an increment of 10 blows. It was observed that density becomes constant after 40 blows, as there is very marginal difference between the density at 40 blows and 50 blows (Fig. 4.7). Moreover, aggregate starts to crush at higher blows, i.e. at 50 blows. Therefore for maximum compaction and to avoid crushing of aggregates, the samples of emulsion mix highly porous skeleton were compacted at 40 blows.

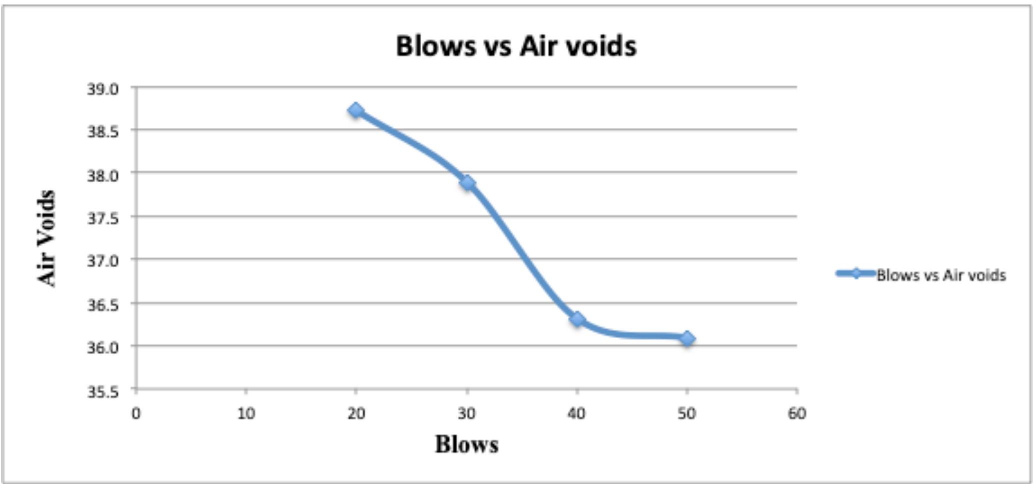


Fig. 4.7. Compaction curve for cold mix CGBM GR-I gradation.

4.5 VOLUMETRIC ANALYSIS OF POROUS BITUMINOUS MIX

The volumetric analysis of each of the compacted sample was based on bulk specific gravity of compacted mix (G_{mb}), loose mixes density or Theoretical maximum specific gravity (G_{mm}) and bulk specific gravity of aggregates (G_{sb}). Six samples of standard mix i.e. CGBM GR-I with VG 30 and OGFC with VG 30 each were prepared three compacted at defined number of blows and three samples to determine the loose mix density. The formula used for calculation of Air Voids (V_a) in the compacted bituminous mix is given in Equation 4.1.

$$V_a = \frac{G_{mm} - G_{mb}}{G_{mm}} X\ 100 \tag{4.1}$$

Where,

V_a Air voids in compacted mix (percentage)

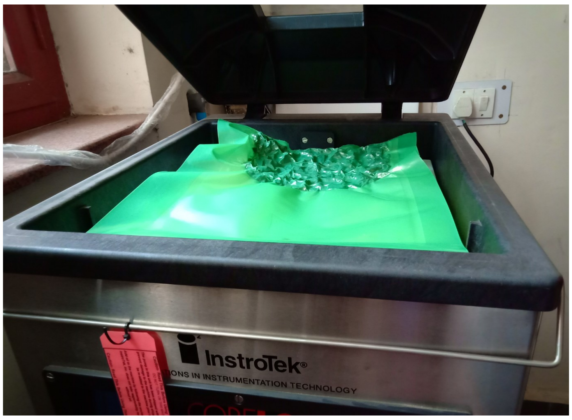
G_{mm} Theoretical maximum specific gravity of mix

G_{mb} Bulk specific gravity of compacted sample

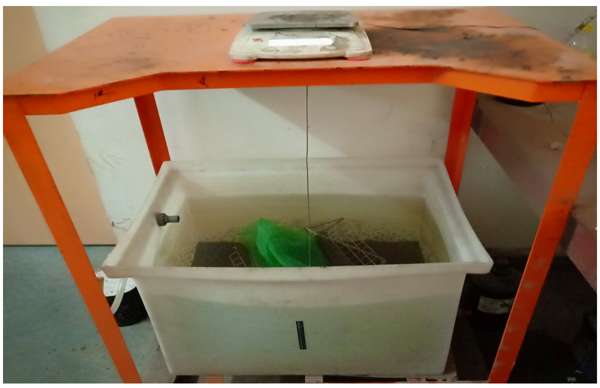
Theoretical maximum specific gravity of loose bituminous mix (G_{mm}) was measured as per ASTM D6752. The CoreLok apparatus was used for the measurement of theoretical maximum specific gravity of the loose bituminous mix is shown in Photo 4.10 (a). Program 1 in the CoreLok apparatus is generally used to determine G_{mm} as per ASTM D6752. First the weight of the empty bag needs to be measured in grams (W1). After that, the loose mix was filled inside the plastic bags and weighed (W2). The loose sample was then kept inside the CoreLok apparatus in the vacuum chamber, which will expel out all the air in the loose mix and will seal the bag. After this process got completed, the sealed portion of the bag is cut keeping it inside the water tank and then weighed in water as shown in Photo 4.11(c).

Since the specimens have high air void content, it is suggested that their bulk density was determined as per ASTM D3203, which mentions the estimation of the specimen bulk volume from dimensions. The optimum values of binder content, degree of compaction and the air voids obtained for the two aggregate gradations considered for preparation of high voids bituminous mix is shown all together in Table 4.6.

a) CoreLok Apparatus



b) Sealing and application of Vacuum



c) Weight of the Loose mix in water.

Photo 4.11. Measurement of Gmm through Vacuum sealing Method (CoreLok Apparatus)

Table 4.6

Optimum Values for Standard Gradations

Gradation Type	Optimum Binder Content (%)	Optimum Marshall Blows	Gmm	Gmb	Air Voids (%)
OGFC	4.7	60	2.63	1.84	30
CGBM GR-I	2.43	40	2.73	1.8	34

4.5.1 Effect of Binder grade and Binder content on volumetric properties

To interpret the effect of varying binder grade and binder content with respect to the standard mix of CGBM GR-I gradation with VG 30, compacted highly High Void Bituminous mix were prepared with variables like different grades of binder such as VG 30, VG 40, PMB 40 and emulsion (Fig. 4.9). Effect of varying binder grade on air voids is also clear from Fig. 4.8. It can be seen that highly porous mix with Polymer modified bitumen (PMB 40) has air void content lesser than mix prepared with viscosity grade bitumen, the reason might be PMB 40 creates a thicker binder film around the aggregates. Mix prepared with emulsion has got the higher air void content of about 36%, this might be because emulsion mix for open graded aggregates creates a reduced or thin binder film thickness around the aggregates and moreover the binder coating might not be uniform and less efficient around the aggregates for open graded emulsion mixes.

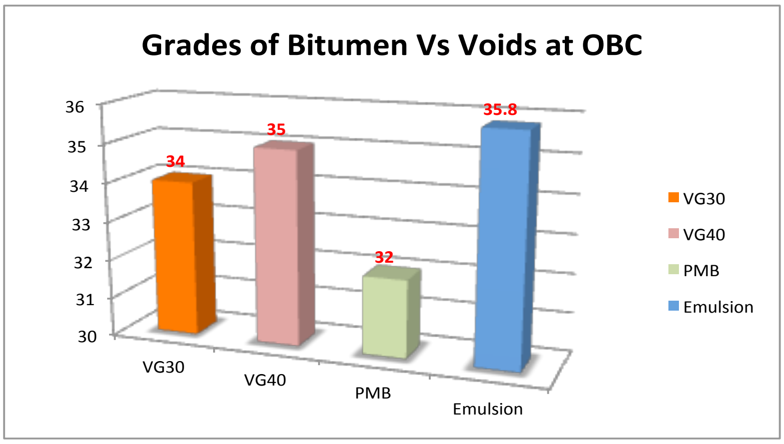


Fig.4.8. Effect of grades of Binder on Air Voids

At a binder content higher than the optimum binder content, high void bituminous mix were prepared with VG 30 and PMB 40 to understand the effect of increasing binder content on volumetric properties (Fig. 4.9). It is inferred that with the increase in binder content, there was a decrease in air voids. With the increase in binder content, theoretical maximum specific gravity decreases because of increase in thickness of the binder film and increase in the volume of voids filled with bitumen, which will reduce the air voids.

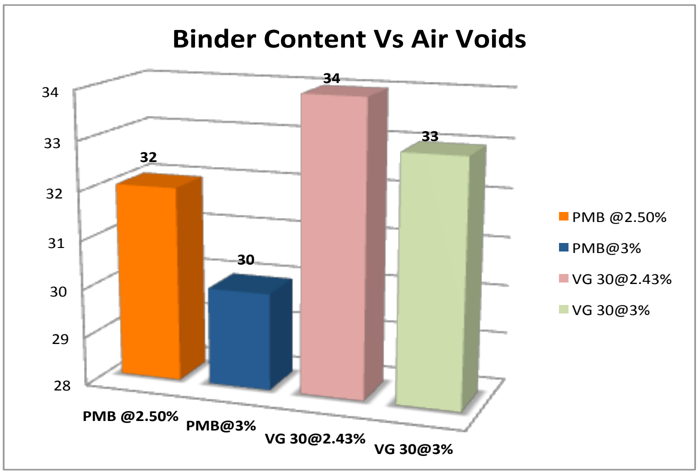


Fig. 4.9. Effect of Binder content on Air void

4.6 AGGREGATE PACKING CHARACTERISTICS

In case of open gradations, where the void content needs to be high as per requirement and suitability, there should be adequate stone-on-stone contact to maintain the proper interconnectivity between voids. To ensure this VCA of the compacted mix should be less than the VCA dry rodded (equation 4.2) of the coarse aggregates.

$$VCA\ dry\ rodded = \frac{G_c\Upsilon_w - \Upsilon_a}{G\Upsilon_w} \tag{4.2}$$

Where,

G_c= Combined bulk specific gravity of the aggregates

Υ_w= density of water in kg/m^3

Υ= bulk density of coarse aggregates in dry-rodded condition

VCA mix can be calculated from the following euation;

$$VCA_{mix} = 100 - \left(\frac{G_{mb}}{G_c} P_c\right) \tag{4.3}$$

Where,

P_c = Percentage coarse aggregates in mix

G_c= Combined bulk specific gravity of the aggregates.

G_{mb}= Bulk density of compacted mix

Since in our study, primary requirement is full depth grouting, so the void content should be in between 25%-35% to maintain the interconnectivity between the voids. The void ratio was calculated by determining the bulk density of the compacted mix as per ASTM D3203 and theoretical maximum density was calculated using ASTM D6752. Equation 4.4 was referred to calculate the void content of standard mix; the values of void content of compacted mix, VCA of the compacted mix and VCA of aggregates by dry rodded technique of the standard mixes and variables with CGBM GR-I gradation are shown in Table 4.7.

$$V_a = \left(1 - \frac{G_{mb}}{G_{mm}}\right) x\ 100 \tag{4.4}$$

Table 4.7

Air Voids of the compacted mix

Mix Description	Gmm	Gmb	Air Voids	VCA $_{dry\ rodded}$	VCA_{mix}
CGBM GR-I-VG30	2.73	1.8	34%	42.65	37.7
CGBM GR-I-VG40	2.75	1.78	35%		38.4
CGBM GR-I-PMB 40	2.62	1.78	32%		38
CGBM GR-I-Emulsion	2.62	1.78	35.8%		41.36

From Table 4.7, it can be inferred that, above combination and grading is suitable to maintain proper inter-connectivity between voids as VCA $_{mix}$ is less than VCA $_{dry\ rodded}$ and void content is in between 25%-35% for full depth penetration of grout. In the case of Emulsion mix, interestingly there is very less difference between VCA_{mix} and $VCA_{dryrodded}$, which may depicts the less effective and non uniform coating of binder over the aggregates and formation of thin binder film incase of emulsion mix highly High Void Bituminous mix.

4.7 OPTIMIZATION OF WATER CONTENT FOR GROUT

From the discussions in the previous clauses, it is very clear that flow of grout material increases with increase in water content with simultaneous decrease in its compressive strength. So, in order to decide the quantum of minimum water content at optimum grout compressive strength, which would be sufficient to achieve full depth of grout penetration in each case, various trials were made at varying water contents. The degree of grout penetration into the voids of porous bituminous mix structure is not only linked to initial air voids but it is also affected by the morphological characteristics of air voids, void structure and its pore size, which significantly influence the interconnectivity of voids in bituminous skeleton (Vavrik et al., 2001). Some of the voids are open, some are partially open and some are closed. The degree of the grout penetration depends upon the quantity of open, and partial open voids along with their size. It is difficult for the grout material to penetrate in the closely spaced voids. To improve this, the closed voids should be reduced as far as possible. The selection criterion for optimum water content as considered in this study was full depth of grout penetration into the voids of bituminous specimens. Grouting of specimens for each mix was started with 16% of water with subsequent increment of 2%. In case of CGBM GR-I gradation; different extent of grout penetration achieved with gradual increase in mixing water content is shown in Photo 4.12. Grout slurry made with 16 % water content by weight of grout could penetrate only to a shallow depth and most of the grout was retained on the surface of the bituminous specimen itself, as shown in Photo 4.12(a). At 18 % water content, grout penetration was observed to be up to half depth of the specimen as shown in Photo 4.12(b). At 20 % water content, the grout slurry could be seen to penetrate to full depth of the bituminous sample and voids in the sample appeared to be filled completely with grout, as shown in Photo 4-10(c). Similarly optimum water content in grout for OGFC aggregate gradations was measured. For OGFC full depth grout penetration were observed at very high water content in grout material of about 24%, For OGFC, grouting was started at 20% water content which shows limited depth of grouting, 22% water content shows partial depth of grouting because of lower amount of open interconnected air voids and higher number of closed air voids in these bituminous mix gradations and 24% shows full depth of grouting, which are shown in Photo 4.12(c). Table 4.8 gives the optimum water contents for two aggregate gradations, which is sufficient to achieve full depth of grout penetration in that particular bituminous mix gradation.

(a) Shallow depth of grouting

(b) Partial depth Grouting

c) Full depth of grouting

Photo 4.12. Different extent of grout penetration for CGBM GR-I

It was observed that even at higher binder content (increase by 0.5%), there was a full depth penetration of grout slurry but for polymer modified bitumen (PMB 40), at some void spaces interconnectivity was not there and as a result grout could not be able to penetrate through those spots.

Photo 4.13. Full depth grouting at 20% W/c for CGBM GR-I.

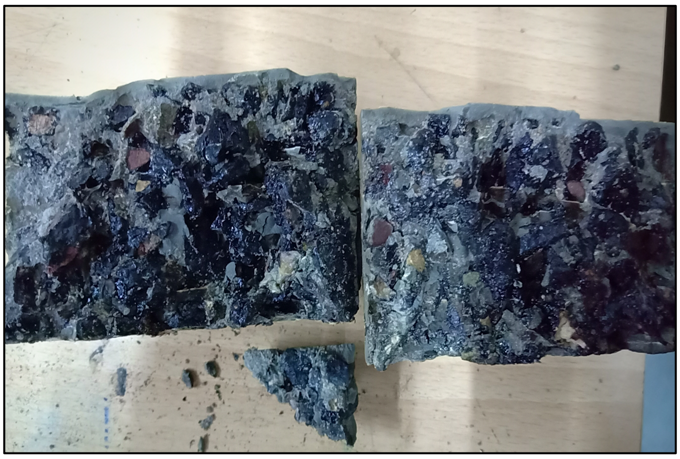


Photo 4.14. Full depth grouting at 24% Water content with less interconnectivity for OGFC

From the Photo 4.13 and 4.14 it is clearly seen that for CGBM GR-I, there is full depth of grouting and there is full inter-connectivity of voids in between as a result there is proper flow of grouts. Moreover, due to requisite binder coating requirement of aggregates and less fines, the voids are not getting clogged.

Whereas, in OGFC, though there is full depth grouting, but inter-connectivity between voids is not there because of higher binder content and a greater number of fines almost 6-7 per cent, as compared to 0.75-1 per cent in CGBM GR-I. Therefore, out of these two, CGBM GR-I is chosen to be suitable gradation for CGBM. Although as per literature other gratings such as OGPC, SD 19, etc. can incorporate voids in between 25%-35% may be suitable for CGBM. Photo 4.15 and Photo 4.16 shows the inter connectivity between voids for emulsion mix grouted composite course and PMB 40 mix grouted composite course. In case of emulsion mix proper interconnectivity between voids are there and there is a full depth flow of grout material, though at some points there are some blocked pores because of binder.

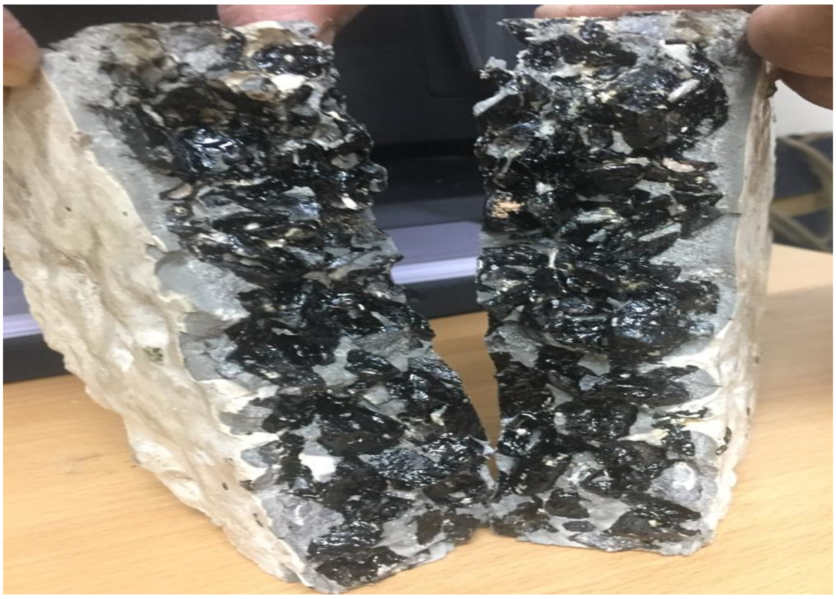


Photo 4.15. Extent of grout penetration in Emulsion mix grouted specimens

Photo 4.16. Extent of Grout Penetration in PMB 40 mix grouted specimens.

CHAPTER 5

5. EXPERIMENTAL INVESTIGATION OF CGBM SAMPLES

5.1 PREPARATION OF GROUTED SPECIMENS

As per the spectrum of the research study, for comparative analysis CGBM samples were prepared by varying the binder grades such (VG 40, PMB 40, emulsion) and binder content for PMB 40 and VG 30, only to check the adequacy and compared with standard CGBM sample of VG 30 in-terms of mechanical properties like Resilient modulus, Indirect Tensile Strength, Marshall Stability and compression test samples. In performance criteria samples were prepared to test for Moisture Induced susceptibility test (MIST), Cantabro loss and Dynamic fracture energy. For Resilient modulus, Marshall stability, ITS, MIST and cantabro loss standard marshal samples of 63 mm height and 101mm diameter were prepared in the laboratory. Compression test samples of 100 mm diameter and 180 mm height were prepared using gyratory compactor and grouted in the laboratory in 100mm x 200 mm mould. As per Texas-248f for dynamic fracture energy special kind of samples is required of size 38mm x 76 mm, for this modified marshal samples (6 inch) were prepared. For the application of grout slurry in the porous bituminous skeleton, dry grout material at requisite water content was mixed for about 20 minutes (Refer clause 2.3) to get a uniform mix of slurry. Then the slurry was allowed to flow into the confined bituminous skeleton (here in our study split marshal mould was used) under the action of gravity (Photo 5.1) and excess grout on the surface needs to squished out to expose the aggregate rough surface for proper friction so as to simulate the field conditions.

Photo. 5.1: Process of Grouting in Porous Bituminous Skeleton

After the process of grouting, the specimen is then allowed to set for 24 hours inside the mould. After 24 hours, the specimen was set ready to be opened (Photo 5.2) and then its dimensions and weight was taken to determine the volumetric properties. For curing purpose all the grouted specimens were kept in curing tank under covered by weight gunny bags and then cured for 7 days before testing.

Photo: 5.2. CGBM samples prepared in Laboratory.

5.2 MECHANICAL CHARACTERIZATION OF CGBM

Mechanical properties of the mix generally comprises of Resilient Modulus, Indirect Tensile Strength, Marshall Stability and Compression Resistance of the mix. To determine these properties for cement grouted bituminous mix three samples were being prepared and average value was being considered**.**

5.2.1 Indirect Tensile Strength Test

Indirect tensile Strength simulates the tensile properties of the mix and gives us an idea about the tensile strength of the mix. A higher tensile strength corresponds to a better cracking resistance. Cracking resistance will simulate the fracture strength and toughness of the mix.

The test was conducted as per ASTM D 6931 test method on Marshall samples of 100 mm diameter and 63 mm height. ITS test method consists of applying a load along the cylindrical specimen's diametrical axis at constant deformation rate of 51 mm/minute and determining the total vertical load at failure of the specimen at different test temperatures. Failure point is defined, as the point after which there is no further increase in load and full fracture of the sample occurs along the diametrical axis. The maximum load P, taken by the specimen was used to calculate the indirect tensile strength as per Equation 5.1.

$$ITS = \frac{2P}{\pi D t} \qquad (5.1)$$

where

ITS	Indirect tensile strength (MPa)
P	load at failure (N)
t	Height/thickness of specimen (mm)
D	Diameter of specimen (mm)

Three samples for each combination varying the binder grade and binder content were casted and then conditioned at test temperatures of 25°C, 35°C and 45°C respectively. The jig of loading in ITS test is shown in Photo 5.3. The cracked samples after test can be visually analyzed for any bulging or interconnectivity of the voids to understand the full depth penetration of grout, as shown in Photo 5.5. The complete test setup for the indirect tensile strength test is shown in Photo 5.4.

Photo 5.4. Test Setup of ITS

Photo 5.3 Loading Jig for ITS

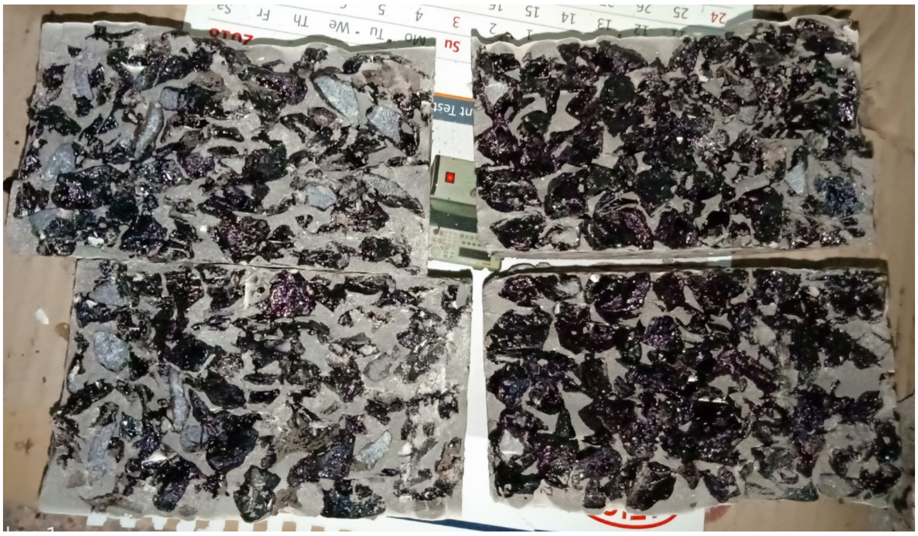


Photo 5.5. Cracked sample after ITS test

5.2.2 Resilient Modulus (M_r)

When the pavement is subjected to wheel load, transverse stresses develop and as a result strain develops, the strains developed are either permanent or elastic. Resilient modulus of the mix is basically the response of the mix to cyclic loads applied in the pavement due to the wheel loads, which estimates the elastic modulus or recoverable strain of the mix. It is a very important parameter, which is used for the design of the pavement.

Since CGBM is a very concept in India, there is a very little less idea about the resilient modulus property of CGBM and as per the nature of the study since CGBM is going to be used as a surface layer, it is necessary to assign appropriate resilient modulus to the CGBM mix for analysis of the pavement design incorporating surface course or overlay course as CGBM. The resilient moduli for CGBM prepared using different combinations by varying the binder grade and binder content and Bituminous Concrete were measured using the indirect tensile strength modulus method as per ASTM D4123 at different temperatures of 25°C, 35°C and 45°C. The test setup for the testing of resilient modulus of elasticity value under indirect tensile loading condition is shown in Photo 5.6. CGBM samples were cured for 7 days first for determination of the resilient modulus. Three Marshall Samples of each combination were prepared and then cured for 7 days. After 7 days curing before testing, the samples were conditioned as per ASTM D4123 procedure for a period of 4 hours at different test temperatures of 25°C, 35°C and 45°C. The test was conducted at maximum load of 10 percent of ITS to keep the mix within the elastic range.

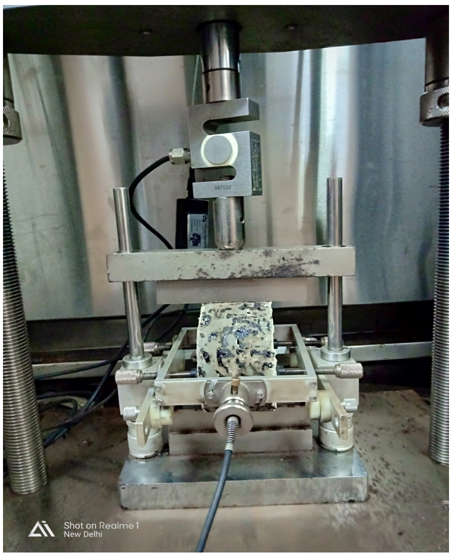


Photo 5.6. Jig arrangement for Resilient Modulus test for CGBM.

5.2.3 Marshall Stability

Marshall Stability test depicts the idea of shear resistance of the mix. This test was performed in accordance to ASTM D1559. Samples of 100 mm diameter and 63 mm height were prepared for various combinations varying the binder content and binder grade. The grouted samples were cured for 7 days and then conditioned at 60^{o}c Water bath for 30 minutes before testing. The test set up is shown in Photo 6.5 below. Because of the composite nature of the CGBM mix, it cannot be tested in normal marshal test apparatus for bituminous mix, thus a special assembly was prepared in CRRI laboratory for stability test in UTM machine as shown in Photo 5.7.

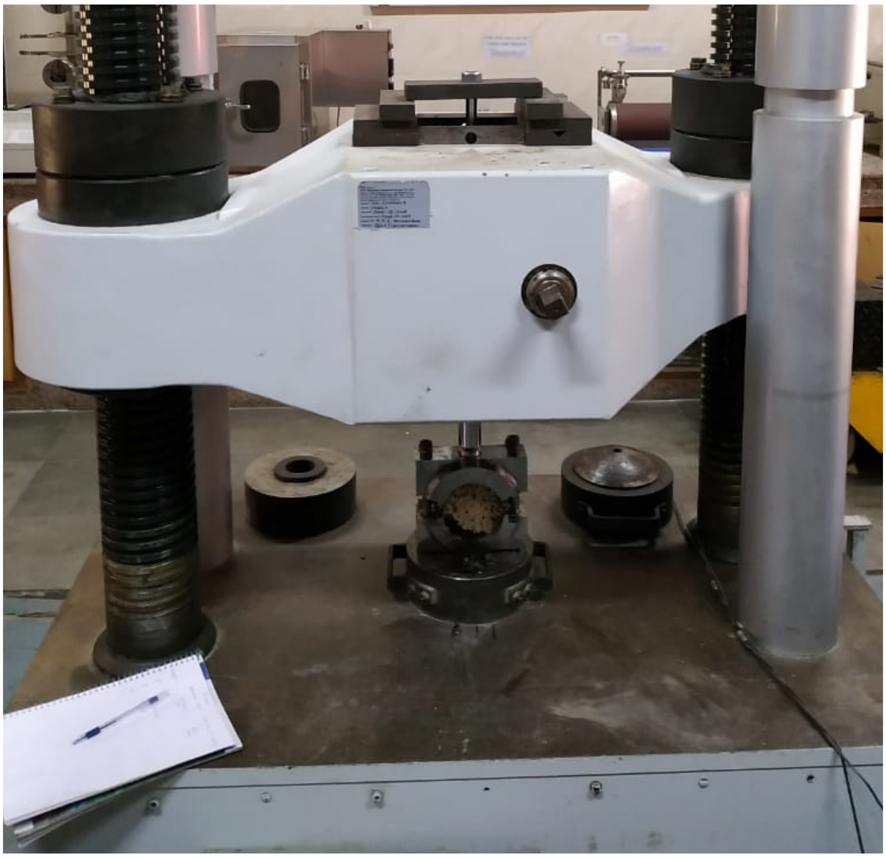


Photo 5.7. Marshall stability test for CGBM

5.2.4 Compressive Strength Test of CGBM

Compressive strength of the CGBM generally gives us a view of the strength of the CGBM mix under direct compression or axially loaded compressive force so that the mix does not get crushed. Compressive strength test were carried out as per ASTM C39. In this test 100 mm diameter and 180 mm height samples of different combinations considered in the study were prepared in Gyratory compactor as discussed in clause 4.2. The samples were grouted in 100 mm diameter and 200 mm height moulds and then cured for 7 days before testing in the UTM at test temperature of 25^{o}C. Compressive strength test are shown in Photo 5.8. The formula to calculate the compressive strength is shown in equation 5.2.

$$f_{cm} = \frac{4000\, P_{max}}{\pi D^2} \tag{5.2}$$

Where

f_{cm} Compressive strength, MPa

P_{max} Maximum load, kN

D Average measured diameter, mm

Photo 5.8. Compressive strength test

5.3 PERFORMANCE CHARACTERIZATION OF CGBM

Along with the mechanical properties, it is very much necessary to study the performance parameters of CGBM, which will simulate the actual field performance. Since CGBM is altogether a new concept in Indian road infrastructure scenario, therefore certain performance parameters like fatigue, rutting, moisture susceptibility and abrasion needs to be performed. In this study for Dynamic Fracture energy to simulate the fatigue characteristics of CGBM, Cantabro loss for abrasion and Moisture Induces susceptibility test to simulate raveling/stripping due to moisture ingress has been performed and are described below.

5.3.1 Moisture Induces Stress Test (MIST)

Moisture damage resistance was evaluated in terms of Tensile Strength Ratio (TSR) for CGBM and BC mixes. Moisture induced damage is basically related to the loss in strength due to reduced adhesion between aggregate and bitumen along with reduction in cohesive property of binder and the filler material, when there is a water stagnation specially in case of urban roads, continuous movement of wheels lead to ingress of water into the pavement which leads to development of pore water pressure due to this pore pressure the bond between the bitumen and the aggregates become weak and leads to stripping. In order to simulate such kind of field conditions; Moisture Induced Sensitivity Test (MIST), as per ASTM D7870 was incorporated in which an accelerated moisture conditioning method with cyclic loading is used. The test parameters were set as: pressure at 40 psi, temperature at 60°C and number of cycles to be 3500. The MIST equipment and Marshall Samples being kept inside the MIST chamber are collectively shown in Photo 5.9. After subjecting the samples to cycles of moisture induced damaging effects, the specimen was kept in water bath at 25°C for 2 hours and then tested for their retained indirect tensile strength. The Tensile Strength Ratio (TSR) was evaluated using Equation 5.3. The obtained results for the TSR values are given in Table 8-3. CGBM mixes were found to have significantly far better resistance to moisture damage as compared to conventional bituminous concrete mix.

$$TSR\ (\%) = \frac{T_2}{T_1} \times 100$$

(5.3)

where,

T_2 ITS value of MIST conditioned specimen

T_1 ITS value of unconditioned specimen

Photo 5.9. Moisture Induced Susceptibility Test

5.3.2 Dynamic Fracture Energy

Dynamic fracture energy by Texas Overlay Tester simulates the behavior of mixes to fatigue or reflective cracking. In this test Overlay tester was basically used to identify the fracture properties of CGBM and BC. Critical Fracture Energy and Crack resistance Index are the performance parameters that depict the resistance of these mixes to cracking. Generally, fatigue crack occurs through micro cracks that accumulates and penetrates through the asphalt surface, which leads to fatigue cracking. Generally, fatigue cracking is based on two-stage process, Crack Initiation and Crack Propagation. In this Overlay test, which applies repeated dynamic direct tension to the specimen on strain-controlled mode, which automatically records and measures the load, displacement and temperature in every 0.1 seconds. The tester consists of two blocks: one is fixed and other is sliding. On applying the tension, the sliding plate applies tension to the specimen in a cyclic triangular waveform to a maximum displacement of 0.6 mm (predefined) and this can also me reduced further as per our requirement. The sliding plate reaches the maximum displacement and then returns to its initial position (1 Cycle) in 10 seconds. For 0.06 mm displacement, the peak load carried by the specimen is measured and the test is performed till 93% of load reduction or 900 cycles whichever is earlier. The deciding parameters to simulate the cracking phenomenon of the mixes are Critical Fracture Energy and Crack Resistance Index. Critical Fracture energy is basically the energy required to develop the initial crack on the bottom of the specimen at the first loading cycle. This property symbolizes the fracture characteristics of the specimen during the initial cracking phase. Crack Resistance Index is basically the percentage reduction in Load, which is required to propagate through the sample under repeated loading conditions of the overlay tester.

In our study, 150 mm diameter modified Marshal Samples of different combinations such as sample with Viscosity grading binder; samples with Polymer modified binder and with emulsion were prepared and then trimmed to the specimen required for overlay test of height 38 mm and width 76 mm (Photo 5.10). The detailed procedure is shown in Appendix A. Photo 5.11 shows the overlay test setup.

Photo 5.10. Overlay Test Specimens

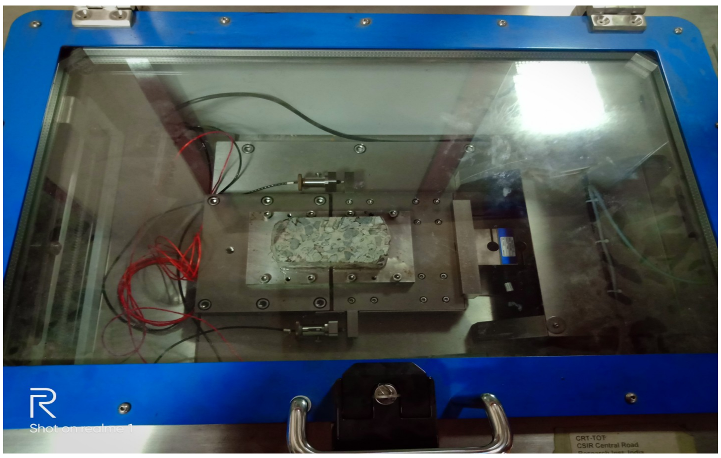


Photo 5.11 Overlay Tester

5.3.3 Cantabro Loss

Durability is a very important phenomenon in the performance analysis of CGBM. To check the durability of CGBM samples prepared with VG 30, VG 40, PMB 40 and Emulsion mix, Cantabro Test was conducted using the Los Angeles abrasion machine without the application of charge (steel balls) as per ASTM D7064. Standard Marshall Samples were conditioned for 4 hours at 25°C prior to abrasion test. After the test loss in weight of the samples were measured. The loss in weight of the samples expressed, as percentage of the initial weight is the simulating property for the abrasion loss caused due to wearing and tearing.

CHAPTER 6

6. RESULTS AND DISCUSSIONS

6.1 BULK DESNITY

Bulk density of the CGBM samples were measured by measuring the dimensions of the grouted samples. After preparing samples with different combinations of binder and binder content, they are cured for 7 days before measuring the dimensions for bulk density as shown in Table 6.1.

Table 6.1

Effect of Binder Grade and Binder Content on Bulk density of CGBM

Sample	Avg. Dia.	Avg. Height	Weight	Density	Average Density
	mm	mm	g	g/cc	g/cc
CGBM with PMB 40 at OBC	102.9	61	1244.6	2.45	2.47
	103	60	1237.8	2.48	
	102.74	60	1236	2.48	
CGBM with PMB 40 at 3% BC	102.8	63	1268	2.43	2.41
	102.8	63	1256	2.40	
	102.8	64.6	1283.5	2.40	
CGBM with VG 40	102.7	60	1232	2.48	2.49
	102.8	60.8	1251	2.48	
	103.3	59	1243	2.51	
CGBM with VG 30 at OBC	103	58.8	1233.2	2.52	2.49
	102.8	59.5	1215	2.46	
	103	61	1265	2.49	
CGBM with VG 30 at 3% BC	103.2	59.1	1208.4	2.45	2.45
	102.7	61.0	1236.5	2.45	
	102.7	59.1	1196.8	2.44	
CGBM with Emulsion	102.9	61.4	1305	2.55	2.53
	102.4	60.6	1283	2.53	
	103.6	62.8	1332	2.52	

It can be inferred from Table 6.1 that, variation of binder grade effects the bulk density of CGBM. The densities of the CGBM mix with PMB 40, VG 30 and VG 40 are almost similar. Though it can be said that, with the increase in binder film thickness, the volume of grout required to fill the voids would be less incase of modified binders and as a result the density would be less as seen for CGBM with PMB 40 mix, but marginal difference is there. Whereas, in Viscosity grade binders due to the lesser binder film thickness, the volume of grout filled will be more and thus the density will be slightly high. Moreover, this is further evident from the fact that, in case of Emulsion mix CGBM, due to the non-uniformity in binder coating and thin binder film thickness, the volume of grout required to fill the voids would be more and thus the density is high, (2.53 g/cc) which is higher than other grade of binders.

It is also evident from Table 6.1 that, with the increase in binder content also, the density of the CGBM mix decreases, this is because, with the increase in binder content, more amount of voids get filled with binder and thus volume of voids available to fill from grout remain less, thereby the volume of grout penetrating into the CGBM mix gets reduced.

6.2 RESILIENT MODULUS (M_R)

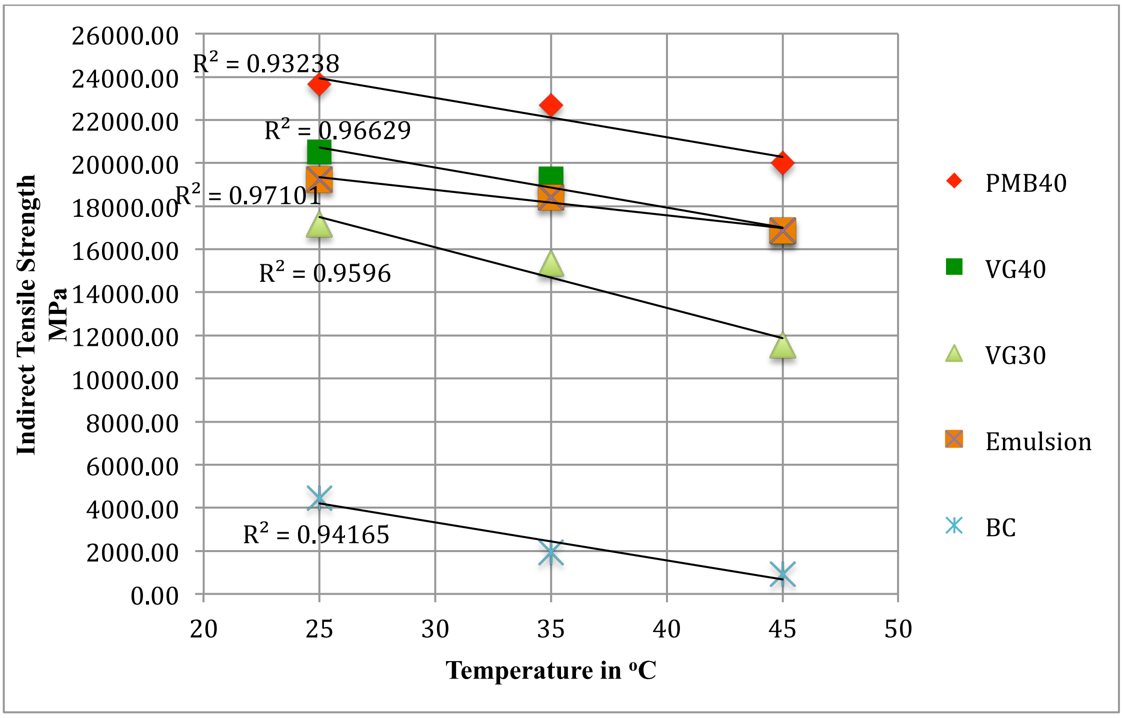


Figure 6.1. Variation of Resilient Modulus with temperature for different combinations of CGBM

Table 6.2

Resilient Modulus at different test temperatures for all variation of mix.

	Test Temperature (°C)	25 °C	35 °C	45 °C
Resilient Modulus (MPa)	PMB 40	23653	22679	20003
	VG40	20521	19264	16804
	VG30	17162	15350	11539
	Emulsion	19227	18400	16867
	BC	4456	1930	926

Figure 6.1 shows the resilient modulus values of different combinations of CGBM mix at different temperatures. CGBM mixes have higher resilient modulus values than BC samples because of the stiffness provided by the grout material. For both CGBM mixes and BC mix, M_r Value reduces with the increase in temperature which depicts the flexibility of the CGBM mix and thus we can say that CGBM mix can also be referred as semi-flexible mix. In case of BC there is almost 80 % reduction in the stiffness from 25°C to 45°C whereas, in case of standard VG30 CGBM mix, the reduction in the stiffness value is only 33% from 25°C to 45°C.

Varying the binder grade there is notable change in the resilient modulus values of CGBM mix, it is quite clear that resilient modulus of PMB 40 is high (Table 6.2) as compared to other grade of binders. Moreover, at higher test temperature also, PMB 40-CGBM mix showed higher value of resilient modulus as compared to VG 30, VG 40 and Emulsion. This shows the better performance and less temperature effect susceptibility of PMB40-CGBM mix at higher test temperature. Because of the more stiffness of the CGBM mix, it can be said that, the strain develops under cyclic load will be less or we can say that the load required to generate strain at failure will be much more as compared to BC mix.

From the Figure 6.1, it is also very clear that, Emulsion mix CGBM samples has higher value of resilient modulus at test temperature of 45°C as compared to VG40 and VG 30 mix, this may be due to the thin binder film and non-uniform binder coating, the grout rigidity dominates here and thus, there is less reduction in modulus value (which is a binder phenomenon) at higher temperature.

Further study was also carried out to understand the effect of Binder content on CGBM mix as shown in Figure 6.2.

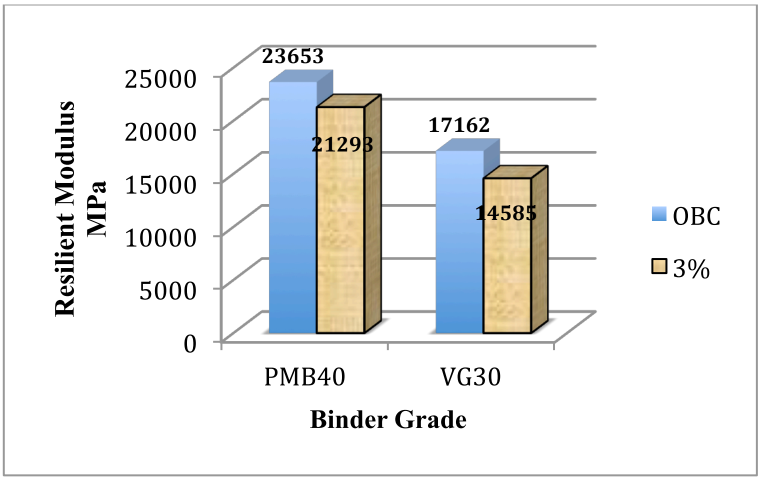


Figure 6.2. Effect of Binder content on Resilient Modulus of CGBM mix

With the increase in binder content, the resilient modulus or stiffness decreases. As discussed previously, more binder content will fill a greater volume of voids and thus effective void volume for grout penetration will reduce and subsequently, the stiffness of the mix gets reduced.

From the above test results, it is very clear that, reducing the binder content and incorporation of harder grade binder will induce increased stiffness of the CGBM mix.

6.3 INDIRECT TENSILE STRENGTH

The results of Indirect Tensile Strength test for CGBM mix with VG30, VG40, PMB40 and Emulsion are shown in the Figure 8.3. ITS result of Standard CGBM mix with VG30 is also shown in the figure for comparison.

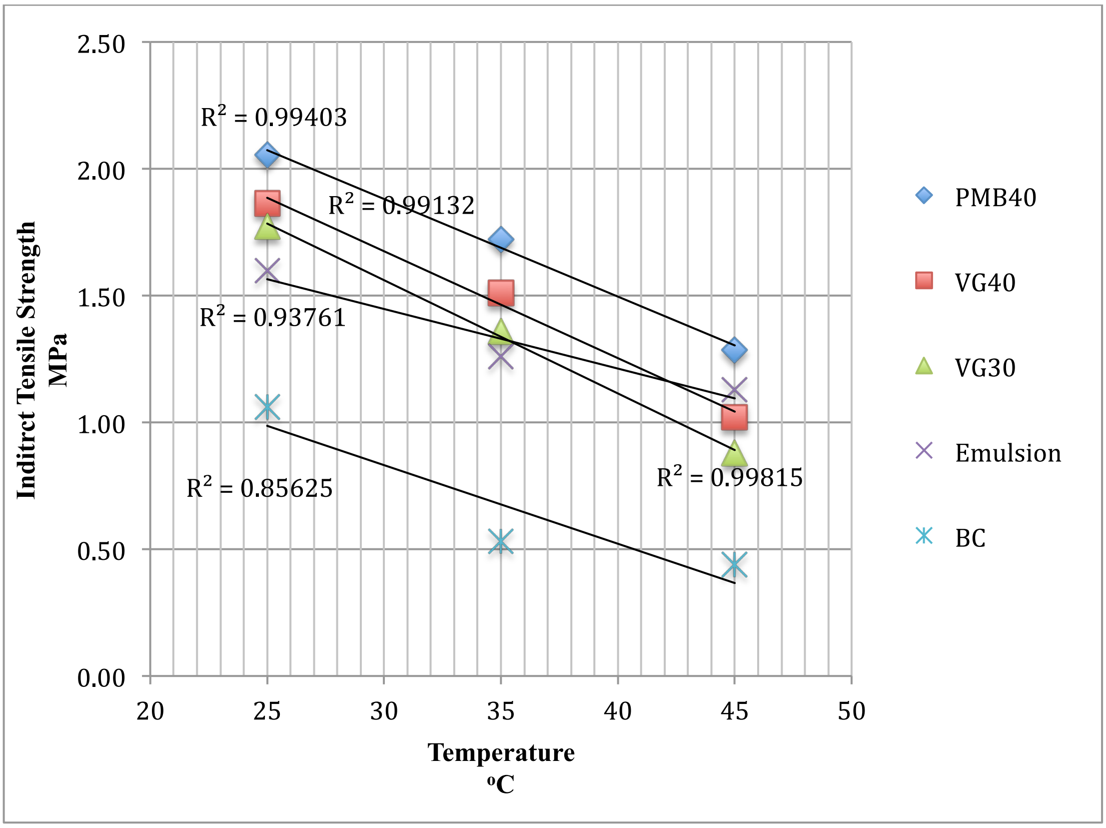


Figure 6.3. Variation of ITS with temperature for different combinations of CGBM

Table 6.2

ITS values at different test temperatures for various Binder Grades.

	Test Temperature (ºC)	25 ºC	35 ºC	45 ºC
Indirect Tensile Strength (MPa)	CGBM-PMB 40	2.06	1.72	1.29
	CGBM-VG40	1.86	1.51	1.02
	VG30	1.77	1.36	0.88
	CGBM-Emulsion	1.60	1.26	1.13
	BC	1.06	0.53	0.44

It is quite evident from Figure 6.3; indirect tensile strength of CGBM mix with PMB 40 is higher as compared to VG30, VG40 and Emulsion mix. Because of the elastomeric property of the modified binder, it influences the indirect tensile strength of the CGBM, thus PMB40 CGBM mix shows good resistance to fracture as compared to viscosity grading binder and emulsion. Moreover, performance of PMB40 CGBM mix is also seem to be good at higher test temperature which shows the less temperature susceptibility. Reduction in ITS value at 45°C is 37% for PMB as compared to 50% for VG30. Thus, the fracture strength of CGBM with PMB40 is higher than that of other grades of binder used in the study.

Emulsion mix CGBM performs better as compared to viscosity grading bitumen at higher test temperature of 45°C (Table 6.2). The reason is same as discussed in clause 8.2. In case of Emulsion mix CGBM, grout material is dominating as the volume of the grout is high which brings the rigidity in the mix, thus has a less temperature susceptibility as compared to VG30 and VG40 CGBM mix.

From Table 6.2, it can be inferred that CGBM mixes has higher ITS value as compared to BC mix. The CGBM mix altogether has a very less temperature susceptibility than BC, which is quite clear from Figure 6.3.

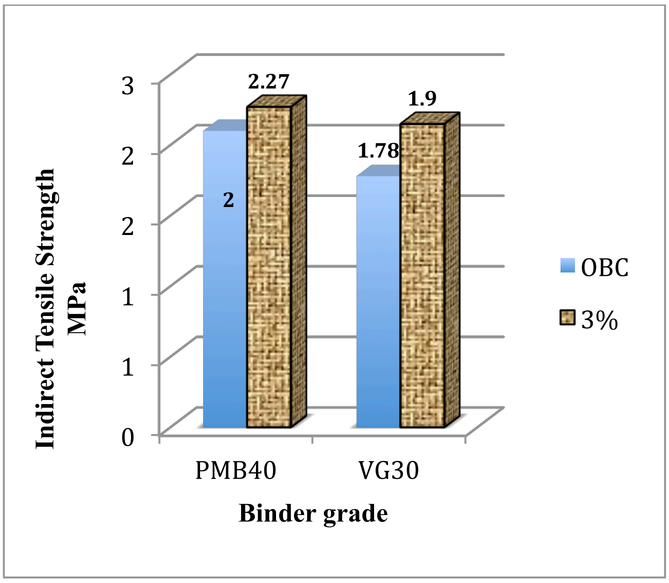


Figure 6.4. Effect of Binder Content on ITS of CGBM

With the increase in binder content, the flexibility of the CGBM increases, at 3% PMB, the ITS value increases by 10% as with 2.5% OBC for PMB40. It has to be noted that further increase in the binder content may affect the interconnectivity of the voids which will perhaps reduce the stiffness as grout material may not penetrate to full depth and also reduce the ITS. In our study at 3% BC for PMB40 and VG30, interconnectivity between the voids the voids were there and grout has penetrated to full depth.

6.4 MARSHAL STABILITY

Marshall Stability test were performed with CGBM samples prepared with various grades of binder such as VG30, VG40, PMB40 and Emulsion and standard mix of CGBM with VG 30 was compared with the stability values of BC. The test results are shown in the Table 6.3.

Table 6.3

Effect of Binder Grade on Marshal Stability

Sample No.		Stability (kN)	Correction Ratio	Corrected Results (kN)	Average Values (kN)
CGBM-PMB 40	1	84.92	1.04	88.32	98.07
	2	96.68	1.1	106.35	
	3	90.5	1.1	99.55	
CGBM-VG30	1	66	1.09	71.94	77.67
	2	78.4	1.06	83.10	
	3	69	1.13	77.97	
CGBM-VG40	1	72	1.14	82.08	84.74
	2	73.56	1.09	80.18	
	3	85.93	1.07	91.95	
CGBM-Emulsion	1	85	1.04	88.40	98.84
	2	102	1.06	108.12	
	3	99	1.01	99.99	
BC	1	12.75	1.02	13	13.76
	2	13.96	1.04	14.52	
	3	13.23	1.04	13.76	

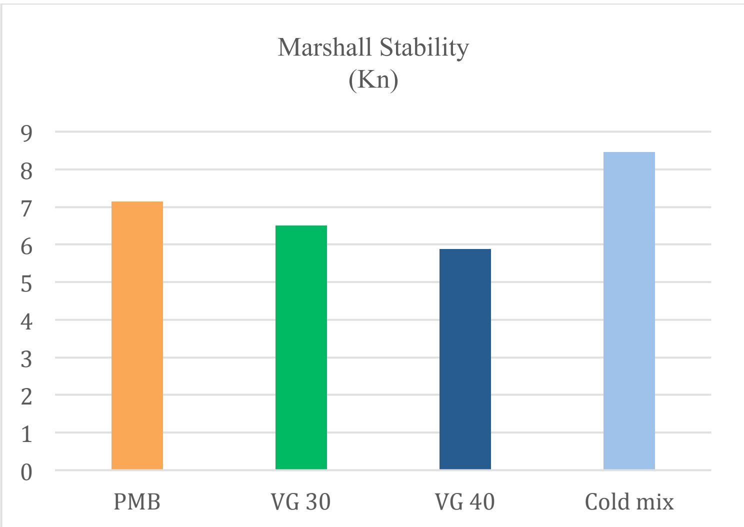


Fig. 6.5 Marshal Stability for CGBM and BC

Marshal stability test were performed after 60°C water bath conditioning of the sample for 30 minutes to get the idea of resistance to shear flow. Since CGBM mix with PMB40 has very less temperature susceptibility or performs well at higher test temperatures, which is also clear from Table 6.3, therefore marshal stability value of the PMB 40 is maximum as compared to VG30 and VG 40. Due to the rigidity of the grout material the stability value of all the CGBM are much higher as compared to BC sample. It can also be inferred that at worst condition of pavement in case of pavement temperature of 60^{o}C and submerged condition, CGBM will perform much better as compared to BC.

6.5 COMPRESSIVE STRENGTH TEST

Compression Test were performed on all variations of CGBM specimen and BC mix specimen having 100 mm diameter and 180mm height to determine the compressive strength of the CGBM and BC mix as per ASTM C39 as shown in Table 6.4.

Table 6.4

Compressive strength of different combinations of CGBM and BC

Specimen		Compressive Strength (MPa)	Correction Factor	Corrected Values (MPa)	Average Values (MPa)
CGBM-PMB 40	1	7.5	0.965	8.19	7.14
	2	6.77	0.976	6.61	
CGBM-VG 40	1	7.11	0.98	6.96	6.50
	2	6.16	0.98	6.04	
CGBM-VG 30	1	5.89	0.98	5.77	5.88
	2	6.11	0.98	5.99	
CGBM-Emulsion	1	8.56	0.98	8.38	8.46
	2	8.7	0.98	8.52	
BC	1	1.7	0.96	1.63	1.78
	2	2	0.96	1.92	

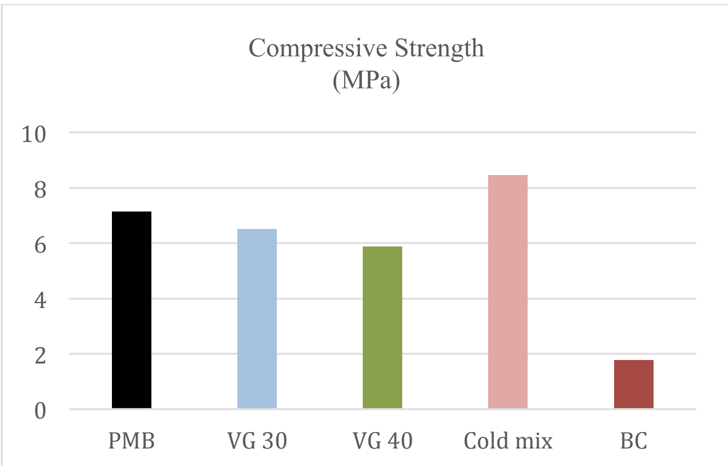


Fig 6.6. Compressive strength for CGBM mixes and BC mix

The compressive strength of the mixes is generally checked after 7 days curing to determine the resistance of this mixes toward axial loads or direct compressive loads so that the mixes does not get crushed under the action of these loads. The compressive strength of the emulsion mix CGBM samples is maximum at 8.46 MPa as compared to other combinations This may be because of the rigidity of the mix, where the volume of grout inflow is more as compared to other combinations due to thin binder film and less efficient coating which allows more flow of grout into the porous skeleton. But on a whole, all the combinations of CGBM have great compressive strength as compared to the literatures and when compared to BC, which has the compressive strength of about 1.78 MPa.

6.6 MOISTURE INDUCED SENSITIVITY TEST (MIST)

This test was basically performed to determine the susceptibility of CGBM and BC mixes to moisture induced damage due to water ingress into the pavement core under pressure due to the movement of vehicles in water logged surfaces of pavement, which will reduce the adhesion or bond between aggregate, binder and grout material and in turn leads to stripping failure. The results of MIST for different combinations of CGBM and BC samples are shown in Table 6.5.

Table 6.5

TSR values for CGBM and BC mixes

Specimen	ITS without MIST (MPa)	ITS after MIST (MPa)	TSR (%)	Limiting Value of TSR
CGBM-PMB 40	2.06	1.86	90.3	80%
CGBM-VG 40	1.86	1.72	92.5	
CGBM-VG 30	1.77	1.65	93.2	
CGBM-Emulsion	1.60	1.04	65	
BC	1.06	0.86	81	

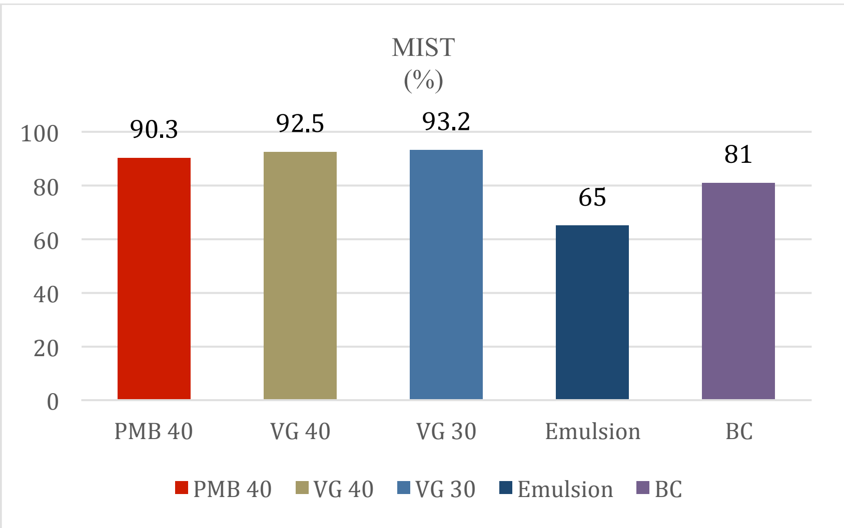


Fig. 6.7. TSR ratio for CGBM and BC

From Table 6.5, it is inferred that, TSR values for all the combinations of CGBM mixes are in the range of 90% to 94% and are approximately similar for all the variations of CGBM except that of Emulsion mix CGBM specimen. In case of Emulsion mix CGBM, due to the constant application of pressure at 40 psi at 60^{o}C (test conditions) coupled with less efficient coating of binder on the aggregates, the bond between the binder, grout and aggregates got affected. Due to the less efficient coating and under application of pressure, the adhesion between the aggregates and binder may got weaken and subsequently, the bond between binder and aggregates, which has made the mix less efficient under moisture induced stress damage.

Overall for other variations of CGBM mix with VG30, VG40 and PMB40, TSR ratio is almost similar and is well beyond the limiting criteria of 80% as per MoRT&H specifications, it depicts CGBM is very less susceptible to moisture damage as compared to BC. Due to rigid nature of the CGBM because of the grout material, the permeability is very less and as a result there is no or very little ingress of water under pressure and therefore the retained strength of CGBM samples are high as that of BC.

6.7 CANTABRO ABRASION LOSS (DURABILITY)

Cantabro abrasion loss test are per ASTM D7064, was carried out to establish the nature of CGBM mixes against durability. This test simulates the cohesion, bonding between bitumen the coated aggregates and the effects of abrasion of the mix. By varying the binder content CGBM mixes were prepared and tested for abrasion loss and then compared with BC also. This test was performed by allowing the samples to undergo 300 cycles without the application of steel balls in Los Angeles Abrasion Testing machine after curing the samples at 25^{o}C for 4 hours. The results are shown in Table 6.7.

Table 6.7

Effect on grade of Binder on abrasion loss.

Cantabro Abrasion Loss					
Specimen	Initial Weight (gms)	Final weight after test (gms)	Weight Loss (%)	Average Weight Loss (%)	Limiting Loss (%)
CGBM-PMB 40	1274.4	1089	14.55	15.83	
	1263.5	1109	12.23		
CGBM-VG40	1275.8	1011	21	22	
	1184.3	915	23		
CGBM-VG30	1245	898	28	26	20
	1196.1	916	24		
CGBM-Emulsion	1213	1079	11.05	13	
	1174	1002	14.62		
BC	1156	1121	3.12	3	
	1149	1118	2.7		

From the above test results, out of the all three grades of bitumen (VG30, VG40 and PMB40), PMB40 has shown notable aversion to disintegration with an average loss of 16% whereas in case of VG30 and VG40 CGBM mix abrasion loss is 26% and 22% respectively. This may be because of the less cohesion or bonding between the grout and the bitumen as compared to PMB40 mix. PMB 40 has shown improved cohesion and bonding with very less disintegration due to elastic properties as compared to VG30 and VG40 CGBM mix. But as compared to BC, the disintegration of CGBM is very high because of the rigid nature and presence of cementitious material, whereas BC due to the flexibility, the adhesion keeps the skeleton integrated and thus the loss is very less. Surprisingly, loss from emulsion mix CGBM is very less as compared to other variations. Although, the exact reason cannot be apprehended but this may be due to the more aggregate to aggregate locking due to the inefficient binder film. The disintegrated samples after test are shown in Photo 6.1.

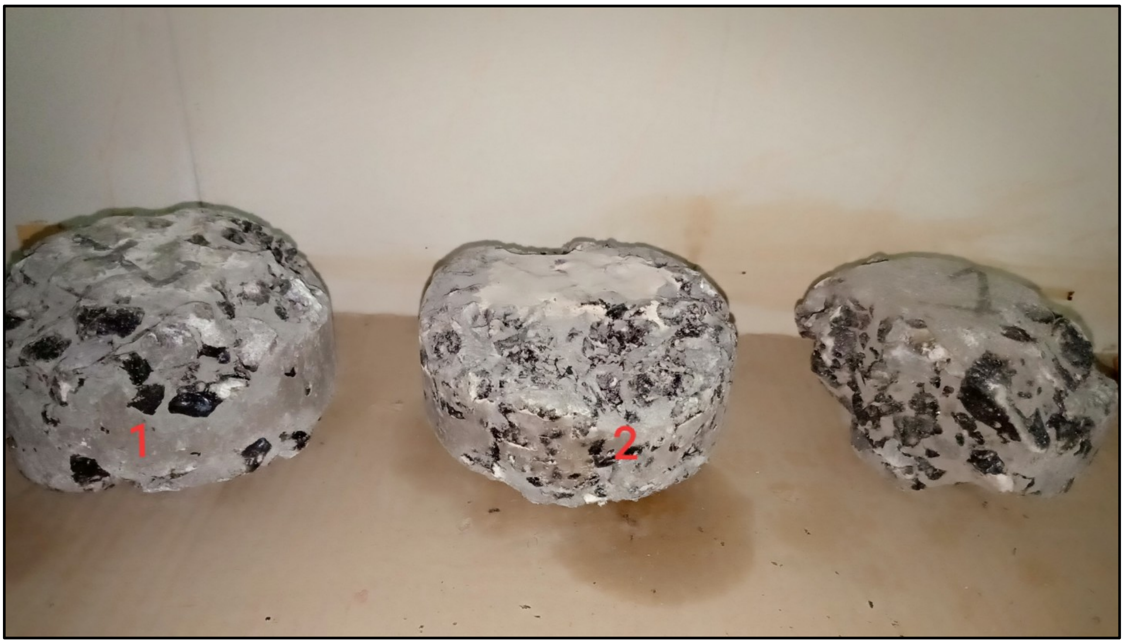


Photo 6.1. Specimens of (a) PMB40-CGBM (b) Emulsion-CGBM (c) VG-CGBM after abrasion test.

Various research studies have conducted Cantabro tests to assess the suitability of CGBM. However, the analysis done by various researchers revealed that this test has a very less significance for this type of mix.

6.8 DYNAMIC FRACTURE ENERGY (TEXAS OVERLAY TESTER)

Dynamic Fracture Energy of the CGBM mixes and BC mix were analyzed with the help of Texas Overlay tester. Dynamic Fracture Energy simulates the fracture properties of the mix. Two important parameters such as Critical Fracture Energy and Crack resistance Index were interpreted to define the fracture properties of CGBM. Critical Fracture energy (G_c) is the energy, which is required to develop the crack at the bottom of the specimen at the first loading cycle, and Crack Resistance Index (CRI) is the reduction in load required to propagate the cracks under the dynamic loading conditions, which interprets the flexibility of the specimen. CGBM with VG30, PMB40 and BC were tested in the overlay tester at a predefined displacement of 0.4 mm (Strain controlled) at test temperature of 25^{o}C.Specimen having dimension 38 mm height and 76 mm width is fixed in two plates which is further mounted in the test assembly. One of the plates is fixed and other is sliding. The sliding block applies tension force in a dynamic triangular waveform to a predefined displacement. The sliding plate reaches the maximum displacement and then returns to its initial position within 10 seconds (1 cycle). The peak load corresponding to the displacement of

0.4mm was noted first and the failure point was defined at 93% of the load reduction as per the test procedure according to Tex-248-F.

While in the case of bituminous samples, it is well specified in Tex-248-F code to run the test at predefined displacement of 0.6mm, therefore for bituminous samples, the predefined displacement considered was 0.6 mm. In CGBM mixes, there is no such established data. In our study, IITPAVE analysis was done incorporating both CGBM and BC as a surface layer to analyze the difference in strain value for both the pavement composition. It was analyzed that, horizontal strain generated at the base of the surface layer in CGBM is less than 50% of the strain generated in case of pavement incorporating BC. Therefore, for dynamic fracture analysis, 0.4mm displacement is considered in CGBM.

The Peak load is the load, which is required to achieve 0.4 mm displacement. The phenomenon of loading is cyclic loading in overlay tester and the energy require to develop the initial crack at peak load is the critical fracture energy. Once the crack appears, the load reduced and test terminates at 93% load reduction. The load drop and number of cycles to failure can be interpreted from the test. The results of the test are shown in Table 6.7.

Table 6.7

Dynamic Fracture Energy Characteristics for CGBM and BC

Binder Grades	PMB 40	VG 40	BC
Crack Resistance Index (CRI)	60	76	88
Critical Fracture Energy (W_c) (Joule)	136	183	204.5
Peak Load (kN)	4.43	3.66	1,89
Load at Failure/Test Termination (kN)	0.306	0.0225	0.048
Displacement (mm)	0.4	0.4	0.4
No. of cycles to failure/Test Termination	215	290	126
Percent decline in the load (%)	93	99	97
Test Termination Reason	Peak Load Reduction	Peak Load Reduction	Peak Load Reduction

From Table 6.7, it can be concluded that, Critical fracture energy for CGBM specimens are less as compared to BC samples, which depicts the brittleness of the CGBM mix and flexibility of the BC. Due to the brittleness of the CGBM mixes, the energy require to develop the initial crack on the bottom of the surface is less as compared to BC. It was also seen that, crack propagates in the CGBM samples within first few cycles (15-20 cycles) whereas in case of BC samples, the crack propagation to the surface takes more cycles than CGBM. However, it is clearly evident from the table 6.7 that, the peak load require for CGBM samples (4.6 kN) to develop that displacement of 0.4mm is almost 2.5 times than that of BC samples (1.9 kN). Even if, the CGBM mix is brittle, but the load require developing the failure strain is more.

In terms of Crack Resistance Index, i.e. reduction in load required to propagate cracking under dynamic loading, is less in CGBM samples as compared to BC, this also highlights the brittle nature of CGBM mixes. The reduction in load with Cycles phenomenon is shown in Figure 6.7.

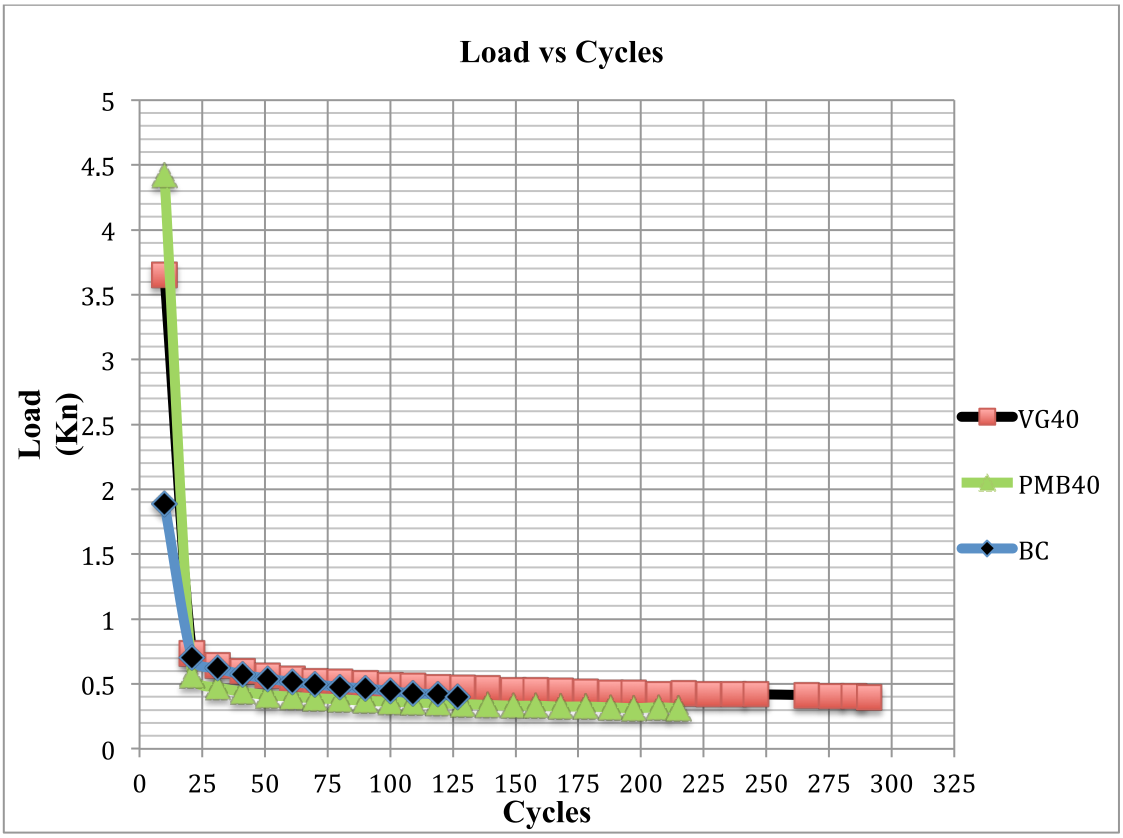


Figure 6.8. Load Vs Cycles for determination of Crack resistance Index (CRI)

The Figure 6.8 shows load required to develop crack in the mix vs no. of cycles to attain 93% of load required for crack development. It is seen from the graph that CGBM mix takes more than 2.5 times load as compare to BC mix which indicate higher load bearing capacity of CGBM as compare to BC mix. Further total no. of cycles required attaining 93% of load reduction or failure is almost 2 times in CGBM as compare to BC mix which indicate higher life of CGBM as compare to BC mix. Also it can be well defined from the graph that CGBM mix using VG 40 bitumen takes 25 % more load as compare to CGBM mix using PMB to develop cracks in mix.

Through limited samples were tested, it can be concluded that CGBM mix if used as wearing course as overlay on existing bituminous layer in Flexible pavement may have higher load bearing capacity and higher fatigue life and retard reflective cracks as compare to overlay of BC mixes.

From Table 6.7, it is evident that, the number of cycles to failure or test termination is more for CGBM samples of PMB40 and VG40 as compared to BC, which is also very clear from the above graph. This shows that, even if after crack initiation within first few cycles or steep reduction in load, the CGBM samples can withstand the stress developed at 0.4mm displacement and fails at 290 cycles and 215 cycles for VG40 and PMB 40 CGBM mix. However, in the case of BC, the cycles to failure is 126 cycles, which is much lesser than CGBM samples, which depicts as soon as the crack propagates or appears in the specimen, the resistance to cyclic load for BC is less and cannot resist stress developed due to 0.4 mm displacement and thus the test got terminated.

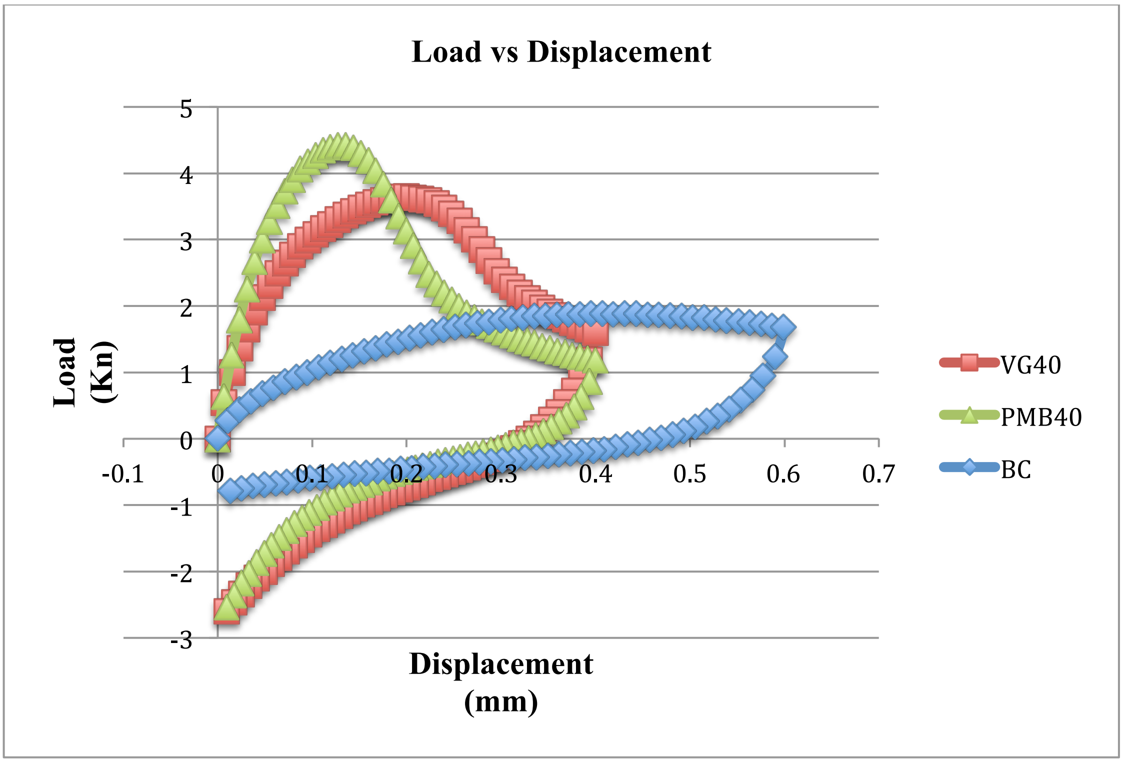


Figure 6.9. Load vs Displacement curve to obtained under dynamic loading

The Figure 6.9 shows load required to develop crack and displacement in the CGBM mix. It is seen from the graph that CGBM mix takes less displacement as compare to BC mix. In general, CGBM specimens sustained maximum stress than the bituminous mix. However, after reaching the maximum value, stress in CGBM specimens decreases faster than that in BC mix specimens, which can be depicted from the steep nature of the load displacement curve for CGBM specimens showing a more brittle behaviour of CGBM mixes. Though BC mix shows gradual reduction in load but it takes fewer loads and less no. of cycles compare to CGBM. It indicates semi flexible behavior of CGBM.

CHAPTER 7

7. PAVEMENT DESIGN

Cement Grout Bituminous Macadam is a great alternative to composite pavement. In India very few research studies have been conducted on CGBM and therefore the design parameters are not yet established. Although it comes under the category of semi flexible pavement, but the design parameters followed as per IRC 37-2018 guidelines cannot be implemented into this as the failure pattern/ propagation of cracks has not yet been established for such kind of pavements. Here in our study, the crust composition presumed for design consists of a layer of 40 mm CGBM over a layer of 50 mm BC (CGBM layer being a rigid layer cannot be directly laid on WMM, which is an unbound layer) 250 mm WMM and 200 mm GSB over a sub-grade having CBR of 8%. Mechanistic Elastic layer approach as given in IRC: 37-2018, is used for the design and analysis of the composite pavement. For structural analysis IITPAVE software were carried out to check the structural adequacy of the pavement.

7.1 FAILURE CRITERIA

The failure criteria considered in the study were horizontal tensile strain in the bottom of the bituminous layer and CGBM layer, tensile stress at the CGBM layer and Vertical strain at the top of the sub-grade layer. All these criteria must meet the limiting values to consider the pavement design to be safe.

a) *Horizontal tensile Strain at the Bottom of the Bituminous Layer and CGBM*:

The fatigue model, according to IRC: 37-2018 considers the allowable horizontal tensile strains at the bottom of the bituminous layer. These horizontal tensile strains are the cause of the cause of fatigue failure, where the cracks develop at the bottom of the bituminous layer propagates to the top and leads to fatigue failure.

In our analysis, Since CGBM is a new concept in India and due to lack of research data, 80% reliability concept for 20 MSA is considered. The fatigue model used in this study corresponding to 80% reliability and 20 MSA are given in equation 7.1

$$N_f = 1.6064*C*10^{-04}\,[1/\varepsilon_t]^{3.89}\,[1/MR]^{0.854} \text{ (for 80 \% reliability)} \quad (7.1)$$

C = 10^M and $M = 4.84\left(\frac{V_b}{V_a+V_b} - 0.69\right)$

Where,

V_a= volume of air void in the mix used in the bituminous layer

V_b= per cent volume of bitumen in the mix.

N_f = fatigue life

ε_t = maximum horizontal tensile strain at the bottom of the bituminous layer

M_R = resilient modulus of the bituminous mix.

The horizontal tensile strain at the bottom of the CGBM layer has been evaluated according to a research study conducted on CGBM by IIT Kharagpur, where they develop a fatigue equation as shown equation 7.2,

$$N = 10^{17.6019} * (\frac{1}{\varepsilon_t})^{4.6099} * (\frac{1}{E})^{0.6171} \quad (R^2 = 0.77) \tag{7.2}$$

Where,

N= Fatigue life of the CGBM layer

ε_t=Maximum Horizontal Tensile strain at the bottom of CGBM.

E= Elastic Modulus.

Another Fatigue life equation has also been developed by university of Nottingham, horizontal tensile strain at bottom of CGBM can be been determined using Eq.9.3 (Oliveira et al., 2008)

$$N = 2.7 * 10^{-9} * \varepsilon_t^{-3.9718} \tag{9.3}$$

Where, N = fatigue life of CGBM in MSA

ε_t= maximum tensile strain at bottom of CGBM

From laboratory Evaluation, the elastic modulus of the CGBM was evaluated to be in the range of 10000-15000 MPa at 35oC. However, according a study done by CRRI and IIT Kharagpur, due to the variation in aggregates or aggregate gradation, depth of penetration of grout, Volume of Voids after grouting, Stiffness of the grout material and cracks caused to the movement of the construction traffic, a reduced modulus value of 5000 MPa may be considered, which will also not make the design uneconomical.

b) *Modulus of Rupture and Tensile stress developed at the CGBM layer:*

The modulus of rupture is an important phenomenon to be checked in CGBM. The modulus of Rupture value of 1.25 MPa may be considered for design purpose. This value should be less than the stress developed at the layer of CGBM, which can be evaluated through IITPAVE analysis.

c) *Rutting/Permanent Deformation at top of Sub-grade:*

Rutting failure occurs generally when the vertical strain on the subgrade exceeds the bearing capacity or bearing stress of soil. The model used is given in equation 7.4, as per IRC: 37-2018,

$N_R = 4.1656 \times 10^{-08} [1/\varepsilon_v]^{4.5337}$ (for 80 % reliability) (7.4)

Where,

N_R = cumulative standard axles

ε_v = vertical strain on sub-grade top.

7.2 DESIGN OF PAVEMENT

Elastic Layer Mechanism is applied for the design of the pavement incorporating CGBM. The design was carried out according to IRC: 37-2018 and IITPAVE software for structural evaluation and the design parameters considered are shown in Table 9.1.

Table 7.1

Design Parametrs

Design Life	10 Years
MSA	20
Temperature(^oC)	35
Poisson's Ratio	0.25 for CGBM 0.35 for Bituminous layers
CBR%	8
Tire Pressure(MPA)	0.56
Equivalent Axle Load(kN)	80
VDF	3.5
Elastic Modulus (Bituminous Layer) (MPa)	2000
Elastic Modulus for CGBM (MPa)	5000
Elastic Modulus Granular layers (MPa)	208
Elastic Modulus of Subgrade (MPa)	67

Presumed pavement crust composition incorporating CGBM has been shown in Table 7.2.

Table 7.2

Pavement Crust Composition

CBR	MSA	Pavement Crust Composition (mm)				
		CGBM	BC	WMM	GSB	Total
8%	20	40	50	250	200	40

The allowable strains are calculated as per fatigue and rutting model for bituminous layers and CGBM layer. Allowable fatigue strain for bituminous layers has been calculated as per equation 7.1 and for CGBM layer as equation 7.2 and 7.3. Allowable vertical strain at top of the sub grade has been computed using equation 9.4. Pavement structural analysis for actual horizontal strain at the bottom of the bituminous layer, bottom of CGBM layers, tensile stress at the bottom of CGBM layer and vertical compressive strain on top of sub grade for 20 MSA with design CBR of 8% as per IITPAVE is shown in Table 7.3.

Table 7.3

Pavement Structural analysis for 20 MSA

Horizontal tensile strain in bottom bituminous layer		Tensile strain in CGBM		Tensile stress in CGBM layer		Vertical Compressive strain on sub grade	
Actual Strains (Micro)	Allowable strains (Micro)	Actual Strains (Micro)	Allowable strains (Micro)	Actual Strains (MPa)	Allowable stress (MPa)	Actual Strains (Micro)	Allowable strains (Micro)
318.5	287.53	42.78	54.87	0.14	1.25	411.2	577.7

From the above table, it is quite evident that, pavement is failing in fatigue as the actual horizontal tensile strain at the bottom of bituminous layer is more than that of the allowable strain. Therefore the thickness of the bituminous layer needs to be increased and 50 mm BC is not sufficient. Detailed IITPAVE output is being presented in Figure 7.1.

```
IITPAVE fail - Notepad
File  Edit  Format  View  Help
 No. of layers               4
 E values (MPa)         5000.00 2000.00  208.00   67.00
 Mu values                 0.250.350.350.35
 thicknesses (mm)         40.00   50.00  450.00
 single wheel load (N) 20000.00
 tyre pressure (MPa)       0.56
 Dual  Wheel
   Z        R      SigmaZ      SigmaT     SigmaR      TaoRZ      DispZ       epZ        epT        epR
  40.00    0.00-0.4098E+00 0.1361E+00 0.9869E-01-0.2440E-01 0.5053E+00-0.9371E-04 0.4278E-04 0.3342E-04
  40.00L   0.00-0.4098E+00-0.9651E-01-0.1104E+00-0.2440E-01 0.5053E+00-0.1687E-03 0.4278E-04 0.3343E-04
  40.00  155.00-0.8284E-01 0.3383E-01-0.3945E+00-0.3450E+00 0.5042E+00 0.1463E-05 0.3063E-04-0.7644E-04
  40.00L 155.00-0.8284E-01-0.3577E-01-0.1944E+00-0.3450E+00 0.5042E+00-0.1142E-05 0.3063E-04-0.7644E-04
  90.00    0.00-0.1960E+00 0.7891E+00 0.6308E+00-0.1969E-01 0.4932E+00-0.3465E-03 0.3185E-03 0.2116E-03
  90.00L   0.00-0.1960E+00-0.1251E-01-0.2898E-01-0.1970E-01 0.4932E+00-0.8726E-03 0.3185E-03 0.2116E-03
  90.00  155.00-0.1461E+00 0.5699E+00 0.5516E-01-0.9209E-01 0.4992E+00-0.1824E-03 0.3009E-03-0.4658E-04
  90.00L 155.00-0.1461E+00-0.1123E-01-0.6476E-01-0.9209E-01 0.4992E+00-0.5746E-03 0.3009E-03-0.4658E-04
 540.00    0.00-0.2400E-01 0.3545E-01 0.3009E-01-0.4234E-02 0.3196E+00-0.2257E-03 0.1602E-03 0.1254E-03
 540.00L   0.00-0.2417E-01 0.2596E-02 0.9129E-03-0.4228E-02 0.3196E+00-0.3791E-03 0.1602E-03 0.1263E-03
 540.00  155.00-0.2603E-01 0.3803E-01 0.3441E-01-0.6082E-02 0.3292E+00-0.2470E-03 0.1687E-03 0.1453E-03
 540.00L 155.00-0.2603E-01 0.2751E-02 0.1577E-02-0.5838E-02 0.3292E+00-0.4112E-03 0.1688E-03 0.1452E-03
```

Figure 7.1. IITPAVE output for first analysis.

Since the pavement is failing for the above crust composition, therefore the crust composition considered for further design is shown in table 7.4 and IITPAVE analysis in Table 7.5.

Table 7.4

Final Pavement Crust Composition

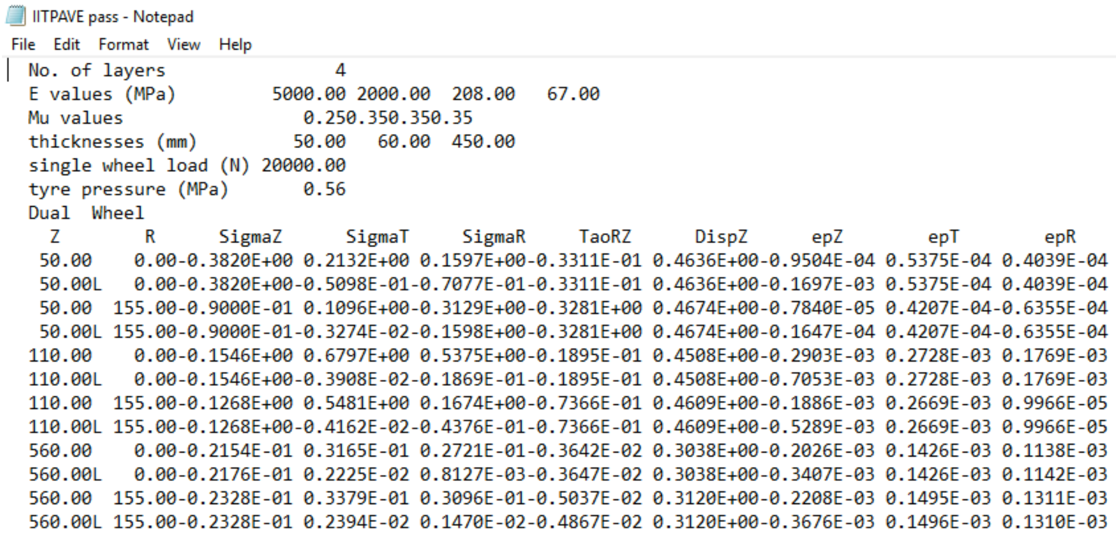
CBR	MSA	Pavement Crust Composition (mm)				
		CGBM	BC	WMM	GSB	Total
8%	20	50	60	250	200	40

Table 7.5

Pavement Structural analysis for 20 MSA-Safe Design

Horizontal tensile strain in bottom bituminous layer		Tensile strain in CGBM		Tensile stress in CGBM layer		Vertical Compressive strain on sub grade	
Actual Strains (Micro)	Allowable strains (Micro)	Actual Strains (Micro)	Allowable strains (Micro)	Actual Strains (MPa)	Allowable stress (MPa)	Actual Strains (Micro)	Allowable strains (Micro)
272.8	287.53	53.75	54.87	0.21	1.25	367.6	577.7

Table 7.5 shows that all the stress and strain values are well within limits and therefore the thickness of the crust considered as per table 7.4 is safe. Detailed IITPAVE outputs are shown in Figure 7.2 for better view.



IITPAVE pass - Notepad

File Edit Format View Help

```
 No. of layers              4
 E values (MPa)         5000.00 2000.00   208.00    67.00
 Mu values                 0.250.350.350.35
 thicknesses (mm)         50.00    60.00  450.00
 single wheel load (N) 20000.00
 tyre pressure (MPa)       0.56
 Dual  Wheel
   Z        R      SigmaZ      SigmaT      SigmaR      TaoRZ      DispZ       epZ        epT        epR
  50.00    0.00-0.3820E+00 0.2132E+00 0.1597E+00-0.3311E-01 0.4636E+00-0.9504E-04 0.5375E-04 0.4039E-04
  50.00L   0.00-0.3820E+00-0.5098E-01-0.7077E-01-0.3311E-01 0.4636E+00-0.1697E-03 0.5375E-04 0.4039E-04
  50.00  155.00-0.9000E-01 0.1096E+00-0.3129E+00-0.3281E+00 0.4674E+00-0.7840E-05 0.4207E-04-0.6355E-04
  50.00L 155.00-0.9000E-01-0.3274E-02-0.1598E+00-0.3281E+00 0.4674E+00-0.1647E-04 0.4207E-04-0.6355E-04
 110.00    0.00-0.1546E+00 0.6797E+00 0.5375E+00-0.1895E-01 0.4508E+00-0.2903E-03 0.2728E-03 0.1769E-03
 110.00L   0.00-0.1546E+00-0.3908E-02-0.1869E-01-0.1895E-01 0.4508E+00-0.7053E-03 0.2728E-03 0.1769E-03
 110.00  155.00-0.1268E+00 0.5481E+00 0.1674E+00-0.7366E-01 0.4609E+00-0.1886E-03 0.2669E-03 0.9966E-05
 110.00L 155.00-0.1268E+00-0.4162E-02-0.4376E-01-0.7366E-01 0.4609E+00-0.5289E-03 0.2669E-03 0.9966E-05
 560.00    0.00-0.2154E-01 0.3165E-01 0.2721E-01-0.3642E-02 0.3038E+00-0.2026E-03 0.1426E-03 0.1138E-03
 560.00L   0.00-0.2176E-01 0.2225E-02 0.8127E-03-0.3647E-02 0.3038E+00-0.3407E-03 0.1426E-03 0.1142E-03
 560.00  155.00-0.2328E-01 0.3379E-01 0.3096E-01-0.5037E-02 0.3120E+00-0.2208E-03 0.1495E-03 0.1311E-03
 560.00L 155.00-0.2328E-01 0.2394E-02 0.1470E-02-0.4867E-02 0.3120E+00-0.3676E-03 0.1496E-03 0.1310E-03
```

Figure 7.2. IITPAVE analysis for final thickness

Final crust composition for CGBM incorporated pavement is illustrated in Figure 7.3.

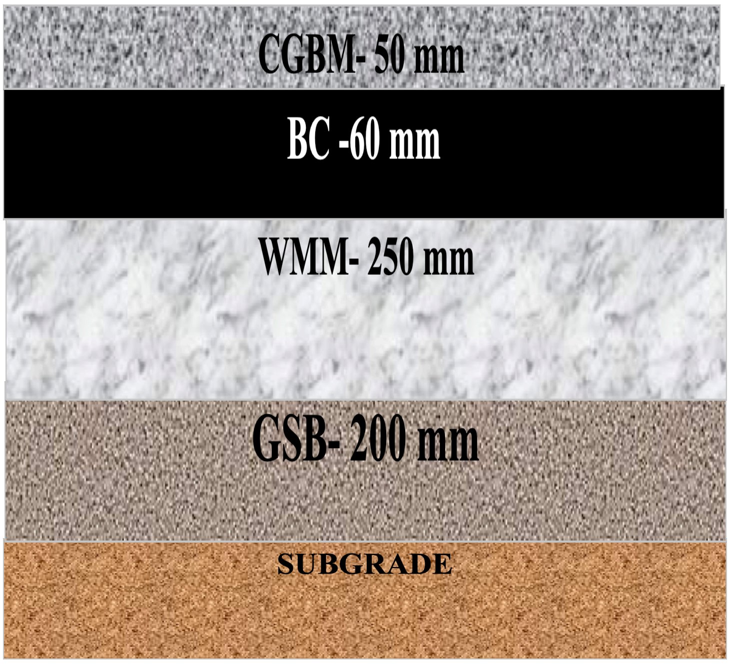


Figure 7.3. Pavement Crust Composition for CGBM

CHAPTER 8

8. CONCLUSIONS AND FURTHER SCOPE

8.1 INTRODUCTION

The main objective of the present study was to evaluate CGBM for its mechanistic properties and performance in fatigue, moisture resistance and durability by varying the different grades of Binder like VG30 (standard mix), VG40, PMB40 and Emulsion. Further the standard mix of CGBM was also compared with BC.

The following main issues were examined in the present study.

- ✓ Identification of volumetric parameters for preparation of high void bituminous mix with different combinations of Binder.
- ✓ Study on cementitious grout material for the consideration of flow ability and strength.
- ✓ Evaluation of Mechanical properties such as Marshal stability, Indirect Tensile Strength, Resilient Modulus, Compressive Strength and performance in terms of, Fracture Energy, Moisture Resistance, Durability.
- ✓ Comparison of the characteristics of CGBM with that of conventional bituminous concrete mix.
- ✓ Design of Pavement incorporating CGBM

8.2 OUTCOMES OF THE STUDY

The following are the outcomes of the study –

- PMB40-CGBM mix has generally less void content with respect to other combinations, although the difference is very less. Bitumen Emulsion based CGBM mix has the high void content, which might be attributed to the fact that, the binder film thickness over the aggregates in case of emulsion based CGBM mix is less, which results in higher void content.
- With the increase in binder content, its theoretical maximum density decreases, and thus the volume of air voids also decreases. Although there is a marginal difference in the decrease of air voids with an increase of 0.5% binder content from OBC.
- With the increase in water content, the flow ability of grout increases and compressive decreases.

- PMB 40-CGBM mix had significantly higher Resilient Modulus value as compared to other mixes. Moreover, the reduction of resilient modulus value at higher test temperature of 45^{o}C was very less of about 15%, which shows less temperature susceptibility of PMB40-CGBM mix, as compared to 33% for VG30-CGBM mix and thus PMB40 will perform better at higher temperatures as compared to other grade of binders.
- With the increase in temperature, the resilient modulus value decreases in CGBM; this shows the semi-flexible nature of the mix.
- CGBM mixes showed higher resilient modulus value as compared to BC, and have very less temperature susceptibility as compared to BC. CGBM-VG30 mix has only 33% reduction in stiffness from 25^{o}C to 45^{o}C, whereas in BC there was 80% reduction in the stiffness.
- On increasing the binder content, the stiffness value of the CGBM mix gets reduced, which means the flexibility can be increased by reducing the binder content, or increased by reducing the binder content. But it has to be noted that, binder content should be increased up to suitable content so as not to effect the penetration of flow of grout.
- Emulsion mix-CGBM, shows better stiffness as compared to VG 30 and VG 40 mix at 45^{o}C test temperature. In emulsion mix due to the less efficient coating of binder, grout material dominates and thus because of the cementitious nature, higher temperature effect is less on the emulsion mix CGBM
- The ITS value of PMB40-CGBM mix is higher as compared to other mix such as VG30, VG40 and emulsion mix, which shows good fracture strength.
- ITS value at higher temperature of PMB40-CGBM is notably less as compared to VG30, VG40 and PMB40. Its value reduced by 37% as compared to 50% in case of VG30.
- Because of the rigid nature of the Emulsion mix-CGBM, the ITS value of the mix is higher than the VG mixes at 45^{o}C.
- CGBM mixes showed much higher ITS and thus high fracture strength as compared to BC. BC mix is very much susceptible to temperature changes with respect to CGBM mixes, especially at higher temperatures.
- Marshal Stability value of PMB40-CGBM mix is 98 kN, which is higher than VG40 and VG30 of value 85 kN and 78 kN. PMB40 showed the resistance to

shear at higher temperature due to its flexibility, corresponding to the test conditions of 60^{o}C water bath before test.

- CGBM has much high stability as compared to BC, which has stability value of 13 kN.
- As already discussed, due to less efficient coating and thin binder film in Emulsion-mix, grout volume into the mix is more thus the stability value is more than that of VG 30 and VG 40.
- Compressive strength of Emulsion mix CGBM is higher than that of all the variations because of the probable concrete nature due to less binder content. PMB40 has higher compressive strength (good resistance to crushing) of 7MPa as compared to 5.88 MPa and 6.5 MPa of VG30 and VG40 respectively. The compressive strength of BC is very less, almost 4 times (1.78 MPa) less than that of CGBM.
- Moisture Induced damage depicted through Retained Tensile Strength (TSR ratio) is almost similar, in-between 90%-94% for all CGBM mixes except Emulsion mix.
- TSR ratio of BC is 80% as compared to 90% of CGBM mix, which shows its susceptibility towards stripping due to the induction of moisture. CGBM has less susceptibility to Moisture induced damage due to its impermeable nature.
- Emulsion mix has shown the lowest TSR value of 65% as compared to other mixes and thus failing in the criteria of Moisture induced damage limiting to 80% as per MoRT&H specifications. Therefore further study is required in this regard.
- The abrasion loss of PMB40 mix-CGBM specimen has average abrasion loss of 16%, which shows improved cohesion in the mix and has shown excellent resistance to disintegration as compared to VG 30, which has a loss of about 27%. This may be because of the better cohesion and bonding due to elastomeric property of PMB40.
- It was scene that, there was a edge breaking phenomenon in case of CGBM mix, as the disintegration of the mix has taken place mostly from the edges of the marshal samples of CGBM.

- BC mix due to its flexibility has an average abrasion loss of only 3%, which is very much expected due to the flexibility and integration due to the binder with the aggregates.
- According to the research studies conducted by CRRI on CGBM and based upon other research studies, It was established that CGBM is not susceptible to rutting due to the stiffness and rigidity provided by the grout material as a result rutting phenomenon is not incorporated in this study. Although from literatures and research studies, performance of CGBM in rutting is much high as compared to BC.
- In Dynamic Fracture Energy test, CGBM mix has taken more than 2.5 times load as compare to BC mix which indicate higher load bearing capacity of CGBM as compare to BC mix during the first cycle to initiate the crack.
- Total no. of cycles required attaining 93% of load reduction or failure is almost 2 times in CGBM as compare to BC mix which indicate higher life of CGBM as compared to BC mix.
- It can be seen that CGBM mix using VG 40 bitumen takes 25 % more load as compare to CGBM mix using PMB to develop cracks in mix.
- Though CGBM mixes takes more load to predefined displacement (0.04mm), but the reduction in the load in CGBM mixes is steep as compare to BC mixes, where the reduction in the peak load is quite gradual, this shows the brittle nature of the CGBM mix.
- Although in BC mix the load reduction is gradual but it takes fewer loads and less no. of cycles to failure compare to CGBM. It indicates semi flexible behaviour of CGBM.
- For pavement consisting of CGBM as a surface layer, the thickness adopted and analysed was 50mm-CGBM, 60mm-BC, 250mm-WMM and 200mm-GSB, and are safe according to IITPAVE analysis as the actual values of stress and strain are within limits.
- It can be concluded that CGBM mix if used as wearing course as overlay on existing bituminous layer in Flexible pavement may have higher load bearing capacity and higher fatigue life and retard reflective cracks as compare to overlay of BC mixes.

8.3 SCOPE FOR FURTHER STUDY

- The study was conducted on various grades of Bitumen like VG30, VG40 and PMB40, and also using bitumen emulsion of MS grade. The test results on CGBM have shown progressive behaviour using softer grade to harder grades of bitumen including PMB40. However, the test results on CGBM using Bitumen emulsion have shown dissatisfactory results and the results were not conclusive. Therefore, further research study is required on the use of suitability of different grades of Emulsion/Modified Bitumen Emulsion on CGBM.
- Incorporating rich bituminous mix for CGBM, to improve the flexibility of the skeleton.
- To Study the effect of Thermal stresses on CGBM mix.

Appendix A: Procedure and Analysis of Overlay Test (Tex-248-F)

<u>Scope:</u>

a) To determine the critical fracture energy (G_c) and crack resistance index (CRI) which characterize the fracture resistance of bituminous mixtures.
b) Susceptibility of bituminous mixtures to reflective cracking.

<u>Terms:</u>

a) *Critical Fracture Energy (Gc):* The energy needed for a sample to initiate a crack at the bottom during the first loading cycle of the Texas overlay tester, which simulates the fracture strength of the mix.
b) *Crack Resistance Index (CRI):* The reduction in the percentage of peak load due to the subsequent propagation of the crack under cyclic loading, which shows the flexibility of the specimen.

<u>Apparatus:</u>

a) *Overlay Teste*r: It follows an Electro-Hydraulic mechanism which applies direct tensile loads to the specimen. Loads, displacement and temperature are the parameters which is being measured by this device every 0.1 sec. The device consists of two blocks, one is fixed and the other one slides horizontally. The sliding block applies tension in a triangular cyclic form to get a constant displacement of 0.06mm before returning to its original position to form an entire cycle of 10 seconds.

   It additionally includes, LVDT to measure the displacement, load cell, temperature measuring gauges, and steel base plates to fix the specimen and a 4.2mm thick spacer bar. Refer Figure 6.9 for having an idea about the entire assembly.

<u>Specimen:</u>

a) Laboratory made specimen of 150 mm diameter (Modified Marshal samples) shall be casted according to the requisite density either through gyratory compactor or through modified marshal compactor, which is further trimmed to the test specimen of height 38 mm and width 76 mm as shown in Figure 6.8.

Specimen Mounting:

a) Two-part epoxy glue, meeting the requirements to Tex-614-J, shall be used to glue the specimens on the plates.

b) The base plates shall be mounted to the mounting jig and fixed. Insert the spacer bar of thickness 4.2mm to maintain a distance of 4.2 mm between the base plates.

c) On the specimen draw a line in the middle to guide the placement of the specimen on the base plates and then fix a tape of 4.2 mm width on the face of the specimen facing the base plates, which shall be removed once the specimen is mounted.

d) Glue the base plates by taking 16 gms of glue (8gm each part) and mix them well and uniformly for some time before mount the specimen over the glued base plates and then put a weight of 5-lb on the glues specimen to ensure full pressure on the specimen to form full contact on the base plates. Any epoxy on the sides shall be removed as it may cause an appropriate test results, carefully remove the tape and spacer bar from the middle after gluing (Photo A-1).

e) For adequate bonding, allow the epoxy to cure for minimum 24 hours. The specimen should be well in shape and size and the glue shall be uniformly applied on the base plates so that glue failure does not take place during the test.

Photo A.1: Mounted specimen ready for test

Testing Specimens:

a) The test shall be performed at a test temperature of 25oC. Specimen shall be cured minimum for 2 hours before testing.
b) 10 minutes relaxation time shall be given prior to start the test and the test will run automatically after the relaxation period is over.
c) The test will run until 93% reduction of peak load or 900 cycles whichever is earlier. If there is need to determine the number of cycles to propagate the crack at the surface, the specimen needs to be painted and closely visualise to determine the number of cycles to crack reflection to the surface.

Calculations:

a) Critical Fracture Energy at maximum peak load:

$$G_c = \frac{W_C}{b * h} \quad \text{(A-1)}$$

Where,

Gc= Critical Fracture energy (kN-mm2)

Wc= Fracture area (Area under Load displacement curve (Figure 8.7)

B= Width of the specimen (mm)

H= Height of the specimen (mm)

b) Crack resistance index shall be calculated based on load vs no. of cycles curve (figure 8.6), using a power equation,

$$y = x^{(0.00756\beta - 1)} \quad \text{(A-2)}$$

Where,

β is the crack resistance index

y= Load reduction

x= no. of cycles

c) Report the maximum load, Critical Fracture Energy, Crack resistance Index and no. of cycles to failure
d) However, nowadays, a programme has been established which calculates the critical fracture energy and crack resistance index automatically and the data can be exported from the machine with the help of external storage drives. Figure A-1 shows the output window.

Filename:	22_05_2019_13_23_39_p40.txt										

OVERLAY TESTER DATA FILE											

DATE AND TIME											
User Name:	CRRI										

SAMPLE INFO											

Sample ID:	p40										
Replicate Number:	1										
Sample Description:	pmb-40										
Sample Sitting Time (DD:HH):	00:00										
Trimmed Specimen Density (kg/m³):	2.45										

TEST PARAMETERS											

Failure Limit (%):	93										
Cycle Limit:	900										
Test Temperature (C):	25										
Loading Displacement Rate(mm/s):	0.08										
Frequency (Cycle Time (s)):	10										
Recording Interval (s):	0.1										

TEST SUMMARY											

Starting Peak Load (kN):	4.42938										
Final Load (kN):	0.306597										
Percent Decline in Load (%):	93										
Number of Cycles to Failure or Test Termination:	215										
Number of Observed Cracks:	0										
Critical Fracture Energy (J):	136.212										
Crack Progression Rate:	-0.5										
Crack Resistance Index:	60										
Stop Reason:	PEAK LOAD REDUCTION										

TEST DATA											

Time(s)	Cycle Number	LVDT1(m	LVDT2(m	Displace	Load(kN)	Cycle Pea	Chamber	Dummy T	CFE (J/m²	CPR	CRI
0.001	1	0.4425	0.4283	########	-0.003	0	26.002	24.612	0	0	
0.101	1	0.4351	0.4225	0.0066	0.6293	0.6293	26.102	24.612	0.8106	0	
0.201	1	0.4263	0.4149	0.0148	1.2555	1.2555	26.002	24.612	3.5644	0	

Figure A.1: Output Window of Texas overlay test

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