ESTIMATION OF MODULUS OF RESILIENCE OF HOT MIX ASPHALT BY UTILIZING THE INDIRECT TENSILE STRENGTH TEST DATA OF THE BITUMINOUS MIXES

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Submitted By

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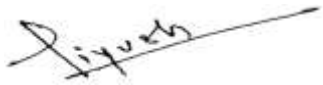
DECLARATION

I, Piyush Prasad hereby declare that the work presented in this dissertation entitled "Estimation Of Modulus Of Resilience Of Hot Mix Asphalt By Utilizing The Indirect Tensile Strength Test Data Of The Bituminous Mixes" in fulfillment of the requirement for the award of degree of Master of Engineering (CE) submitted at Civil Engineering Department, Thapar Institute of Engineering & Technology (Deemed to be University), Patiala is an authentic record of work carried out under supervision of Dr.Ambika Behl (Head and Principal Scientist, Flexible Pavement Division, CSIR-CRRI) and Dr.Tanuj Chopra (Assistant Professor, Civil Engineering Department, Thapar Institute of Engineering & Technology from 15/07/2021 to 15/06/2022. The matter presented in this has not been submitted either in part or full to any other university or institute for the award of any other degree.



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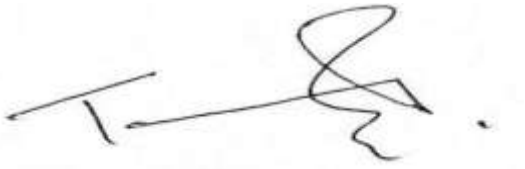

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ABSTRACT

The value of modulus of resilience is a prime variable, which is being used by the pavement engineer and pavement designer in the design of pavement as well as in the evaluation of pavement performance subjected to hammering effect due to dynamic traffic loading. There are many empirical methods developed by the expert researchers to determine the resilient modulus based upon the factor which is affecting it mostly or the parameters which is showing good dependency. As a result, the dependence of resilient modulus on indirect tensile strength (ITS) test data has been a topic of interest. This research study aims to provide approach to validate the analysed ITS test result of bituminous mix with the laboratory resilient modulus values thereby reducing the need for testing on expensive equipment and trained personnel. Therefore, in order to reach at the value of both indirect tensile strength as well as the resilient modulus, it would be required to prepare marshall specimen on which the performance test need to be carried. Different grade of specimen were prepared consists of bituminous concrete grade 1 (BC-1), bituminous concrete grade 2 (BC-2), dense bituminous concrete grade 1 (DBM-1), dense bituminous grade 2 (DBM-2), stone matrix asphalt with nominal maximum aggregate size of 13 mm (SMA NMA 13 mm) and stone matrix asphalt with nominal maximum aggregate size of 19 mm (SMA NMA 19 mm). The binders used for these grade mix includes bitumen of viscosity grade 40 (VG 40), polymer modified bitumen grade 40 (PMB 40) and crumb rubber modified bitumen grade 60 (CRMB 60). The various materials used in the mix design for specimen preparation were tested to check their characterstics to meet the specified limit and standard. The gradation has been fixed as per the MoRT&H specification. Trial and error method is adopted to fix the mix gradation for different grade and to further minimize the effect of gradation, blending is such that for each mix it is near to center line. The marshall specimen were prepared at an obtained optimum bitumen content for different types of mix grade. Indirect tensile strength (ITS) test was conducted on each samples were analysed and further a trial has been conducted to arrive at the modulus value from the ITS data analysis. It has been seen that the obtained modulus from the ITS test data will be nearly same as the resilient modulus value obtained from the laboratory testing using the universal testing machine carried out as per the ASTM D 4123 code.

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CHAPTER 1: INTRODUCTION

This chapter covers the general background related to approach towards design of pavement and material characterstics. Further, the parameters like modulus of resilience and dynamic modulus or complex modulus of hot mix asphalt were discussed as per the codes and guidelines. The objective and scope, necessity as well as the methodology in the form of flow chart of the research project also presented in this chapter.

1.1 Background

In India early civilisation was focused around the Indus Valley where in 1922 archaeologists constructed roads of bricks and properly piped surface water drainage systems. The large stone slabs were used by the Romans for the construction of top layer of the road which was referred to as the PAVIMENTUM, that is pavement in general term for actual road structure. The primary purpose of the pavement structure is to reduce the stress or pressure which get exists due to wheel load to a certain value which the ground under that structure can support. The two main stress condition that can be occuring on the pavement surface are static stress (vehicle at rest exerts direct load on the small area of contact between tyres and the road surface) and dynamic stress (vehicle is moving causing up and down movements due to uneveness of the surface, flurry wind,etc). The effect that is generated due to dynamic stress condition on surface when the vehicle travels along the roads is called as hammering effect.

Flexible pavements are designed on the basis of the mechanistic-empirical design approach as per IRC:37 and the theory selected for the analysis of pavements is “linear elastic layered theory” in which pavement has to be modelled as a multi-layer system. All the layers are assumed to be finite in thickness whereas infinite in the horizontal scale except the subgrade or foundation which is to be considered as a semi-infinite layer. Elastic modulus or resilient modulus of the layer is one of the parameter which is used as an input in IITPAVE software to get the stresses, strains and deflections originated by the applied load at the surface of the pavement.

1.1.1 Characterization of Material for Pavement Design

Flexible pavement comparises of generally two layers i.e. top layer(surface course, binder course) and bottom layer (base course, sub-base course, subgrade). The top layer which is the

asphalt layer behaves like the visco-elastic whereas the bottom layer also called as granular layer exhibits the elastoplastic behaviour. For the design of pavement it is necessary to have certain information regarding the behaviour of the material such as modulus value for the analysis of the pavement structure.

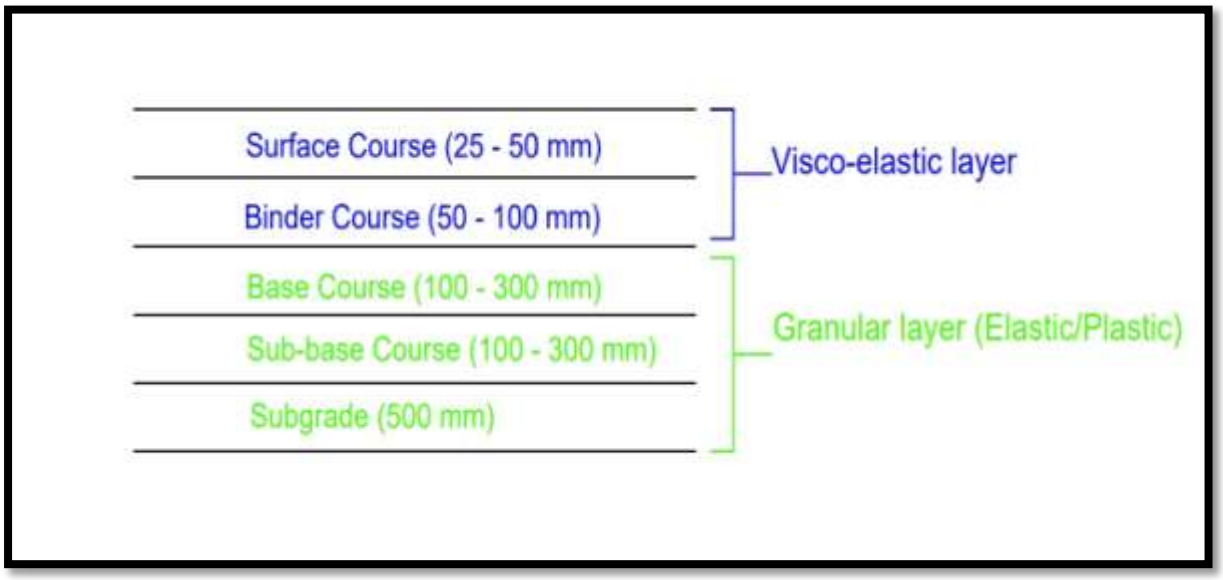

Figure 1.1. Typical cross-section of a asphalt pavement (NPTEL)

1.1.2 Fundamentals of Resilient Modulus and Dynamic Modulus

The resilient modulus is just a single number which is used to express the material behaviour and to anticipate performance of pavement in various pavement design procedures. It is similar to Young's Modulus in the small strain theory of elasticity. Resilient modulus is one of the parameter out of the two requires to fully characterise the linear elastic, rate-independent material. The subsequent parameter is Poisson's ratio. In general terms resilient modulus is a property to measure stiffness of the material in designing thickness of asphalt pavement. It can be determined from a repeated load tests and the concept based on the recoverable part of the strain. Hence, it is defined as the ratio of the repeated deviator stress to the respective recoverable strain.

$$Mr = \frac{\sigma d}{\varepsilon a}$$

Where,

Mr = Resilient Modulus;

σ_d = Repeated deviator stress;

ε_a = Recoverable strain.

The Mechanistic Empirical Pavement Design Guide (MEPDG) uses stiffness of hot mix asphalt for different environmental conditions, traffic speeds, etc in order to calculate strains in pavement which in turn used to calculate the pavement distresses. Therefore, one of the most critical criterion needed for the MEPDG that would be used in flexible pavement is the dynamic modulus.Currently, as per AASHTO TP62-07 (Standard Method of Test for Determining Dynamic Modulus of HMA), the stiffness properties (Dynamic Modulus or Complex Modulus or E*) of HMA can be determined.Compressive Load is applied in the form of continuous haversine wave without rest period over the range of frequencies from 0.1 to 25 Hertz and the stress level upto 2800 kPa. The conditioning of the specimen will be also required over a temperature range from -10 to 60°C to an accuracy of ±0.5°C. The dynamic modulus is calculate as per the equation shown below,

$$|E^*| = \frac{\sigma o}{\varepsilon o}$$

Where,

E* = Dynamic Modulus;

σ_o = applied stress amplitude;

ε_o = measured strain amplitude.

Figure 1.2. Various Testing Machines Used to Measure the |E*| of HMA (Bennert et.al)

The granular layer which should be considered as the bottom layer subjected to substantial amount of loading during their compaction or the construction phase. This lead to cause a lot of permanent deformations or the plastic strains in the material and reaches a steady state. Hence, the continuous loading of the material will have both recoverable as well as the irrecoverable deformations. But after a certain time, a state might comes where the irrecoverable deformation will get cease and there is only recoverable deformations. Therefore, when the pavement is laid compacted and the traffic loading takes place on top layer, the amount of load intensities due to traffic that has been distributed to the bottom granular layers will be substantially less and also, the layer material is preloaded sufficiently. So, only the resilient deformation will occurs in the material.

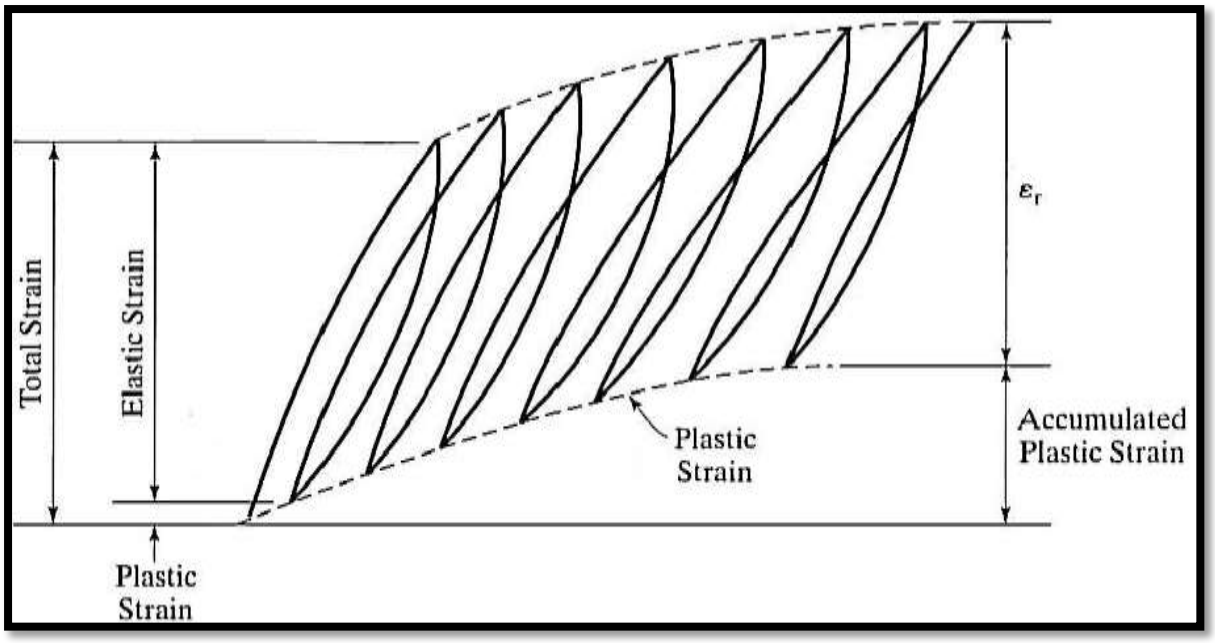

Figure 1.3. strain versus time for the repeated loading cycles (ASTM D 4123)

1.1.3 Design of Pavement Structures

American Association of State Highway and Transportation Officials (AASHTO) design method for pavement structures was developed as a part of AASHTO road test during 1958 to 1960. The strategy has been based upon the collection of significant amount of information from the road data such as roughness, deflection, strains, pavement surface index. So, based upon this, AASHTO design of pavement in 1986 proposed design strategies in which resilient modulus and change in pavement serviceability index was selected for the design of flexible pavement layers and the layer coefficient concept was the fundamental principle for the design. Later in 1993, AASHTO guide for the design of pavement introduced the concept of structural number for the flexible pavement layers which should be adequate to carry the estimated design equivalent standard axle load (ESAL) and based upon it the thickness of the pavement shall be decided. Structural number was calculated using the layer coefficient and the thickness of the layer. Elastic modulus or resilient modulus shall be used for assigning layer coefficient as per the code ASTM D 4123. The latest code developed by AASHTO was in 2002 and revised later in 2008, which would be derived from mechanistic empirical pavement design (MEPDG) approach.

In Indian context as per IRC 37 (1970) code, the design of pavement was introduced based on the CBR of soil subgrade and the same code was modified in year 1984, in which modification of CBR method of pavement design has been established and also minimum thickness for different pavement layers were specified. Again, in year 2001 semi-mechanistic

approach was introduced based on MoRTH's R-56 project for traffic high as 150 msa assuming three layer elastic structure with stresses at critical locations using linear elastic model. The same approach has been again carried out in later revision in year 2012 with additional involvement of the rut resistance mix and fatigue resistant mix for the bituminous surfacing. The latest revision of IRC 37 was introduced in year 2018 with certain modification in the equations for the rutting and fatigue model.

Fatigue Model

$N_f = 1.6064*C*10^{-04}\times[1/\varepsilon_t]^{3.89}*\times[1/M_{Rm}]^{0.854}$ (For 80% reliability)

$N_f = 0.5161*C*10^{-04}\times[1/\varepsilon_t]^{3.89}*\times[1/M_{Rm}]^{0.854}$ (For 90% reliability)

$C = 10^M$, and $M = 4.84\left(\frac{Vbe}{Va+Vbe} - 0.69\right)$

Rutting Model

$N_r = 4.1656*10^{-08}\times[1/\varepsilon_v]^{4.5337}$ (For 80% reliability)

$N_r = 1.4100*10^{-08}\times[1/\varepsilon_v]^{4.5337}$ (For 90% reliability)

Where,

N_f = fatigue life in standard axle load repetitions;

N_r = Rutting life in standard axle load repetitions;

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

ε_v = maximum vertical strain at the top of subgrade layer;

M_{Rm} = Resilient modulus of bituminous mix, Mpa;

Va = per cent volume of air void in the mix used in the bottom bituminous layer;

Vbe = per cent volume of effective bitumen in the mix used in the bottom bituminous layer.

1.1.4 Guidelines and Codes for Resilient Modulus Test

ASTM D4123-82, 1995 (older version) and ASTM D7369 (latest version) are the adoped guidelines and codes for the standard test method for determining the modulus of resilience of asphalt mixtures by indirect tensile strength (ITS). The both codes for test method includes procedures for preparing and testing laboratory-fabricated or field-recovered cores of asphalt

mixtures to determine resilient modulus values using the repeated load ITS test as per D6931-12 code.

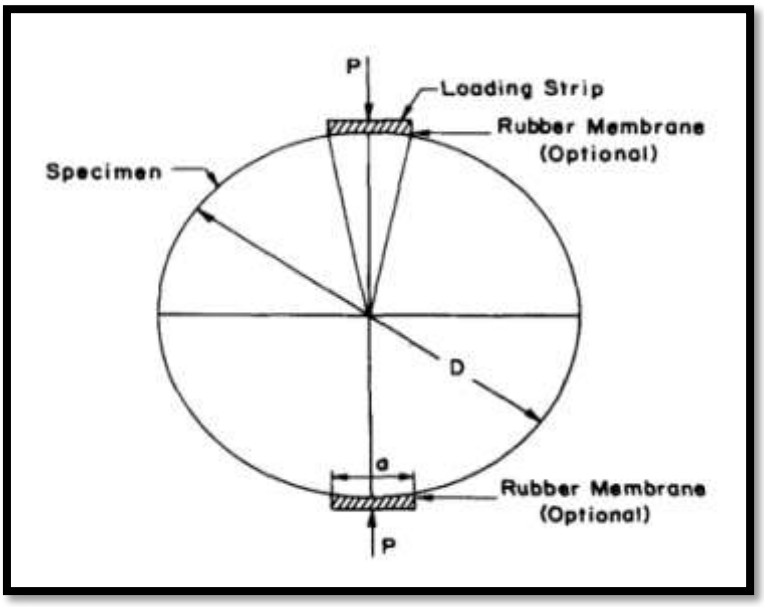

Figure 1.4. Indirect Tension Test (ASTM D6931)

As per ASTM D4123 code, a cylindrical specimen is being subjected to a repeated compressive haversine loading in a diametral direction and the deformation is to be measured. A typical diagram of specimen of diameter 'D' loaded in the diametral axis provided with the loading membrane through which the load 'P' shall be acting is shown in figure 4. The test can be performed on the both smaller size specimen as well as larger size specimen depending upon the nominal maximum aggregate size (NMAS) provided with the appropriate height and the width of the loading strips.

Table 1.1. Required Dimension for the test

Dimension of specimen and loading strips for test		
Diameter of Specimen	102 mm (25 mm NMAS)	152 mm (38 mm NMAS)
Height of Specimen	51 mm	76 mm
Width of loading strips	13 mm	19 mm

The parameters required for the test are mentioned in the table 1.2. The linear variable differential transducers (LVDTs) are required for the test to capture the deformation that should be of full gauge length. The test should be performed on the three specimens with variables of temperature, load, duration and frequency with zero degree and 90 degree position.

Table 1.2. Parameters for testing resilient modulus

Test Parameters	
Preconditioning Cycles	50 to 200 cycles up to resilient deformation is stable
Temperature for testing	5, 25, 40°C
Loading Frequency	0.33,0.5, and 1.0 Hertz
Duration of Load	0.1 to 0.4s
Peak Load	10 to 50% of the tensile strength
Conditioning Time	24 hours if not monitored

The load-time pulse graph is shown in figure 1.5, which demonstrates the haversine shape of the loading denoted by symbol ‘a’, the rest period that is denoted by symbol ‘b’ and the total cycle time which is denoted by symbol ‘c’. This fashion of the repeated haversine loading is to be produced and the corresponding deformations will be measured in the horizontal direction with the help of the LVDTs whereas for the vertical direction there is no need to fix LVDTs becaused the computations are based upon the assumed poisson’s ratio (0.35).

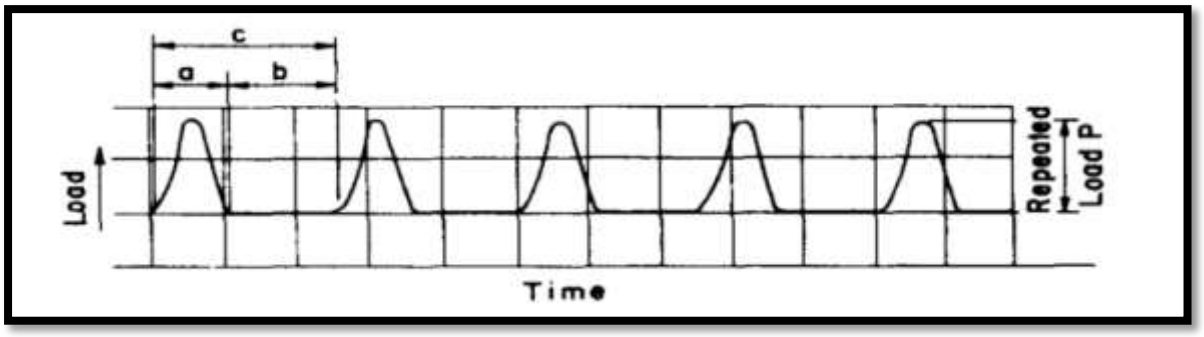

Figure 1.5. Load-Time Pulse (ASTM D4123)

The horizontal deformation-time graph is shown in figure 1.6, which explains that increase in magnitude of loading will lead to cause increase in the deformation and when unloading happens then the deformation will get decreased. Also, there will be recovery of deformation occurs during the rest period. The recovery is of the two types that is the instantaneous recovery and the total recovery. The instantaneous recovery will get to happen when just the unloading starts whereas the total recovery will takes place during the every load cycles. So, using this two recovery or deformation one can able to compute the resilient modulus. The same phenomenon can be seen in the vertical deformation-time graph as shown in figure 1.7.

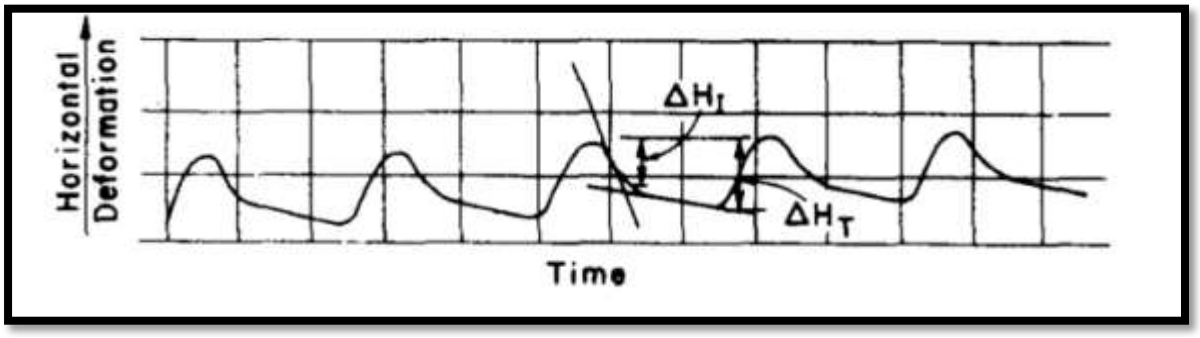

Figure 1.6. Horizontal Deformation verses Time (ASTM D4123)

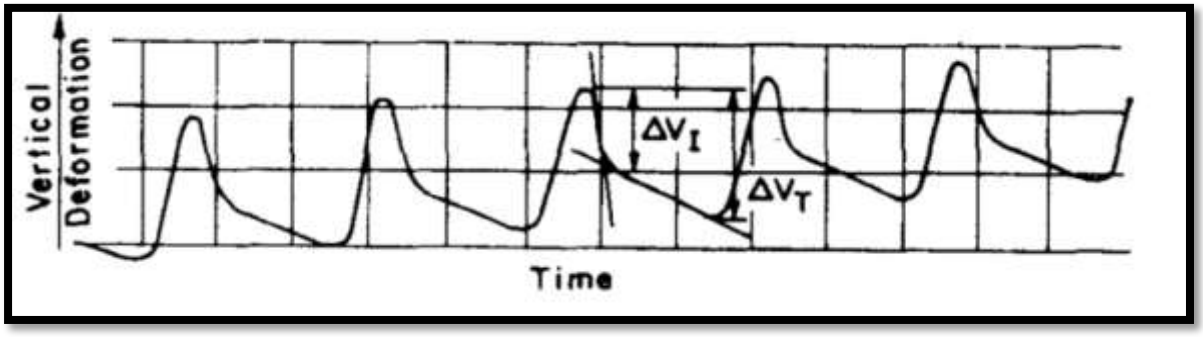

Figure 1.7. Vertical Deformation verses Time (ASTM D4123)

The average value of atleast three loading cycles of the recoverable horizontal and vertical deformations should be measured after the preconditioning cycles that means when the repeated resilient deformation has become stable.

Table 1.3. Resilient Modulus Computation

Evaluation of Modulus of Resilience	
Instantaneous Resilient Poisson's Ratio	$U_{RI} = 3.59(\Delta H_I/\Delta V_I) - 0.27$
Total Resilient Poisson's Ratio	$U_{RT} = 3.59(\Delta H_T/\Delta V_T) - 0.27$
Instantaneous Resilient Modulus	$E_{RI} = P(U_{RI} + 0.27)/t\Delta H_I$
Total Resilient Modulus	$E_{RT} = P(U_{RT} + 0.27)/t\Delta H_T$

Where,
U_{RI} = Instantaneous Resilient Poisson's Ratio;
U_{RT} = Total Resilient Poisson's Ratio;
ΔH_I = Instantaneous Recoverable Horizontal Deformation,mm;
ΔV_I = Instantaneous Recoverable Vertical Deformation,mm;
ΔH_T = Total Recoverable Horizontal Deformation,mm;
ΔV_T = Total Recoverable Vertical Deformation,mm;
E_{RI} = Instantaneous Resilient Modulus of Elasticity,N/mm^2;
E_{RT} = Total Resilient Modulus of Elasticity,N/mm^2;
P = Repeated Load,N;
t = Thickness of the Specimen,mm.

Now, as per the new version of code for resilient modulus test ASTM D7369-20, the same repeated load indirect tension test has been applied but, provided with the fixed load duration and rest period of 0.1 second and 0.9 second respectively. Hence, the total frequency cycle will be of 1 cycle per second or 1 hertz. The contact load should be equal to 4% of the maximum load which might not be less than 22.2 N and not more than 89.0 N whereas the maximum load should be taken as 10-20% of the ITS. The test temperature need to be maintain at 25°C (5°C,15°C and 20°C are optional at one specified loading frequency) with the conditioning time of 6 hours. The compressive haversine load which has to be applied will be form of (1-cosθ)/2.

$$P_{cyclic} = P_{max} - P_{contact}$$

Where,

P_{cyclic} = Applied cyclic load to specimen,(N);

P_{max} = Maximum applied load(N);

$P_{contact}$ = Contact Load,(N).

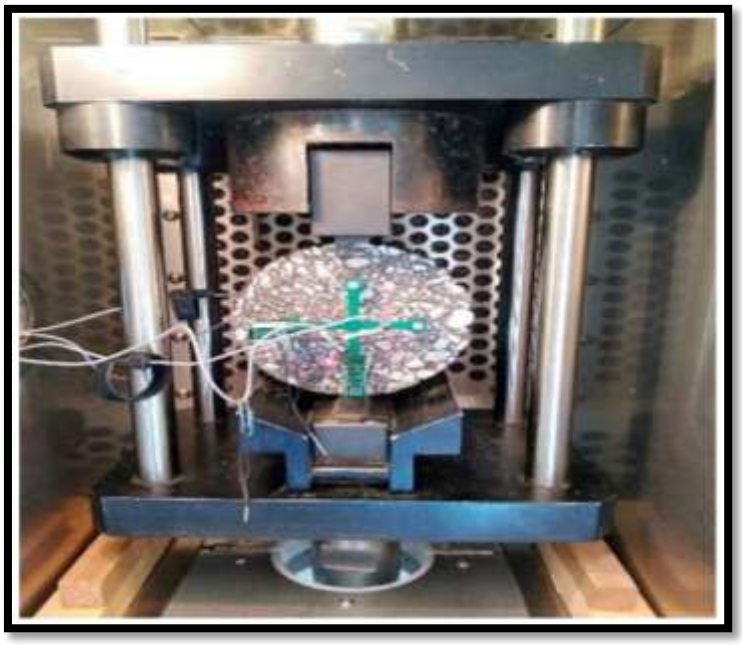

Figure 1.8. Specimen with Loading Strip Parts (ASTM D7369)

The specimen used for the test should be either marshall-compacted or gyratory-compacted or field core having sawed surface with the provided diameter of 101.6 ± 3.8 mm or 152.4 ± 9 mm and the thickness of 38.1 mm or 63.5 mm. This code mandate the fixing of LVDTs on

both the planes in horizontal as well as the vertical directions with the choice of gauge length either quarter, half and full diameter.

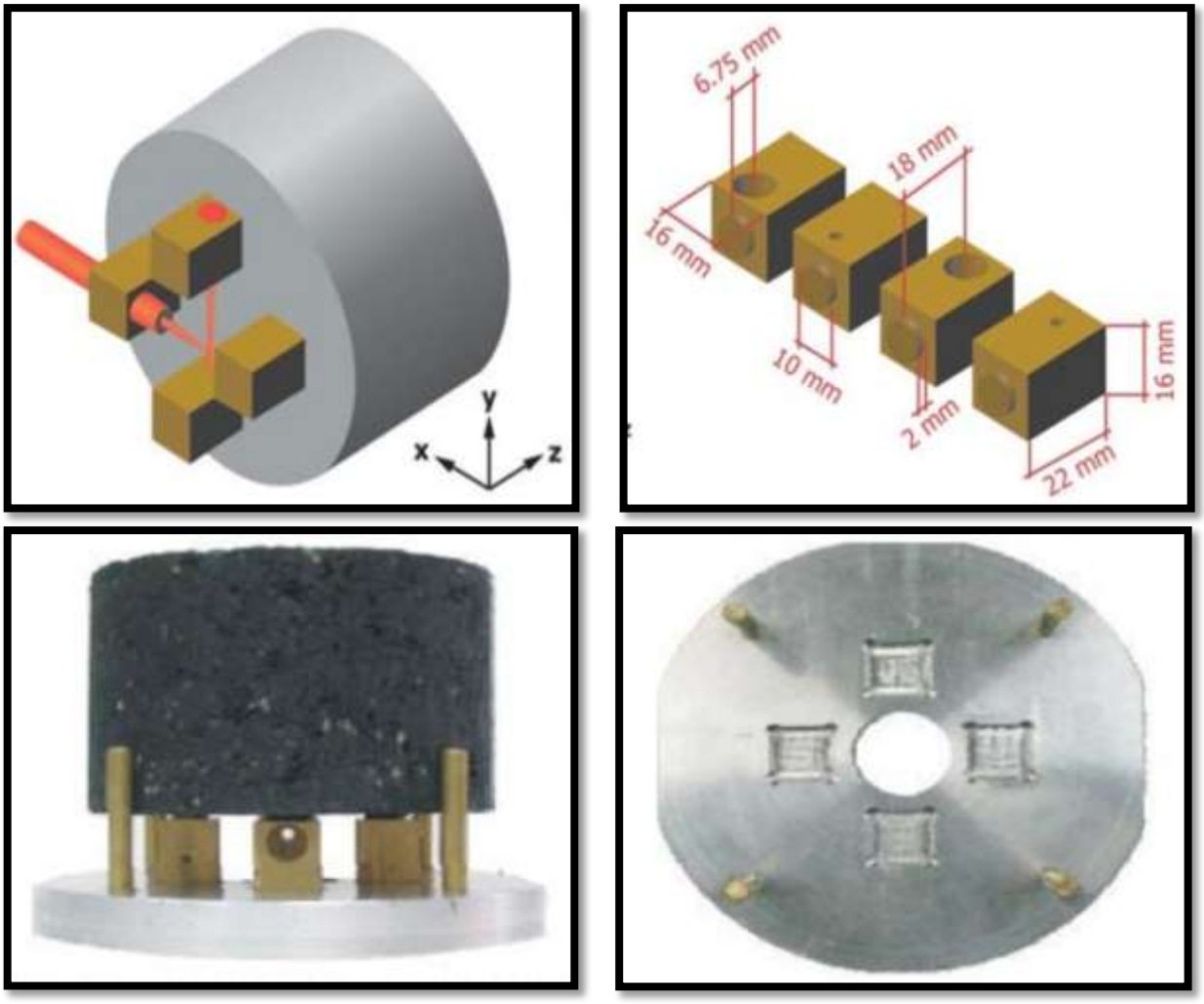

Figure 1.9. LVDT Setup and Arrangement (ASTM D7369)

The steps below explicate the major testing steps as per ASTM D7369-20,

- Indirect tensile strength test as per D6931-12 code need to be performed at the temperature of 25°C in order to select the loading levels (10-20% of ITS).
- After this, testing along first diametral axis should be executed for repeated load cycle provided with 100 cycles of preconditioning and the last 5 for test cycles.
- Similarly, testing along second diametrical axis would be carried out for repeated load cycle for the same cycles of preconditioning and the test cycles.
- It is necessary to look over that the total deformation should be less than 0.025 mm.

POST PROCESSING OF DATA

The graph in figure 1.10 shows the load and deformation curve on the Y-axis with respect to time duration on the X-axis. The load is applied for 0.1s duration provided with rest period of 0.9s. When the strain reaches at a peak point, after that the unloading begins and the point will come from where the recovery of the deformation starts during the rest period. This region of recovery curves can be divided into three portion that would be fit in the form of straight line and hyperbola.

Table 1.4. Curve nature at various region and its equation

Region No.	Nature of curve	Equation
1	Straight line	Y = a + bX
2	Hyperbola	Y = a + (b/X)
3	Hyperbola	Y = a + (b/X)
Where, a and b are the regression coefficient.		

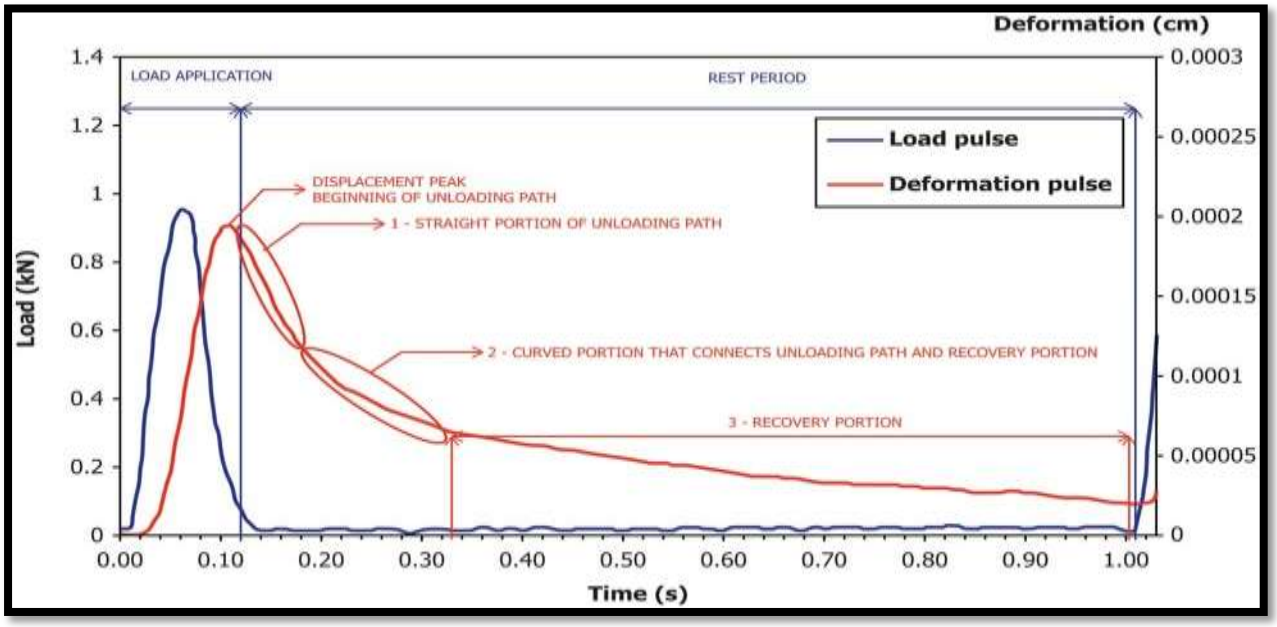

Figure 1.10. Regression in Three Portions of Deformation Curve (ASTM D7369)

After all, main aim is to evaluate the instanstaneous deformation and the total deformation. Based upon this the poisson's ratio and resilient modulus should be determined. So, in order to reach for the same the National Cooperative Highway Research Program (NCHRP) has given the time recommendation for resilient deformation which is shown in the table 4.

Table 1.5. Time Recommendation for Resilient Displacement (ASTM D7369)

NCHRP Research Results Digest January 2004,No.285 Project 1-28A Recommendations		Considering a load pulse of 0.1s and a rest time of 0.9s,the total cycle time for curve estimative are	
$\mathbf{t_c}$	40% of rest period	t_c	0.46s
$\mathbf{t_d}$	90% of rest period	t_d	0.91s
$\mathbf{t_{55}}$	55% of rest period	t_{55}	0.595s
$\mathbf{t_e}$	85% of rest period	t_e	0.865s
$\mathbf{t_f}$	95% of rest period	t_f	0.955s
$\mathbf{T_1}$	Tm + Ta	t_a	0.005
$\mathbf{T_2}$	Tm +Tb	t_b	1/2 $t_{load\ pulse}$ (when $t_{load\ pulse}$ equals to 0.1 s,t_b = 0.05

The graph represented in figure 1.11 and figure 1.12 is used for the determination of instantaneous deformation and the total deformation using the NCHRP data on the time recommendation for resilient displacement. So, in order to identify the point of instantaneous deformation the following procedure need to be followed as per code;

- Drop a tangent from the first portion of straight line and it has to be extended further.
- Again, drop a tangent from the third portion at a time of T_{55}.
- The point where both the tangent will get meet, is the deformation corresponding to point A.
- Substracting this deformation with the maximum deformation will gives the instantaneous deformation.

Now, to identify the total deformation the following steps to be considered,

- Two points towards the end need to be contemplate, first one is corresponding to the 85% of the rest period (t_e) and the other one is corresponding to the 95% of the rest period (t_f).
- The deformation value shall be determined corresponding to the same time and the average of the two value is to be noted down.
- This average value would be substracted from the maximum deformation in order to arrive at the total deformation value.

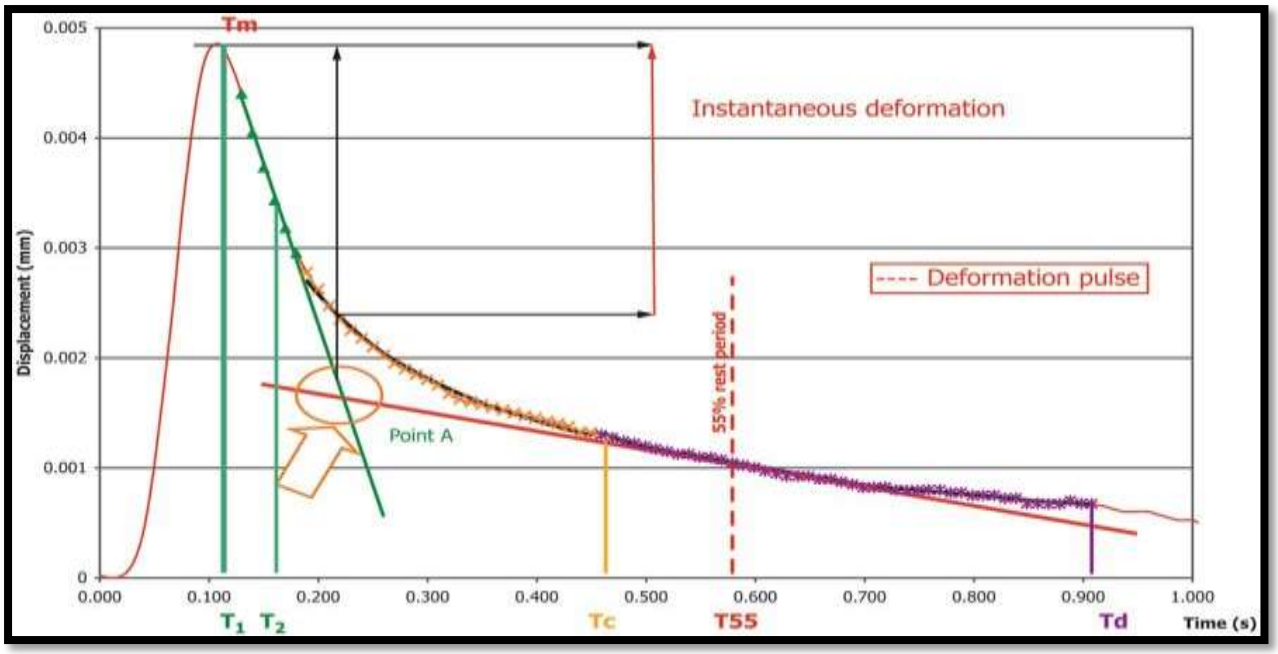

Figure 1.11. Determination of Instantaneous Deformation (ASTM D7369)

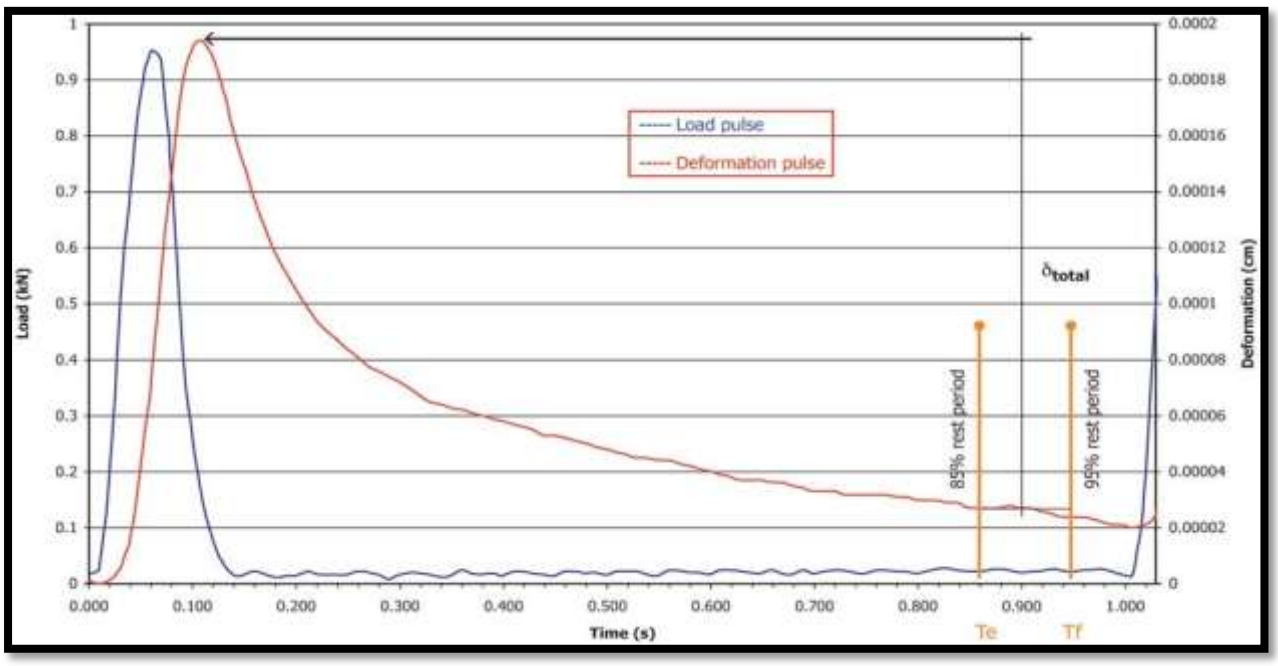

Figure 1.12. Determination of Total Deformation (ASTM D7369)

The poisson's ratio and the resilient modulus value can be determined using the following equation shown below,

$$U = \frac{I4 - I1 \times \left(\frac{\delta v}{\delta vh}\right)}{I3 - I2 \times \left(\frac{\delta v}{\delta vh}\right)}$$

$$Mr = \frac{Pcyclic \times [(I1 \times (I2 \times U))]}{\delta v \times t}$$

Where,

U = instantaneous or total Poisson's ratio;

δv = the recoverable (instantaneous or total) vertical deformation measured over the vertical diameter of the specimen, (mm);

δvh = the recoverable horizontal (instantaneous or total) deformation measured over the horizontal diameter of the specimen, (mm);

Mr = instantaneous or total resilient modulus of elasticity, (MPa);

t = thickness of specimen, (mm);

Pcyclic = cyclic load applied to specimen, (N).

Table 1.6. Constant Values for Resilient Modulus and Poisson's Ratio Calculation (ASTM D7369)

Gauge Length as a Fraction of Diameter Specimen	I1	I2	I3	I4
0.25	0.144357	-0.450802	0.155789	-0.488592
0.50	0.233936	-0.780056	0.307445	-1.069463
0.75	0.265925	-0.952670	0.430875	-1.934486
1.000	0.269895	-1.000000	-0.062745	-3.587913

1.2 Scope and Objectives

In the design of flexible pavement, one parameter that is required in the software like IITPave, KGPBACK, etc as an input is the resilient modulus of different layers. This resilient modulus value one can get by testing the cylindrical specimen of laboratory-fabricated or field-recovered cores of asphalt mixtures in the laboratory by using the Universal Testing Machine(UTM). It is tedious and time taking task required with skilled and experience technical personel, if we use this type of technique to determine the resilient modulus. Therefore, it is necessary to develop a relationship by correlating the resilient modulus with the Indirect tensile strength (ITS).

The following are the objectives of the study mentioned below,

- To perform the basic tests on bituminous binders and aggregates.
- To conduct the performance test on hot mix asphalt (HMA) specimen casted based on different grade and binder types.
- To form the empircal relationship between the resilient modulus and Indirect tensile strength.

1.3 Necessity

The development of empirical relationship between the resilient modulus and indirect tensile strength will make the job easy by saving the time and make the work to happen fast because it is not required to perform the resilient modulus test which is a time consuming test. Also, this type of relationship is free from personel or system error, which is likely to cause while performing the laboratory testing.

1.4 Methodology Flow Chart

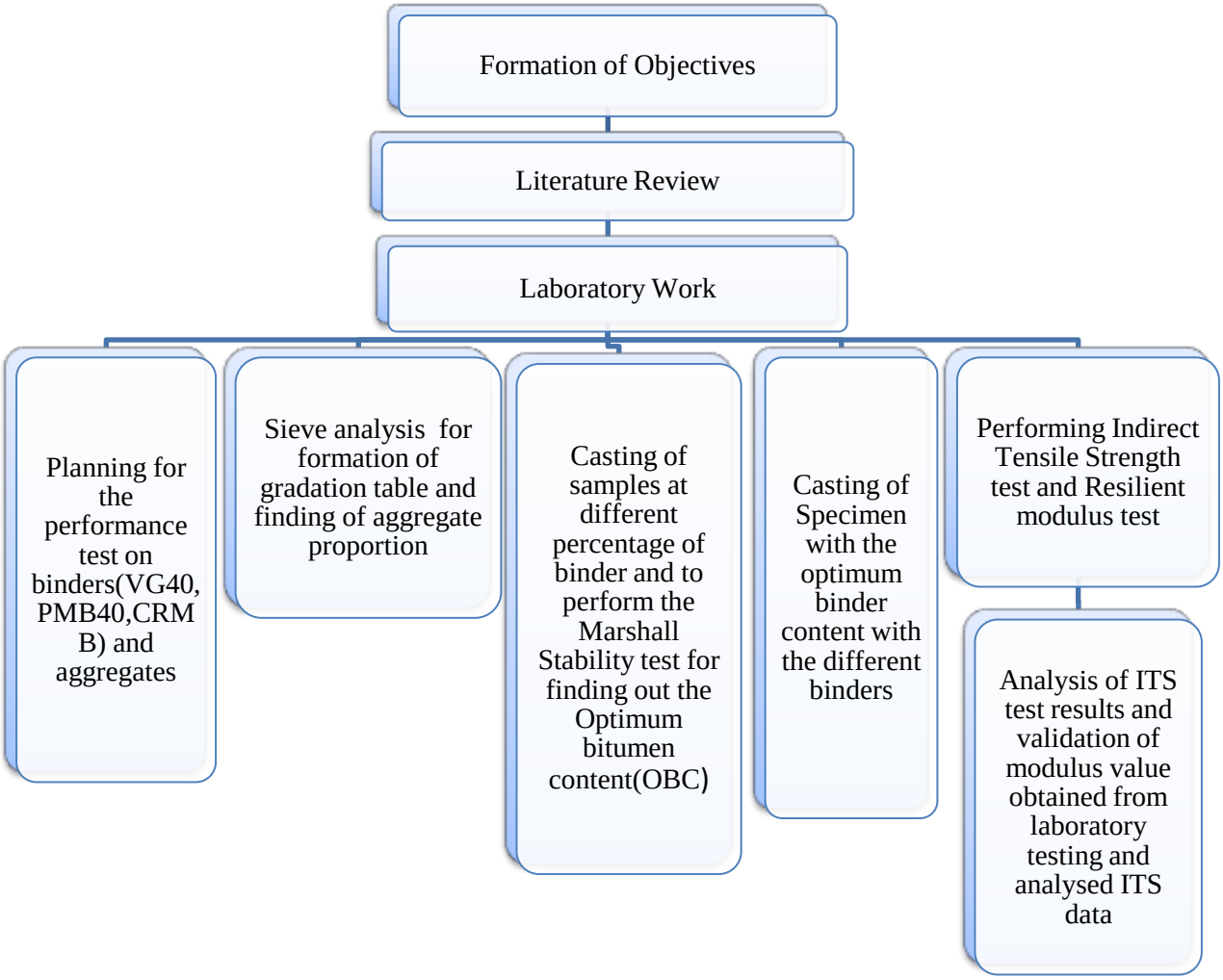

Figure 1.13. Methodology Chart

CHAPTER 2: LITERATURE REVIEW

2.1 Bitumen and its available types in India

Bitumen is a viscous material either in the form of liquid or a solid consisting of hydrocarbons and their derivatives which is manufactured from a fractional distillation of crude oil obtained as the last residue (HP Handbook). Characterisation of bitumen are done according to the obtained chromatographic fractions obtained i.e, maltenes (saturates, aromatics and resins) and the asphaltenes often soluble and insoluble in n-heptane respectively (Rossi et.al,2015). The following are the types of bituminous binder available in india,

- Penetration Grade
- Industrial grade bitumen
- Cutback
- Bitumen Emulsion
- Modified Bitumen
- Viscosity grade Bitumen

2.1.1. Comparing Viscosity Grade 40 (VG 40) Bitumen, Polymer modified bitumen (PMB) grade 40 and Crumb rubber modified bitumen grade 60(CRMB60).

Viscosity grade 40 bitumen is a type of bitumen having the high viscosity and produces stiffer bituminous mixes to resist against the high temperature and heavy traffic loading. (Iran Bitumen Viscosity Grade). Due to high viscosity in the high temperature range, its rutting resistance will get increased (Kumbargeri et.al,2014). Maximum stability of the mix can be achieved by using VG40 grade bitumen (Singh et.al,2015). This grade of bitumen is suitable for seven day average maximum air temperature greater than 45°C (IS 73).

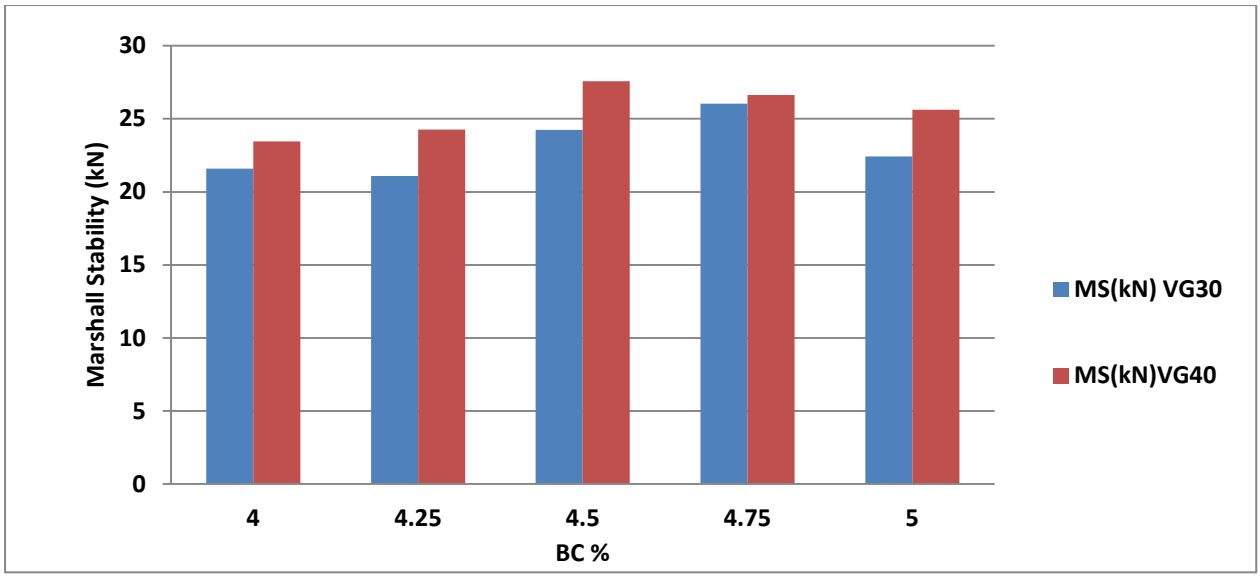

Figure 2.1. Marshall Stability vs Bitumen Content (Gupta et.al,2017)

Polymer modified bitumen is a type of bitumen prepared by blending suitable penetration grade (e.g.60/70) bitumen reconsilable with additives (e.g.styrene butadiene styrene) at refineries plants having suitable mixing and blending facilities (Patel et.al,2015). PMB 40 will show its less sensitivity towards the temperature that means high temperature during summer season and low temperature during winter season does not have an impact towards the flow and the thermal cracking repectively (ATDM). It acquires high cohesiveness , high stiffness modulus which provides the greater resistance to permanent deformation and improves the bonding within the bituminous layers (Bala et.al,2014). Although, this grade of bitumen shall be generally recommended for hot climate areas.

Table 2.1. Significance of using PMB40 (Pareek et.al,2012)

Properties	**Significance**
Elastic Recovery	High elasic recovery(79%)
Age Resistance	Better age resistance
Marshall Stability	Increases by 27%
Rutting Resistance	Significantly higher
Indirect tensile Strength	Increases by 20%
Resilient Modulus	Higher test temperature of 30 to 40°C

CRMB 60 is a bitumen based on softening point modified with crumb rubber powder from discarded truck tires (Devendra et.al,2014). It is manufactured using tyre rubber (10-20%) by weight of bitumen at the plant (Ambika et.al,2013). Morsel elastic is affix to hot bitumen and the mixture should be mechanically stirred till the occurance of response between the

bitumen and scrap elastic (Rohit et.al,2015). The use of this type of modified bitumen increases the resilience at high temeprature, improves the fatigue or reflection cracking and saves the energy and natural resources (Vora et.al,2013).

2.1.2 Application and Advantages of VG40,PMB40 and CRMB60

The application of VG 40 grade binder are mentioned below (Patekar et.al,2014).

- The zone reserved with high stress concentrations (intersections of roads ,truck parking zone,heavy traffic).
- Area susceptible to higher temperature.

The resistance against rutting and fatigue cracking will get improved (Magdi et.al.2013). Hence,this grade of bitumen may takes care of high as well as low temperature (Patekar et.al,2014).

PMB40 is ideally suited for hot climates with the following application (Hindcol):

- Surface Dressing
- Wearing Course
- Crack sealing
- Stress absorbing membrane interlayer
- Base course
- Thin overlays

Qualities exhibited by PMB40 are listed below (RAHA Group)

- Higher rigidity
- Increased resistance to deformation
- Increased resistance to cracks and stripping
- Better water resistance properties
- High durability

CRMB 60 can be used for high traffic roads, busy intersections, bridges deck or roundabout as a wearing course and base course. The applications are in reducing traffic noise and high rainfall areas (MRPL).

Under increased road temperature, resistance to deformation will get increased which improves the smoothness to drive. Adhesion property and the bonding nature with aggregates may get improved. Softening point , flow resistance and impact resistance will be increased.

Reduction in thermal sensitivity will lead to avoids the causing of cracks under stress. Reduction in degree of rutting shall imroves the driving comforts (Suganpriya et.al,2016).

2.2 Hot mix asphalt and its production

Hot mix asphalt is a mixture of aggregates and asphalt that requires heating before placing for the purpose of large scale paving applications like roads, parking lots, driveways (Colorado Pavement Solution). It should be produced at temperature ranges between 140°C to 160°C and even more than that for the mixtures like asphalt rubber and polymer modified asphalt (Yangjoo et.al,2012). Hot mix asphalt technology is very conventional method for the pavement construction and could be structurally satisfying the performance requirements over many years (Meghna et.al,2018).

Asphalt mixing plants (AMP) is used for the production of hot mix asphalt based on the traditional technology that is of batch type. Batch plant is a facility for manufacturing and production of bituminous paving mixtures by proportioning the aggregate and bituminous materials into the mix by weighed batches and add bituminous constituents either by weight or volume, then finally mixes the blend. (Henrikas et.al,2009).

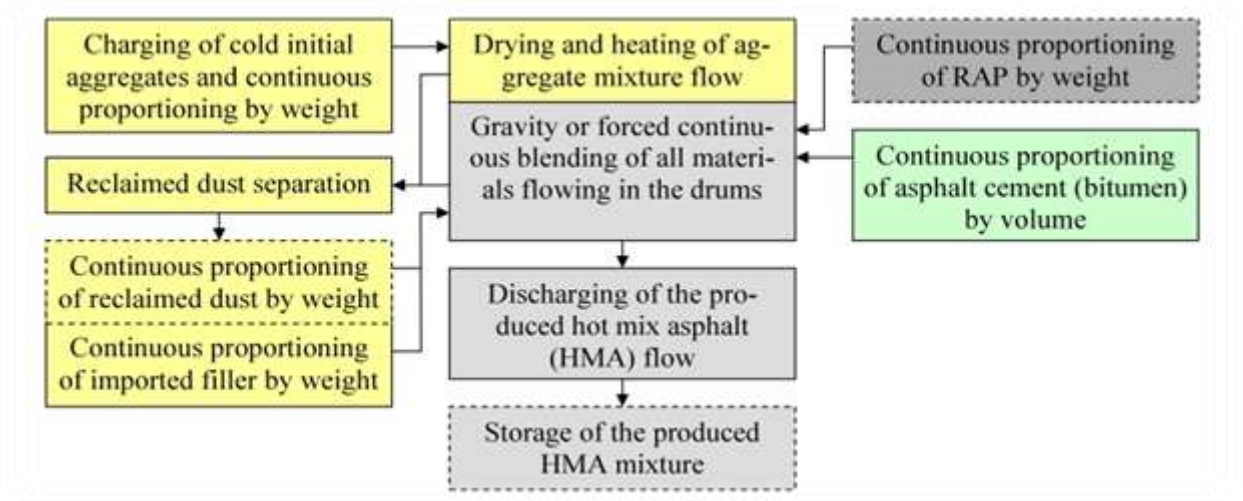

Figure 14. HMA Production Steps (Henrikas et.al,2009)

2.3 Types of aggregates,fillers and fibres used in HMA

Bituminous mixes shall be prepared using the different types aggregates having filler and fibres in it depending upon the requirement.

2.3.1 Mineral Aggregate

A wide variety of mineral aggregates are used to produce asphalt mixtures that are natural aggregates (simply mined and no further processing), synthetic aggregates (industrial byproducts) and processed aggregates**.**

Processed aggregate is a type of aggregate that has been mined, quarried, crushed, separated out in distinct size fractions (MS-2). In processing step of processed aggregates, the two main steps in crushing operation are to feed it first into a jaw crusher and then further to a cone crusher in order to get the reduced sizes (Bessa et.al,2014). The shape (angularity) and texture (roughness) are the two significant chracteristics of the aggregates which affect the mixes performanace in terms of strength and rut resistance (Ahlrich et.al,1996). The particle shape and surface texture exhibit the high dependency on the interlock-internal friction (Shakir et.al,2021).

2.3.2 Mineral filler

The materials which get passed through 0.075 mm sieve are known as fillers, plays a prominent role in charcteristics of bituminous mixture majorly with respect to the air voids and voids in mineral aggregates (Ajay et.al,2019). Minerals filler composed of finely divided mineral substance such as rock dust, slag dust, hydrated lime, hydraulic cement, fly ash, loess and other suitable minerals materials (Kifile et.al,2020).

2.3.3 Stone dust

Stone dust is recommeneded as the normally used filler to modify the performance and quality of conventional asphalt mixes (Azaz et.al,2020). It is also known as quarry dust, used as a filler materials in bituminous mixes without influening its strength characteristics (Praveen et.al,2018). It significantly fills the voids in the mixes which makes the pavement suitable with lesser number of voids (Manzoor et.al,2019).

2.3.4 Lime Powder

Lime is found generally in a powder form which is derived from the crushed rocks (Cesare et.al,2019). It should be in dry state and free from any lumps or aggregations of fine particles (Ismael et.al,2019). The use of lime in HMA mixes will helps in the reduction of moisture damage and striping action of aggregates that are sensitive to moisture (Kollaros et.al,2017). Also, improvement in the fatigue performance and resistance to permanent deformation with increase in resilient modulus can be seen by utilization of lime as a filler material (Al-Tameemi et.al,2015).

2.3.5 Fibers

Fibers are defined as a natural or synthetic materials, used in the form of reinforcement in hot mix asphalt to provide strength, thermal stability and composite frictional properties (Carlos et.al,2019). The incorporation of fibers into a hot mix asphalt mixes serves a three-dimentional secondary reinforcement due to the adhesive nature of fibers with asphalt binders

and interlocking ability with aggregates (Alfalah et.al,2020). The various type of fibers that are used in HMA mixes are mineral, polyester, polyacrylonitrile (PAN), carbon, glass, steel, aramid, coconut, etc.

2.3.5.1 Cellulose Fiber

Cellulose is generally an organic compound processed after removing impurities resulting in cellulosic material with availability of 75 to 80 percent of cellulose with pH measuring between 6.5 to 8.5 (Davidson et.al,1994). Incorporation of these fibres in hot mix asphalt will absorb the bitumen if in higher dose and significantly improves the bleeding of binder and plastic deformations (Juan et.al,2021). Addition of 0.3% of fiber is commonly preferred in HMA mixes (Alheeti et.al,2013).

Figure 2.3. Cellulose Fibres Pellets (Irfan et.al,2019)

2.4 Bituminous pavement layers and their inter-layer bonding

The bituminous pavement layer consists of bituminous base course, bituminous binder course and asphaltic bituminous concrete. Bituminous base course is a mixture of mineral aggregate (stone, gravel and sand) together bonded with a bituminous materials which are generally used as a foundation on which binder and surface course shall be placed. In bituminous binder course an bituminous-aggregate mixture will be used as an intermediate course between the base and surface courses is defined as a first bituminous layer in a resurfacing of two layer bituminous mixes. Asphaltic bituminous concrete is a mixture of aggregates (maximum size of 25 mm and filler smaller than 0.075 mm) with sufficient bitumen and concrete materials to make the compacted mix effectively impervious to provide acceptable dissipative and elastic properties (Shahid et.al,2019).

Bituminous concrete (BC) layer is a mixture of aggregate (acts as a structural skeleton) and binder (acts as a glue) (Magadi et.al,2014). It is used in wearing and profile corrective courses, constructed in a single layer on a previously prepared bituminous bound layer with provided thickness of 30mm/40mm/50mm. Dense bituminous concrete (DBM) layer is also a

mixture aggregate and binder used in base or binder and profile corrective courses in a single or multiple layers on a previously prepared base or sub-base with adopted thickness of 50 mm to 100 mm. Stone matrix asphalt (SMA) is a fibre stabilized aggregate-binder mixture constructed either in single or multiple layer as wearing or binder course having a nominal layer thickness of 40 to 50 mm for 13 mm SMA and 45 to 75 mm for 19 mm SMA (MoRT&H).

It is required to ensure that the proper bonding between the adjacent layers would be offered to prevent against the acting shear force mainly responsible for the interface bond failure of the paving layers (Hooda et.al,2017). This inter-layer bond strength can be improved by using the tack coat of low viscosity grade such as emulsion (rapid setting or medium setting) or VG10 grade bitumen (Giri et.al,2018).

2.5 Brief description on Indirect Tensile Strength (ITS)

The indirect tensile strength of bituminous mixes (marshall specimen) is executed by loading it along vertical diametrical plane at a deformation rate of 50mm/min and temperature of 25°C with the help of two opposite loading strips (Tiwari et.al,2018). The loading system will produce uniform tensile stress perpendicular to the direction of the load applied and across the vertical diametral plane will lead to cause failure of the specimen (Lokesh et.al,2018). The peak load at fracture should be measured (Jens et.al,2010). The theory of elasticity would be used to obtain the tensile strength without considering the effect of multiaxial state of stress (Kamilla et.al,2012). Normally, ITS simulate the damaging effect (fatigue cracks and rutting) due to the traffic load for a long period of time (Rosa et.al,2019).

2.6 Basic understanding of Resilient Modulus(Mr)

Resilient modulus (Mr) is the elastic modulus based upon the recoverable strain under repeated external shocks or disturbance and is defined as the deviater stress to the recoverable elastic strain (Anjan et.al,2008). ASTM D4123 code is used to determine the resilient modulus of the laboratory-fabricated or field-recovered cores of bituminous mixtures with the application of repeated load indirect tension test. “Indirect” word is used because the tensile stress produced to determine the material properties is not directly acting tensile force, but it is due to the application of compressive force acting along the vertical direction,tensile stresses will be producing in the horizontal direction along the vertical direction (Mirza et.al).

Brazilian test was the first test which relates the resilient modulus test, but due the limitation of cracks initiation away from the center of disc, the approach callled flattened Brazilian test

was introduced in which, two parallal flat ends are introduced to in order to improve the stress distribution (Shunchuan et.al,2018).

Resilient Modulus represents the elastic stiffness of the material after the many load repetions due to which it becomes the more appropriate for use in multilayer elastic programs (Mamlouk et.al,1988). Rational understanding can be made on the basis of principle of strength of material of the moduli which are generally used for the structural evaluation of the asphalt mixtures. Also, moduli which are anticipated from the dynamic nondestructive testing of pavements are most indicative of the insitu resilient moduli of the materials.

Surface response methodology (RSM) was used to study the influence of pretreatment time, test temperature, binder percentage, recycled aggregate (construction and demolition) percentage on the value of the resilient modulus (Juan et.al,2020). It has been found out from the study that the temperature is considered as the most influencial parameter because due to increase in temperature, the rigidity of the mixtures get reduced and this lead to decrease the resilient modulus value. Pretreatment time shows the positive influence on the resilient modulus at the high temperatures. Binder percentage should have a little effect on the resilient modulus value. Also the use of recycled aggregates does not shows any effect on the resilient modulus value.

2.6.1 Various Pararmeters affecting Resilient Modulus

Loading Magnitude Effect

Resilient Modulus of asphaltic mixtures depends upon the percentage of load used in indirect tensile strength test (Jamal et.al,1991). Larger is the load, smaller will be the value of resilient modulus and this results in a more conservative design.

Loading features Effect

(Gholamali et.al,2014) used an Adaptive Neuro-fuzzy Inference system (ANIFIS) model is to investigate the effects of loading features on resilient modulus. For each waveform (square and haversine) a model is developed using the temperature (°C), loading time (ms) and R/L as an input layer whereas the resilient modulus of asphalt mixes as an output layer. It was concluded that,

1. the resilient modulus under haversine loading are more susceptive then the square loading in terms of loading time variation but at the same time if we compare the resilient modulus values under square loading and haversine loading, it was seen that the resilient modulus

values under square loading is less than the resilient modulus values under haversine loading for all the cases. So, it is prefer to measure the resilient modulus under square loading.

2. the Mr variation due to change of R/L from 9 to 30 was insignificant and it is recommended that to use R/L ratio equal to 9 as a default but for heavy traffic the same will be taken as 4.

Square loading, haversine loading and bell-shaped loading represents the distribution of vertical compressive stress pulse at various asphalt pavement depths (Li et.al,2014). A generalised logistic sigmoidal function is used to study the behaviour of dynamic modulus and resilient modulus under each type of loading waveform. It was recommended to measure the dynamic and resilient modulus under the bell-shaped loading. It is because of the overestimation of the true modulus of HMA material under haversine loading which indicates underestimation of the tensile strain at the bottom of the asphalt layer in case of dynamic modulus whereas, the overestimation of rutting resistance under the haversine loading due to accumulated lower plastic strain in case of resilient modulus.

Effect of frequency of loading cycle

Increasing the frequency of loading will increase the resilient modulus (Fairhurst et.al,1990). This is because of small recovery of strain due to application of faster loading rate and less time period for resting.

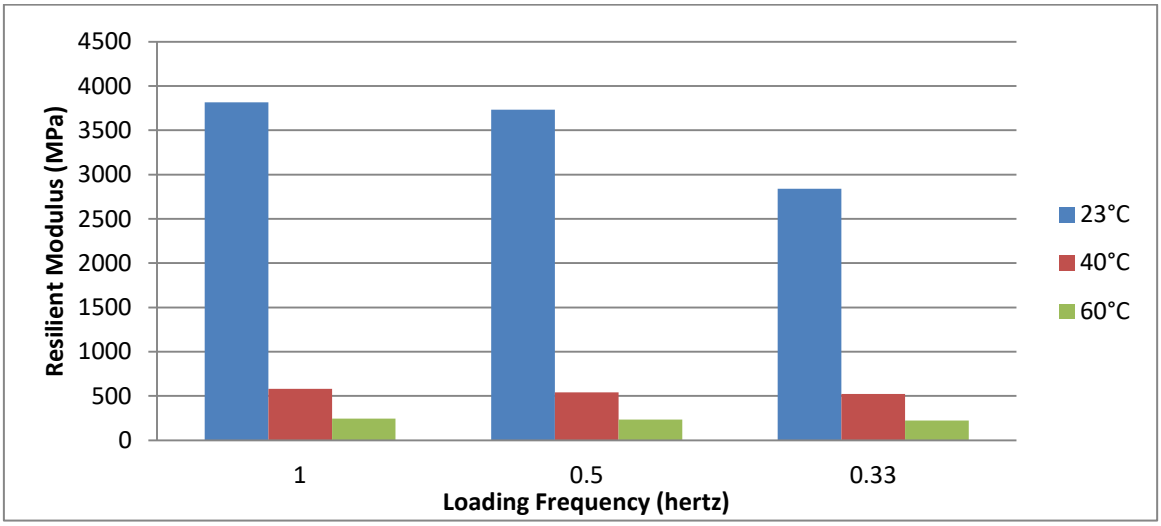

Figure 2.4. Resilient Modulus vs Loading Frequency (Ahmad et.al,2003)

Effect of time duration

Longer is the load duration, it will produce higher strain, thus reducing the resilient modulus (Jahromi et.al,2009). At low temperature, effect of loading time will not have that much effect as compare to resilient modulus of asphalt mixtures at higher temperature (Ali et.al,2013). This is due to elastic behaviour of the mixes at lower temperature but same at higher temperature resilient modulus will get decrease due to increase in loading time. The reason is that at higher temperature, the deformation will be more because of larger loading time and this will lead to get less recovery of deformation.

Effect of rest period

Larger will be the magnitude of rest period, the asphalt mixtures will get more time to get recover the strain and the resilient modulus value will results in decrease (Ali et.al,2013).

Effect of Temperature

Two mixes named as Bituminous B (Bit B, well graded, 50% crushed aggregates) and Bituminous C (Bit C, wider gradation range, no crush count restriction) are used for the analysis of the laboratory resilient modulus at temperature 10°C and 25°C cored from the eight project site. Each project site laid with two side by side section consisting of Bit B over Bit B and Bit B over Bit C respectively. Project site designated with ID B contains 70% RAP materials whereas the others site contains only 30% RAP material. Hence, increasing percentage of RAP material in mixes will tend to increase the stiffness of the mixes same like in the case of project site B which may also make it more brittle at low temperature conditions (Ahmed et.al,2004).

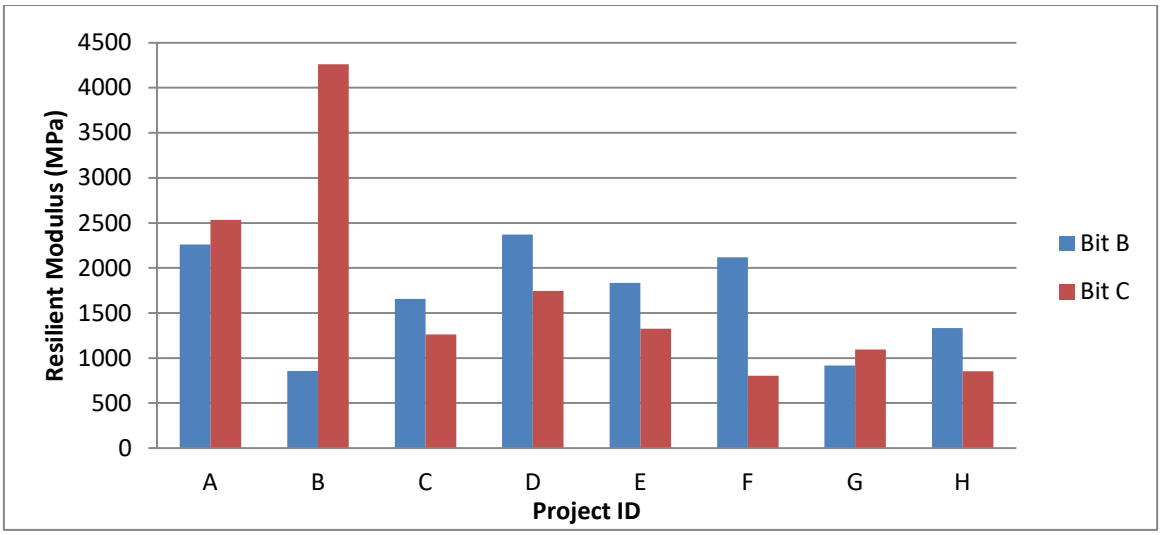

Figure 2.5. Effect of temperature on resilience modulus at 10°C (Ahmed et.al,2004)

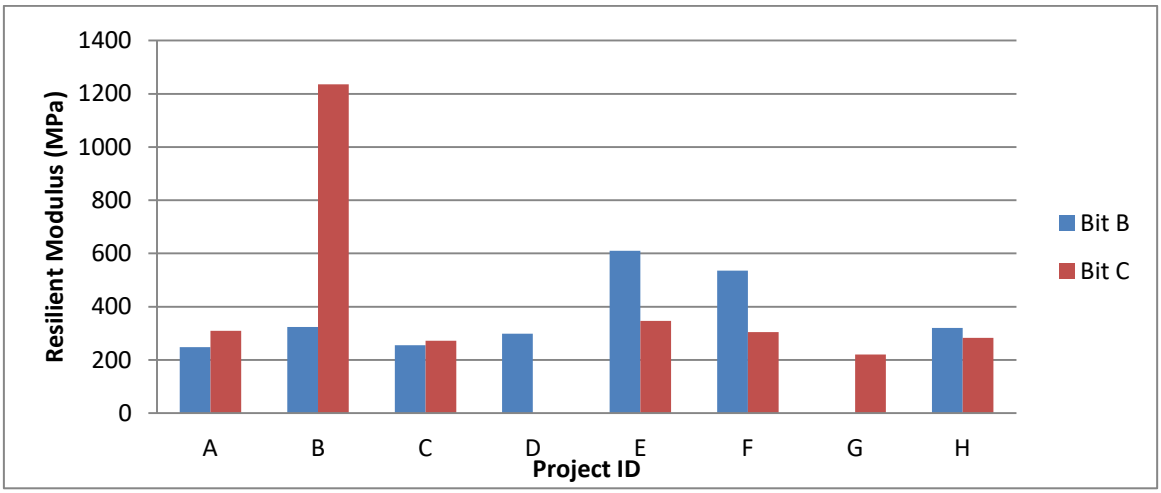

Figure 2.6. Effect of temperature on resilience modulus at 25°C (Ahmed et.al,2004)

Three different type of asphalt mixtures are used to compare the effect of temperature on aged sample and the corresponding resilient modulus. It is clear that with increase in tempetaure from 25°C to 40°C, the resilient modulus is decreasing (Idham et.al,2013). This is because at higher temperature the viscosity will get reduced and this directly lead to increase flow ability and and the strain and vice versa in case of lower temperature.

Higher Mr value may get develop due to the drop in temperature because of evaporation in the wet specimens if tested wet (Jamal et.al,1991).

Effect of Volumetric Properties

Asphalt mixes will be designed in such a way, it can achive a high level of compaction and density (Jamal et.al). Presence of more air void ratio will tend to decrease the resilient modulus as well as the density.

Effect of binder content

Resilient modulus of the specimen will first increase as the binder content percentage increase upto a maximum and then further increase in BC% will shows decrement in thr Mr value (Meor et.al,2008)**.** Resilient modulus of hot mix asphalt mixtures includes 100% steel slag as an aggregates with different percentage binder content has been shown below.

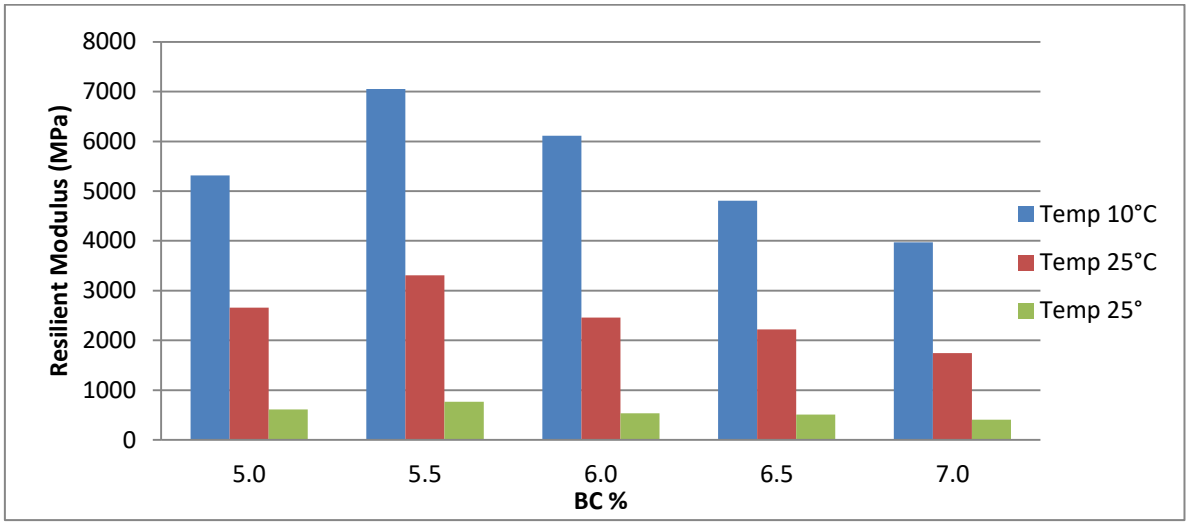

Figure 2.7. Effect of binder on resilient modulus (Meor et.al,2008)

Effect of dimensions of specimen

Specimen diameter of 100 and 150 mm are used in testing the resilient modulus. It was found that the specimen with smaller diameter showed greater resilient modulus than the larger diameter specimen (Barksdale et.al,1997). Specimen having smaller diameter and less thickness relent higher resilient modulus due to the higher confinement of the aggregate particles in the mixes as compare to larger diameter and more thicker specimen (Jahromi et.al,2009).

Effect of variability in test

As per ASTM D4123, there are three sources of variation that is experimental error, orientaion variation and sample variation. If these variation is combined then it will produce a variation in the resilient modulus. Sample variation is one of the prime factor that affect most

in the variation of resilient modulus among the three (Brown et.al,1991). Resilient Modulus values is significantly effected by the stress applied on the sample during the testing. It is suggested that the standard stress value should be 15% of the tesnile strength of the asphalt mixes. The relationship is developed to convert the other stresses to the standard stress using the regression equations tested at 25°C for field and laboratory mixes.

Table 2.2. Developed regression equation for conversion(Brown et.al,1991)

Mix Type/Temperature	25°C	40°C
Field	Y = -0.025X + 1.372	Y = -0.04226+1.668
Laboratory	Y = -0.0225X + 1.34	-
Combined	Y = -0.0238X + 1.36	-
Where, Y = Resilient Modulus at X% upon the resilient modulus at 15%. X = Stress as % of tensile strength.		

Effect of aggregate morphology

Resilient modulus is mostly effected by the aggregate gradation then, aggregate shape and finally aggregate surface texture (Neham et.al,2002). Also, cubical aggregate shape with a rough surface texture shows a high resilient modulus because of the friction and high levels of interlocks between the aggregates. Resilient Modulus of asphalt mixtures will get substantially improved by utilising coarse aggregate with more irregular morphologies by performing test at a temperature of 25°C. Reducing maximum aggregate size from 19 mm to 9.5 mm shows an increasing positive influence of aggregate morphologies on the resilient moduli of asphlat mixes (Pan et.al,2005). Asphalt mixes with coarser aggregate gradation, shows the high resilient modulus due to good particle to particle contact between the larger aggregates (Jahromi et.al).

Effect of Compaction

The two method of compaction that is generally used are marshall compaction and gyratory compaction, out of which marshall compaction imparts the higher resilient modulus for smaller diameter specimen whereas for larger diameter specimen,there is no such effect of compaction method (Saleh et.al,2006).

Effect of Aging

Aged-hardening takes place due to oxidation of asphalt and tend to increase the stiffness of the hot mix asphalt mixtures. This results in high value of resilient modulus (Idham et.al,2013).

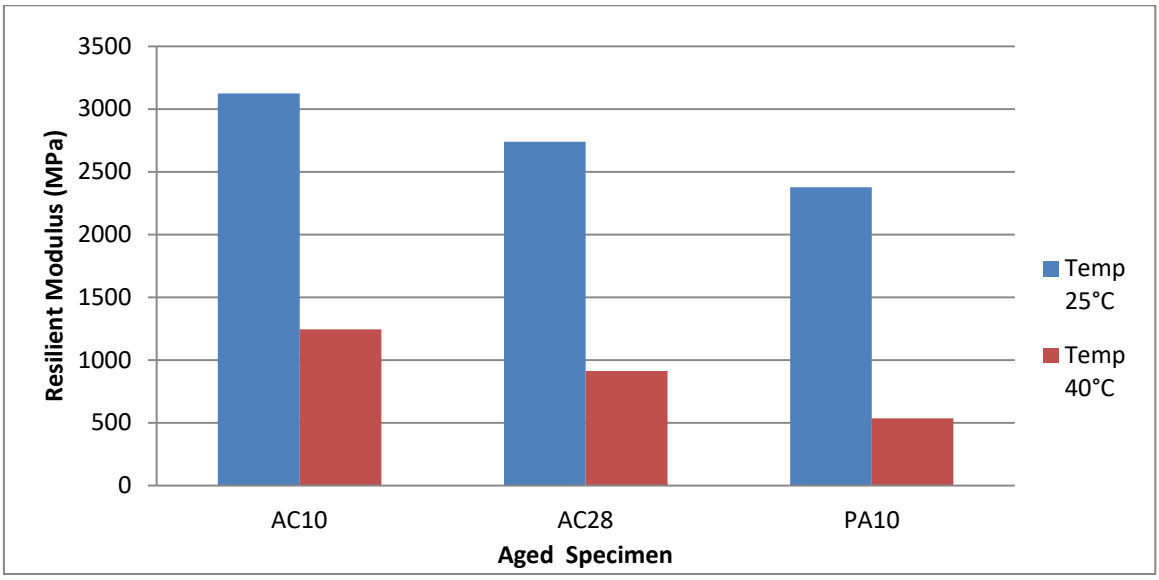

Figure 2.8. Effect of Aging on resilient modulus (Idham et.al,2013)

Effect of Gauge Length

Three types of gauge length are used for the measurement of resilient modulus, these are full gauge length, half gauge length and quarter gauge length. Quarter gauge length will give higher value of resilient modulus for the unmodified binder as compare to modified binder, whereas there is no such difference can be seen if the half gauge length is used (Arbin et.al).

Effect of instantaneous and total deformation

Deformation which get recovered at the end of the unloading cycle is known as the instantaneous deformation whereas deformation which may get recovered at the end of rest period is called as the total deformation. On comparing both deformation and corresponding resilient modulus, it has been found out that the total resilient modulus will be more reliable measures as compare to instantaneous resilient modulus (Boudreau et al. 1992).

Effect of pore pressure

Resilient Modulus test is dynamic in nature and produce short-duration pulsating loads. Due to which the water present in the moisture-conditioned specimens will evolve the pore pressure and results in the increase in the Mr value (Jamal et.al,1991).

2.7 Relationship between resilient modulus and dynamic modulus.

(Amara et.al,2006) used two typical mixes, SM-9.5A and BM-25 for performing the resilient modulus test and dynamic modulus test. The size is one of the parameter which statistically affect the measured resilient modulus value because the Mr value obtained in 100 mm diameter specimen is found to be higher than the 150 mm diameter specimen at all testing temperature. Dynamic modulus performed at temperature of -15°C, 5°C, and 20°C, the frequency is increasing as the phase angle decreases but same at the temperature of 30°C and 40°C, complex behaviour was evolved by phase angle as a function of frequency due aggregate increasing effect. A sigmoidal function shows a good result for fitting dynamic modulus data. Mix type BM-25 identifies a higher dynamic modulus as compare to mix type SM-9.5A at all frequencies and test temperatures. A close relation was noticed betweeen the dynamic modulus and resilient modulus test performed at 5 Hertz and loading time of 0.03s respectively. For the full characterization of the HMA over all temperature and loading frequecies, dynamic modulus test shows a better result than the resilient modulus test. Dynamic test data and reslient modulus test data are used for the analysis of the three pavement structures and it was found that the choice of HMA modulus affects the fatigue life. Also, the fatigue life estimated from the predicted dynamic modulus is smaller than the actual dynamic modulus. Although, the thickness of HMA is indicated as the controlling factor for predicting the fatigue life.

Analytical method is used to calculate the resilient modulus from the dynamic modulus by the implementation of multiaxial linear viscoelastic theory to linear elastic solutions for the indirect tensile test evolved by Hondros (Andrew et.al,2007). The method adopted is verified using the three surface course mixture of 12.5 mm and one base course mixture of 25 mm. The results indicate that the predicted and measured resilient modulus are closely related. Also, a forward model for the back-calculation of dynamic modulus from the resilient modulus was implemented from the result same as the Long Term Pavement Performance material database.

Further, in year 2008 artificial neural network(ANN) is used to develop the methodology for a prediction of the dynamic modulus by using Mr values and binder time - temperature shift factors as an input. The following are the steps involved for conducting the verifcation:

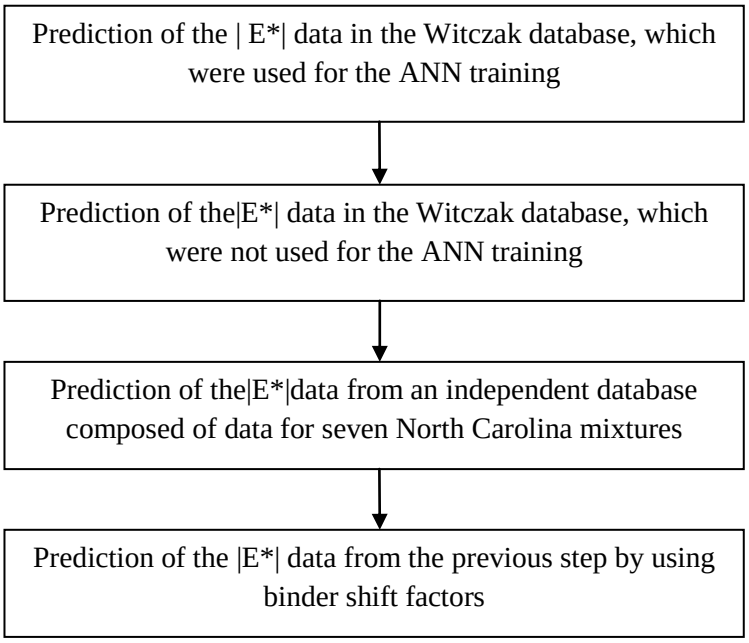

Figure 2.9. Steps in ANN Model (Andrew et.al,2008)

In all the verification steps the ANN model is showing better accuracy than the Witczak model.

Relationship between the dynamic modulus and resilient modulus of asphalt concrete pavement is investigated, and it was concluded that the resilient modulus value is comparable to the dynamic modulus value at the same temperature and at a continuous loading frequency of 5 Hz (Jeff et.al,2015). This type of relationship is also observed in the case of dynamic modulus value which get improved by using the FORTA Fibres and directly tend to improve the resilient modulus value. Laboratory dynamic modulus and fatigue cracking data are used for the design simulation, the maximum reduction of asphalt concrete layer thickness for the FORTA fiber reinforced pavement to get the same predicted pavement performance as the conventional pavement is 32% for both in rutting and fatigue cracking. Hence, as per the given relationship between dynamic modulus and resilient modulus, it is possible to produce the similar thickness reduction in the AC pavement using the resilient modulus data.

Dynamic modulus (various temperature and loading frequency) and resilient modulus (two temperature and loading frequency) test were performed on the asphalt concrete mixtures. Significant changes can be seen on the dynamic modulus(|E*|) test results and the resilient modulus (Mr) test results while increasing the both temperature and loading frequency, i.e.

decrease and increase in both dynamic modulus and resilient modulus with increase in temperature and increase in loading frequency respectively (Yasir et al,2018). Also, the diameter of specimen will have an influencial impact on the both |E*| and |Mr|. A clear trend of direct correlation was developed using the linear regression analysis between the |E*| and |Mr| for a specific loading frequency. A juxtaposition of |E*| and |Mr| indicated that dynamic modulus at a frequecy of 5 Hz and a temperature of 25°C is closely related with the resilient modulus at temperature and loading frequency of 25°C and 300 ms respectively with R^2 of 0.85.Further, a bi-level testing protocol is used to develop the correlation, which is showing that the correlation has a less predictive power. Hence, to overcome this issue statical model is developed and tested.

2.8 Development of Predictive Modeling

Dynamic modulus has been determined from the original bitumen properties among the other variables using the predictive model taking the hardening effect as an account which occurs during the short and long term aging (Witczak et.al,1989). Therefore, actual bitumen viscosity is used as a important predictor variable by replacing the temperature, so that this modification can predict the dynamic modulus at any degree of binder aging. Also, by application of sigmoidal function model form, notable improvement in prediction rationality was attained for using the model at extreme temperature. The final model for dynamic modulus from the statical study was developed with the coefficient of determination equals to 0.93.

Where,

$$\log|E^*| = -0.261 + 0.008225p_{200} - 0.00000101(p_{200})^2 + 0.00196p_4 - 0.03157$$

$$V_a - 0.415\frac{V_{beff}}{V_{beff} + V_a} + \frac{1.87 + 0.002808p_4 + 0.0000404p_{38} - 0.0001786(p)^2 + 0.0164p_{34}}{1 + e^{(-0.716\log f - 0.7425\log \eta)}}$$

E = dynamic modulus of asphalt mix, in 10^5 psi;

η = viscosity of bitumen, in 10^6 poise (at any temperature, degree of aging);

f = loading frequency, in Hz;

V_a= % air voids in the mix, by volume;

V_{beff} = % effective bitumen content, by volume;

p_{34} = % retained on 3/4-in. sieve, by total aggregate weight (cumulative);

p_{38} = % retained on 3/8-in. sieve, by total aggregate weight (cumulative);

p_4 = % retained on No.4 sieve, by total aggregate weight (cumulative);and

p_{200} = % passing No.200 sieve, by total aggregate weight.

Rutting potential of the pavement section is capable to be predicted at 25°C using the resilient modulus (Hamad et.al,2001). Stepwise forward selection regression was used to find relationship between the resilient modulus of asphalt mixes and the material physical properties. Factor of 2 is to be set for the involvement of any criterion. Individual test values are utilized for the development of model. The proposed model for Mr at any temperature is shown below, having coefficient of determination equals to 0.85.

$$M_r \text{ @ Temp °C} = 5.3542 - 0.212\text{Temp.} + 0.111\text{Soft. Pnt.} - 20.170\text{Surf.}$$

Where,

Temp. = Target temperature

Soft.Pnt. = Softening point temperature, °C

Surf. = Surface area

Correlation analysis was carried out to assess the relationship between the resilient modulus at different temperatures, loading frequencies, stability to flow ratio and marshall index values (Jassar et.al,2003). The best correlation was obtained between the Mr at 60°C and stability/flow ratio.

$E_{1.0\ Hz} = 1.155\ SFR$	$R^2 = 0.76$
$E_{0.5\ Hz} = 1.075\ SFR$	$R^2 = 0.81$
$E_{0.33\ Hz} = 1.044\ SFR$	$R^2 = 0.82$

Where,

$E_{i\ Hz}$ = Resilient Modulus at tempetature of 60° C & ith loading frequency,Mpa;

SFR = Stability to Flow ratio,kg/0.01in.

Stepwise linear regression was performed for the modeling of resilient modulus using the SPSS software. Temperature, bitumen content, type of bitumen,aggregates gradation, and

compaction level are the parameter for prediction (Tabatabaei et.al,2008). The equation is shown below for the used model and the Mr correction factor having the reliability of 0.558 and 0.742 respectively.

$$M_r = -106.1TT - 727.2(B) + 5730$$

$$\frac{M_r(T)}{M_r(25)} = 2.35e^{-0.0643(T)}$$

Where,

$M_r(T)$ = Resilient modulus of target temperature

$M_r(25)$ = Resilient modulus of 25 °C

T = Target temperature

BT = Bitumen type.

SPSS statistics analysis was carried out to model the relationship between the resilient modulus of recycled mixes with bitumen emulsion and cement (type 1 cement, pozzolanic cement) and the indirect tensile strength test result, effects of temperature, curing time (Amir et.al,2009). Two step regression analysis has been developed by considering the linear relationship between the Mr and ITS in the first step while Mr with effect of temperature and curing time in the second step. The final combined relationship was established for the both type of cement.

Type 1 cement(R^2 = 0.912)

$$M_r = 12.8ITS - 6535\left(EXP(-0.0381\ T_2) - EXP(0.0381T_1)\right) + 918\ln\left({D_2}/{D_1}\right) - 985$$

Pozzolanic cement(R^2 = 0.907)

$$M_r = 11.2ITS - 6011\left(EXP(-0.0375\ T_2) - EXP(0.0375T_1)\right) + 1088\ln\left({D_2}/{D_1}\right) - 356$$

Where,

M_r = Resilient modulus, MPa

ITS = Indirect Tensile Strength, kPa

D = time of curing, in days

T = temperature at which test performed, °C

Ordinary least square regression analysis has been performed with the application of SPSS statistics software to model and estimate the resilient modulus of RAP, basaltic virgin aggregates and the blend of two, by incorporating effect of stress level, water content and density (Yonghui et.al,2010). The equation proposed was proposed with reliabiity of 0.97.

$$M_r = k_1 Pa \left(\frac{\theta}{\sigma_{atm}}\right)^{k_2} \left(\left(\frac{\tau_{oct}}{\sigma_{atm}}\right) + 1\right)^{k_3}$$

Where,

θ = bulk stress, kPa,

τ_{oct} = octahedral shear stress, kPa,

p_a = 101.32 kPa, atmospheric pressure,

k1, k2 and k3 are regression parameters.

Statical (regression analysis) was carried out using the on a two mix type(AC22 and AC16) for the evaluation of influence of the binder penetration, average size of the fraction replaced, effect of temperature and the additives on the resilient modulus (Maria et.al,2010). This model was further verified using the ANOVA.

$$S = -117.5B + 67.3M + 14101.5 \qquad (R^2 = 0.95)$$

Where,

B = Average binder penetration value

M = Average size of the fraction replaced

It is possible to predict the resilient modulus using the artificial neural network (ANN) model of hot mix asphalt using the predictive index as an input (Chunhua et.al,2011). When comparing this type of model with respect to mathematical statistics method like regression analysis, it indicates the features of preciseness, simple and flexibility.

Artificial Neural Network model for prediction of resilient modulus

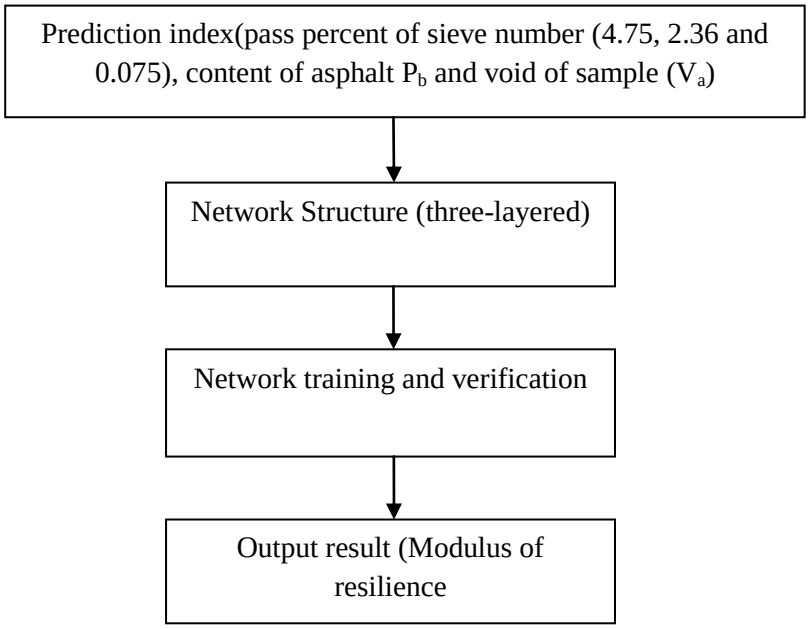

Figure 2.10. Steps for ANN model (Chunhua et.al,2011)

Piezoceramic elements are used to determine the resilient modulus (Mr) of the dense asphlatic concrete (DAC) and semi-dense asphaltic concrete (SDAC). The relation between the Mr with both shear modulus and elastic modulus was expressed provided with the coefficient of determination equals to 0.96 (Patel et.al,2011).

$$M_r = 2.1G_{max}$$

$$M_r = 0.77E_{max}$$

Where,

G = Shear Modulus

E = Elastic Modulus

Witczak's dynamic modulus has been used to derive and estimate the resilient modulus of asphalt concrete layers at equivalent frequency of 4 Hz with coefficient of determination equals to 0.935 (Saglik et.al,2012).

$$\log |E^*| = 3.75 + 0.029p_{200} - 0.00177(p_{200})^2 + 0.0028p_4 - 0.058V_a$$
$$-0.8\frac{V_b}{V_b + V_a} + \frac{3.87 + 0.0298p_4 + 0.004p_{3/8} - 0.000017(p)^2 + 0.0055p_{3/4}}{1 + e^{-2.56+0.89\log(pen.)-0.0015(\log(pen.))^2}}$$

Where,

E* : Dynamic modules, psi

Va : Air Void, %

Vb : Binder Content, %

Pen: : Penetration of binder, 0.1 mm

p_{200} : Percent passing No.200 sieve

p_4 : Percent retained on No.4 sieve

$p_{3/8}$: Percent retained on No.3/8 sieve

$p_{3/4}$: Percent retained on No.3/4 sieve

Characterisation of thermal cracking susceptibility is done using the indirect tensile (IDT) test results (Michael et.al,2016). For this one need to establish a relationship using the empirical equation like the NCHRP developed as shown below.

$$S_t = 7416.712 - 114.016V_a - 0.34V_a^2 - 122.592\,VFA + 0.704VFA^2 + 405.71\log(Pen_{77}) - 2039.296\log(A_{RTFO})$$

where,

St = Indirect Tensile strength test at −10°C (psi,

Va =percent air voids,

VFA = percent voids filled with asphalt,

Pen77 =penetration at 77°F,

ARTFO = intercept of RTFO-conditioned binder's viscosity-temperature relationship.

Further, the relationship was modified to improve the reliability as per the requirement using the microsoft Excel Solver package, SPSS statistics, Artificial Neural Network in three different stage of effort respectively.

$$S_t = -9.901 + 20.737\ PMF + 2.674PG_{High} - 6.407PG_{Low} + 0.669\ Ang + 356.593\ FA + 1.027P_{1/2''} + 2.517P_{3/8''} - 3.768P_4 + 5.151P_8 + 3.452P_{100} - 62.733P_{200} - 0.017AV$$

Where,

PMF = polymer modification factor, either 1 or 0 for polymer modified and unmodified binder

PG_{High} = high PG temperature;

PG_{Low} = absolute value of low PG temperature,

Ang = percent angularity;

FA = fines/asphalt ratio;

$P_{1/2''}$ = percent passing the 1/2''

$P_{3/8''}$ = percent passing the 3/8''

P_4 = percent passing the no.4 sieve;

P_8 = percent passing the no.8 sieve;

P_{100} = percent passing the no.100 sieve;

P_{200} = percent passing the no.200 sieve;

AV = percent air voids.

Prediction model has been developed between the ITS and independent parameter includes temperature, Marshall stability and optimum bitumen content using the regression analysis (Gupta et.al,2018).

$$ITS_{X_N} = A_0 + A_1T + A_2S + A_3OBC$$

Where,

ITS_{X_N} = Indirect tensile strength (N/mm^2) ;

X = Filler materials (stone dust or cement) ;

N = Filler content (1% or 2%) ;

A_0 = Constant ;

A_1, A_2, A_3 = Regression coefficients ;

T = Temperature (°C) ;

S = Marshall stability (Kg) ;

OBC = Optimum binder content (%).

Based on the pneumatic repeated load test device, resilient modulus of the wearing asphalt concrete (AC) layer for different test conditions and mix properties was developed using the SPSS software tools having the coefficent of determination equal to 0.886 (Hilal et.al,2018).

$$M_r = -12979074.90 - 16552.594\,T - 52354.064\,P_s - 15094.311\,A_v + 301011.367\,\eta + 2306479.544\,SA$$

Where,

Mr = modulus of resilience in (psi);

T = testing temperature in (°C);

Ps = asphalt content in (%);

Av = percent air voids;

η = viscosity of asphalt (pas.sec);

SA = surface area (kg/cm2).

Comparision between the different empirical relationship established by different researchers for finding the resilient modulus values using the binder (50/70, PMB) (Canser et.al,2010).

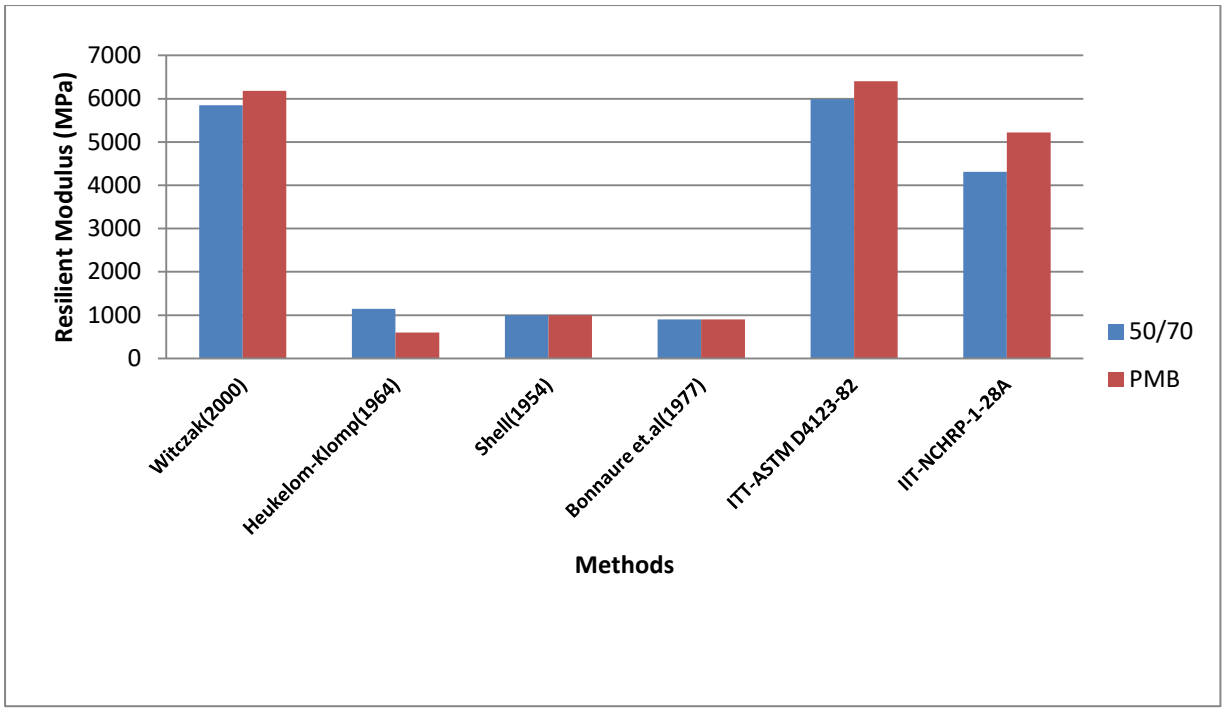

Figure 2.11. Comparison between resilient modulus obtained using different empirical relationship (Canser et.al,2010).

CHAPTER 3 : LABORATORY WORK

This chapter includes the types of materials used and their performance tests as per the standard code. The sieve test, gradation as well as the blending for different grade of mixes and the optimum bitumen content results are represented as well in this chapter.

3.1 Material Used

3.1.1 Bituminous Binder

The bituminous binder used for the experimental research are viscosity grade 40 bitumen (VG-40), polymer modified bitumen grade 40 (PMB-40) and crumb rubber modified bitumen grade 60 (CRMB-60). All three bitumen used are tabulated with the manufacturing company names presented in table 3.1.

Table 3.1. Bitumen Type and its Manufacturing Company

Bitumen Type	Manufacturing Company
VG-40	OOMS Polymer Modified Bitumen Pvt. Ltd.
PMB-40	OOMS Polymer Modified Bitumen Pvt. Ltd.
CRMB-60	ITPL Infrastructures Pvt. Ltd.

The various performance test on the bituminous binder were carried out as per the code includes the specific gravity, softening point , penetration, kinematic viscosity and dynamic shear rheometer (DSR) test for original binder grading to check for the failure temerature, complex modulus, etc. The specific gravity of bitumen is determined at 25°C using the specific gravity bottle of 50 ml capacity as per IS code 1202 which represents the ratio of the mass of the given volume of bituminous material to the mass of the same volume of water at the same temperature as bitumen. The penetration test is performed as per IS code 1203 indicates the penetration depth in tenth of a millimeter into a bituminous material under a standard condition of temperature (25°C), load (100 gram) and time (5 second). Generally, penetration value is the measure of hardness or softness based upon the consistency of the asphaltic material which is the deciding factor for the grading of the bitumen. Ring and Ball apparatus is used to determine the softening point of the bitumen to check for the degree of softening. Thus, asphaltic material having the high value of softening point will shows the lower penetration value. In order to determine the kinematic viscosity, Brookfield viscometer

apparatus is used at a specified temperature depending upon the types of bitumen. The test results are tabulated in table 3.2 to 3.4.

Table 3.2. Summary of VG-40 test results

VG40				
SI.No.	Characteristics	Obtained Value	Limit	Code
1	Specific Gravity	1.05	-	IS 1202
2	Penetration at 25°C, 100 g, 5 s, 0.1mm,Min	49	35	IS 1203
3	Softening Point,°C,Min	58	50	IS 1205
4	Kinematic Viscosity,135°C,Min	525	400	IS 1206(Part 3)
5	Failure Temperature °C	88	-	AASHTO T315
6	G*, kPa	1.14	-	AASHTO T315
7	G*/sinδ,kPa	1.15	≥ 1	AASHTO T315

Table 3.3. Summary of PMB-40 test results

PMB40				
SI.No.	Characteristics	Obtained Value	Limit	Code
1	Specific Gravity	1.03	-	IS 1202
2	Penetration at 25°C, 100 g, 5 s, 0.1mm,Min	33.3	30-50	IS 1203
3	Softening Point,°C,Min	81.3	60	IS 1205
4	Viscosity,150°C,Poise	4	3-9	IS 1206(Part 3)
5	Failure Temperature °C	94	-	AASHTO T315
6	G*, kPa	1.23	-	AASHTO T315
7	G*/sinδ,kPa	1.32	≥ 1	AASHTO T315

Table 3.4. Summary of CRMB-60 test results

CRMB60				
SI.No.	Characteristics	Obtained Value	Limit	Code
1	Specific Gravity	1.01	-	IS 1202

2	Penetration at 25°C, 100 g, 5 s, 0.1mm,Min	58.5	50	IS 1203
3	Softening Point,°C,Min	67	60	IS 1205
4	Viscosity,150°C,Poise	5.25	3-9	IS 1206(Part 3)
5	Failure Temperature °C	94	-	AASHTO T315
6	G*, kPa	1.37	-	AASHTO T315
7	G*/sinδ,kPa	1.51	≥ 1	AASHTO T315

3.1.2 Aggregates

The natural aggregates used are obtained from the Badarpur site. All the important properties were investigated for the aggregates before their use by adopted test method as per the Indian standard code. These test may includes the specific gravity, water absorption, Impact value, combined flakiness and elongation index.

The specific gravity test of aggregates is done to check the quality of aggregates in terms of strength whether the aggregates is weak or strong in nature. The aggregates shows the higher specific gravity will be stronger in nature as compare to the aggregates with lower specific gravity which will be generally weaker in nature. Indirectly using the specific gravity test the water absorption of aggregates is also determined to examine the water holding capacity of aggregates.

The aggregate impact value test is performed in order to measure the toughness ability of the aggregates against the dynamic action of the vehicular loads having the tendency to disintegrate the aggregates into smaller pieces. Therefore, the impact value indicates the resistance of aggregates against the shocking or impact behaviour due to the loading action. The lower impact value of aggregate will have tendency to resist more against the sudden impact or shocks as compare to the higher or moderately higher impact value.

The combined flakiness and elongation index test is conducted to determine the suitability of aggregates shape for the mix design, in the sequence with the flakiness index test followed by the elongation index test on the non-flaky aggregates. As the cubical shape of the aggregate is best suitable for the mix design as compare to the flaky and elongated particles due to proper interlocking between the aggregates which is not possible in the case of the mixes contains the flaky or elongated aggregate particles. Therefore, it is necessary to find out the index to decide which certain range of flaky and elongated particles in the aggregates are suitable for

the particles interlocking and to resist against the compaction effort. The test results for aggregates are presented in table 3.5 and 3.6.

Table 3.5. Summary of 20 mm aggregate size test results

Property of 20 mm Aggregate Size	Test Data	Limit as per MoRTH for BC and DBM grade	Limit as per MoRTH for SMA grade	Code
Aggregte Impact Value (%)	16.96	Max 27%	< 18%	IS 2386 (Part 1)
Combined Flakiness and Elongation Index(%)	26.36	Max 35%	< 30%	IS 2386 (Part 4)
Water absorption(%)	0.32	Max 2%	< 2%	IS 2386 (Part 3)
Aggregate Specific Gravity	2.65	Min 2.5	Min 2.5	

Table 3.6. Summary of 10 mm aggregate size test results

Property of 10 mm Aggregate Size	Test Data	Limit as per MoRTH for BC and DBM grade	Limit as per MoRTH for SMA grade	Code
Aggregte Impact Value (%)	15.88	Max 27%	< 18%	IS 2386 (Part 1)
Combined Flakiness and Elongation Index(%)	25.89	Max 35%	< 30%	IS 2386 (Part 4)
Water absorption(%)	0.27	Max 2%	< 2%	IS 2386 (Part 3)
Aggregate Specific Gravity	2.69	Min 2.5	Min 2.5	

3.1.3 Fillers

The fillers used in the mix design includes the stonedust and lime. The specfic gravity and the water absoption for both the fillers has been determined as per the IS code 2386 - Part 3 to investigate the suitability towards the design procedure of hot mix asphalt. The test results are shown in table 3.7 and 3.8 .

Table 3.7. Summary of test results for stone dust

Property of Stone Dust	Test Data	Limit as per MoRTH for BC and DBM grade	Limit as per MoRTH for SMA grade	Code
Water absorption(%)	1.262	Max 2%	< 2%	IS 2386 (Part 3)
Aggregate Specific Gravity	2.870	Min 2.5	Min 2.5	

Table 3.8. Summary of test results for lime

Property of Lime	Test Data	Limit as per MoRTH for BC and DBM grade	Limit as per MoRTH for SMA grade	Code
Water absorption(%)	1.280	Max 2%	< 2%	IS 2386 (Part 3)
Aggregate Specific Gravity	3.43	Min 2.5	Min 2.5	

3.2 Fixation Of Mixing And Compaction Temperature

Equiviscous temperature ranges is to be used for selecting laboratory mixing and compaction temperatures for asphalt mix design. Compaction temperature and mixing temperature are determined where the viscosity versus temperature line crosses the viscosity range of 0.28±0.03 Pa.s and 0.17±0.02 Pa.s. respectively. The Brookefield viscometer was used to determine the viscosity at different temperatures.

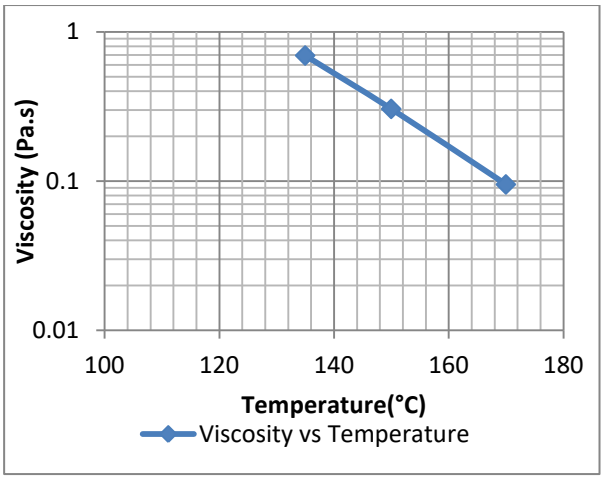

Figure 3.1. Viscosity vs Temperature for VG 40

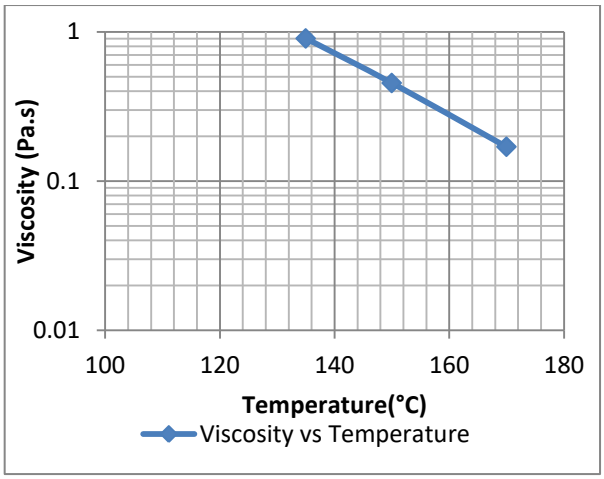

Figure 3.2. Viscosity vs Temperature for PMB 40

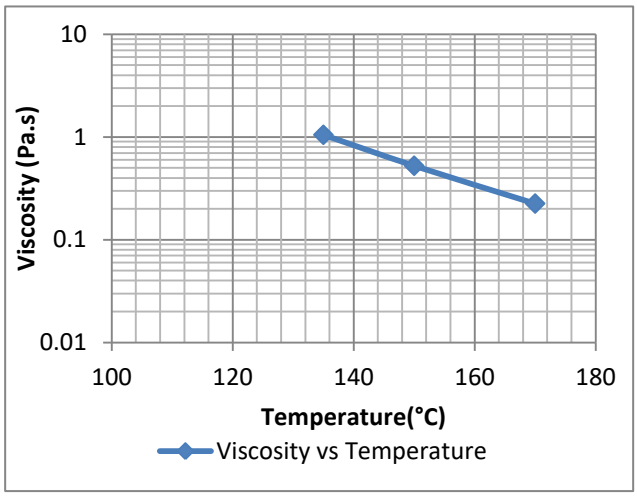

Figure 3.3. Viscosity vs Temperature for CRMB 60

From the graph the compaction and mixing temperature has been determined and summarised in table 3.9,

Table 3.9. Compaction and Mixing Temperature for binders

Type of binder	Compaction temperature (°C)	Mixing Temperature (°C)
VG 40	151	160
PMB 40	161	166
CRMB 60	165	169

3.3 Aggregate Gradation And Blending

The gradation of aggregate is an important factor which should be considered in the mix design, in order to provide an adequate volume of voids in the asphalt-aggregate mixture to

put in the proper asphalt film thickness on each and every aggregate particles. Also, this provision to provide air-void system may allow for thermal expansion of the asphalt in the mix.

The gradation of natural aggregates, stone dust and lime has been conducted using the sieve analysis procedures as per AASHTO T11 and T27 as well as the MoRT&H guideline (fifth revision). The results should be reported as an nearest whole cummulative percentage passing over the specified sieve size. Trial and error method is adopted to fix the mix gradation for different grade i.e., BC Grade I and II, DBM Grade I and II and SMA NMA 13 and 19 mm. To further minimize the effect of gradation, blending is such that for each mix it is near to center line. The blending is done by multiplying the percent passing a sieve for an individual aggregate stockpile and the proportion of aggregate stockpile to be added in the blend which makes the total equal to unity. The gradation curve and proportioning of different aggregates after blending for each grade are shown in figure 3.4 to 3.9 and table 3.10 respectively.

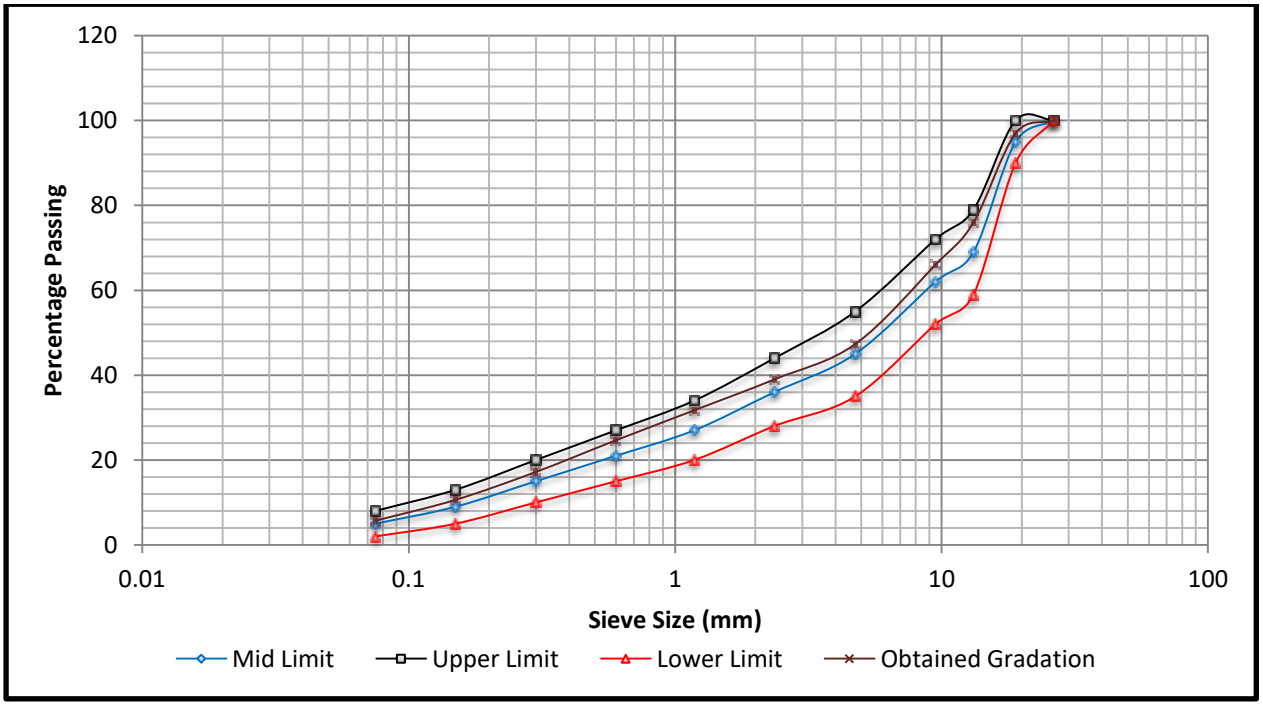

Figure 3.4. Gradation Curve for Bituminous Concrete Grade 1

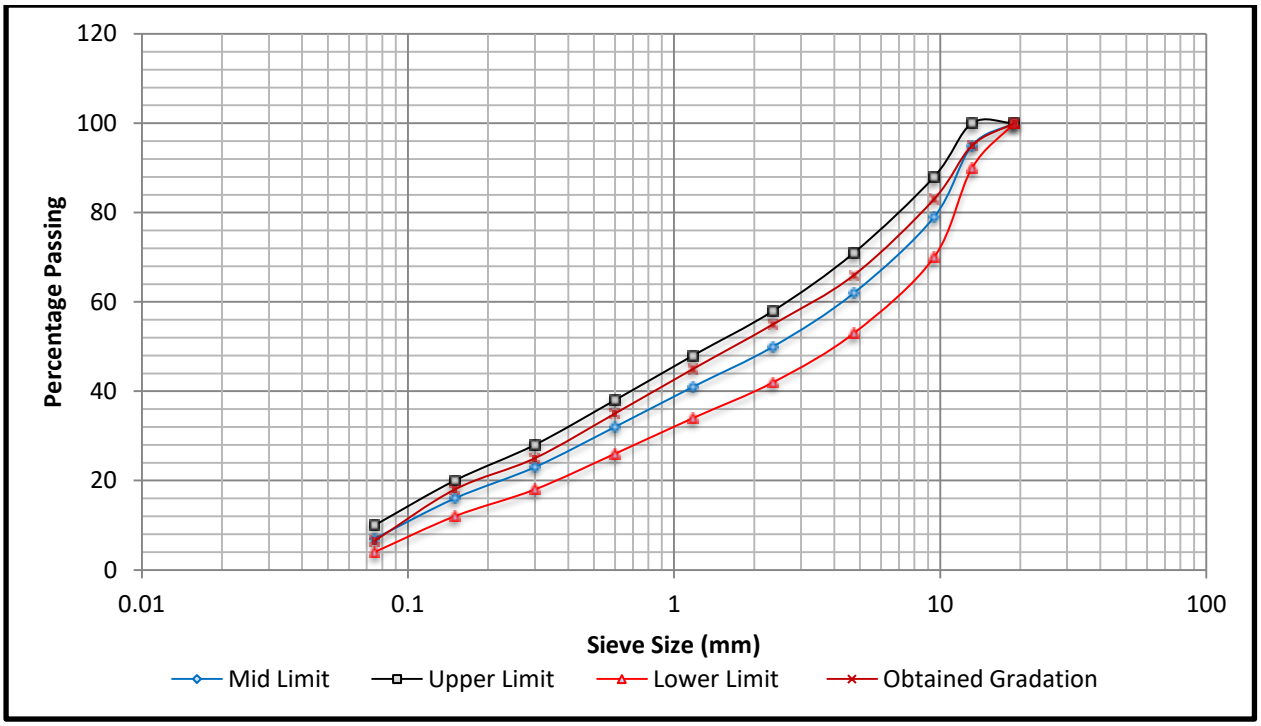

Figure 3.5. Gradation Curve for Bituminous Concrete Grade 2

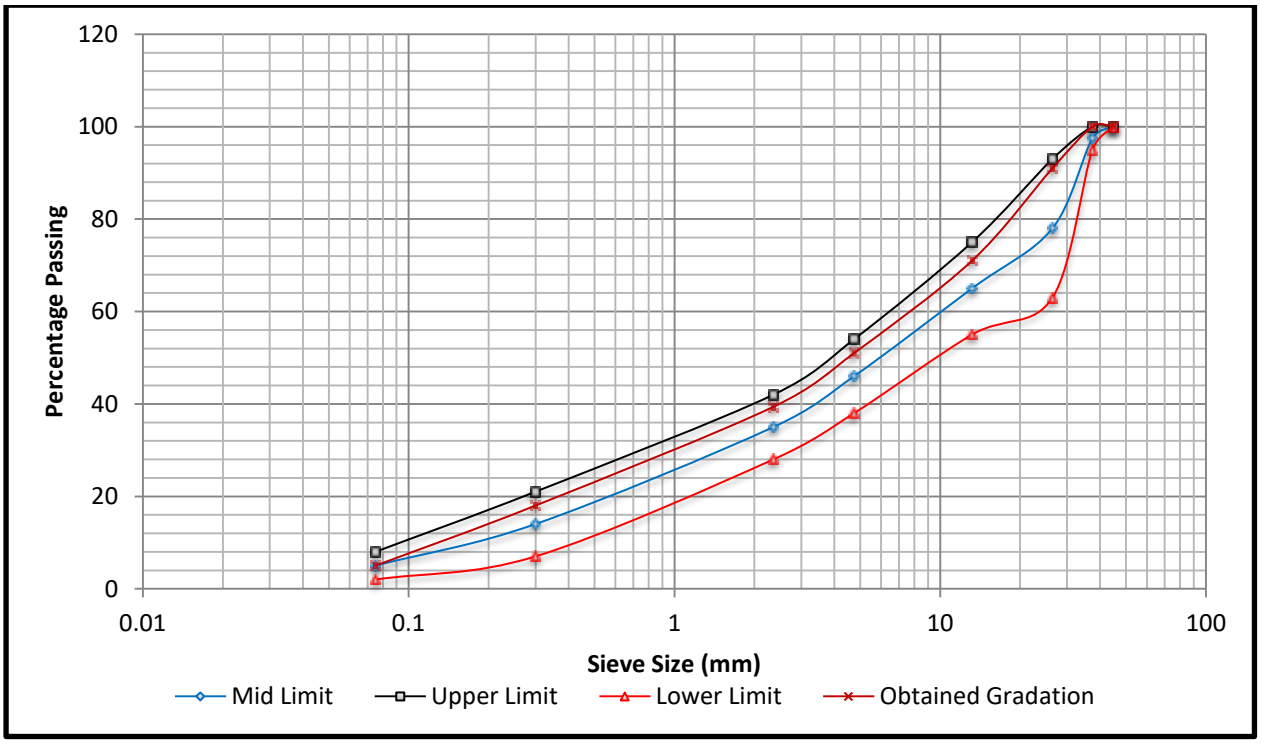

Figure 3.6. Gradation Curve for Dense Bituminous Concrete Grade 1

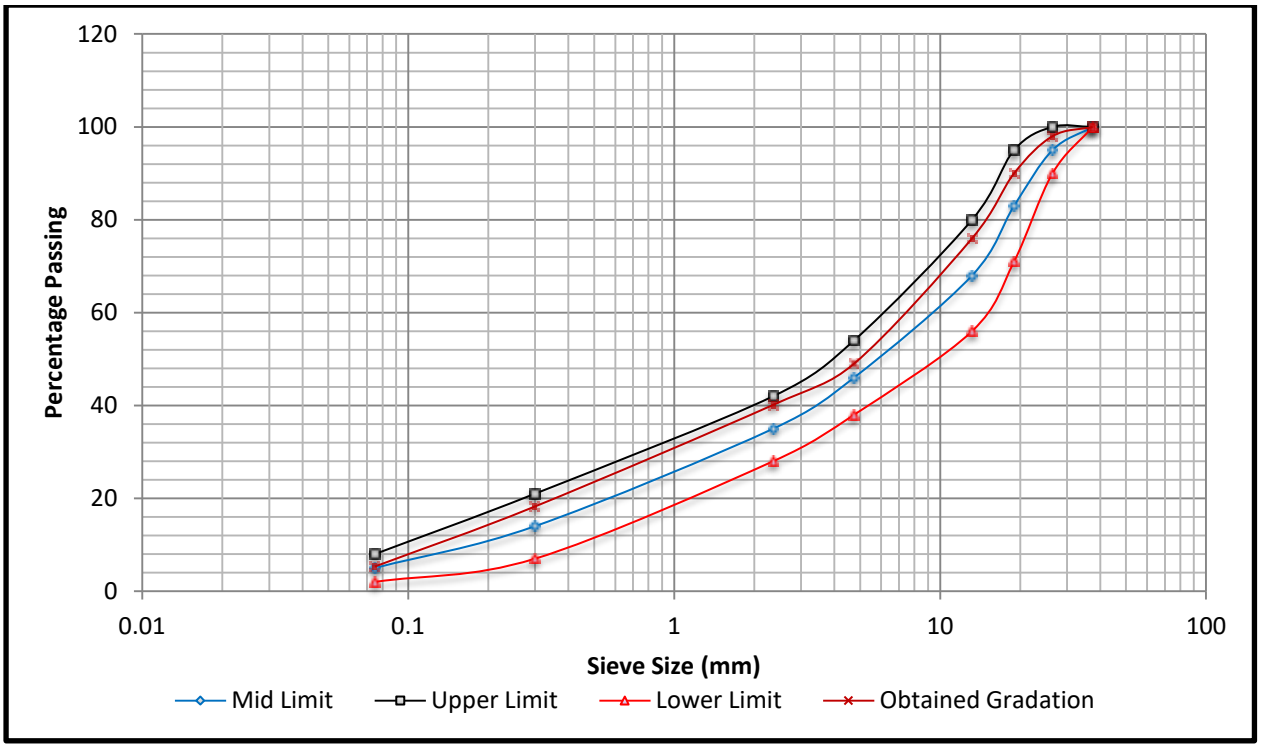

Figure 15. Gradation Curve for Dense Bituminous Concrete Grade 2

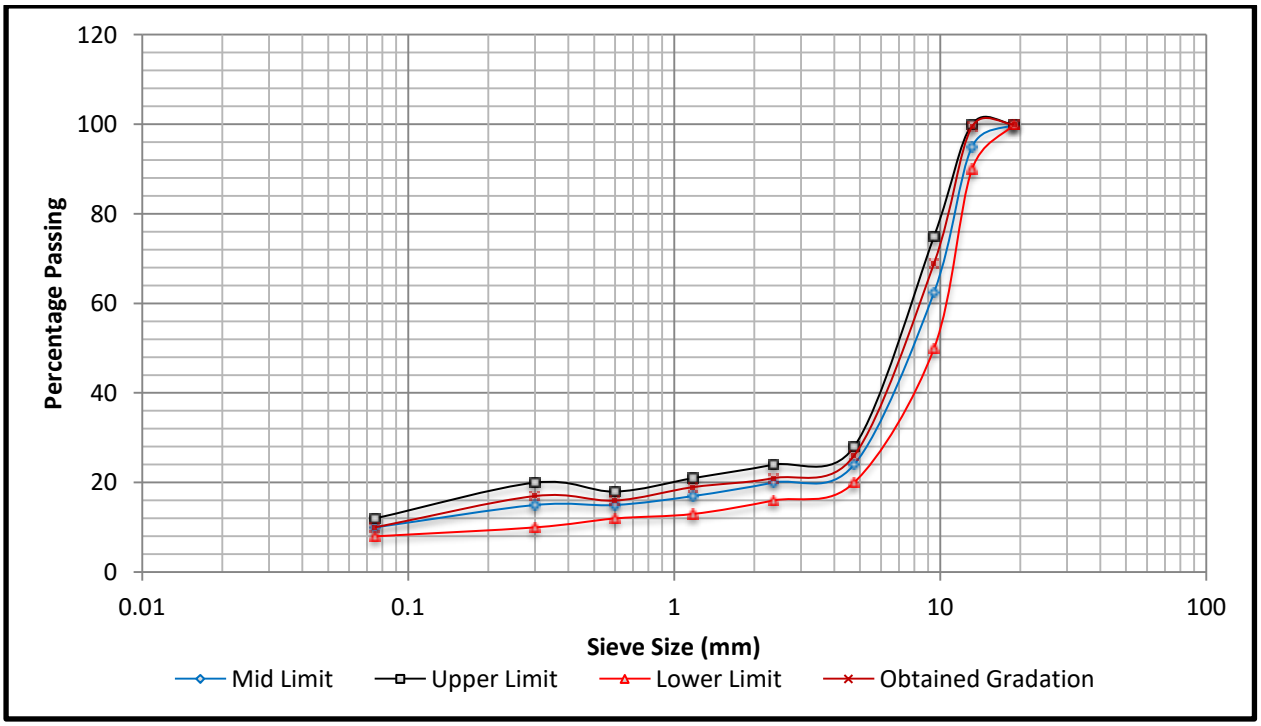

Figure 3.8. Gradation Curve for SMA NMA 13 mm

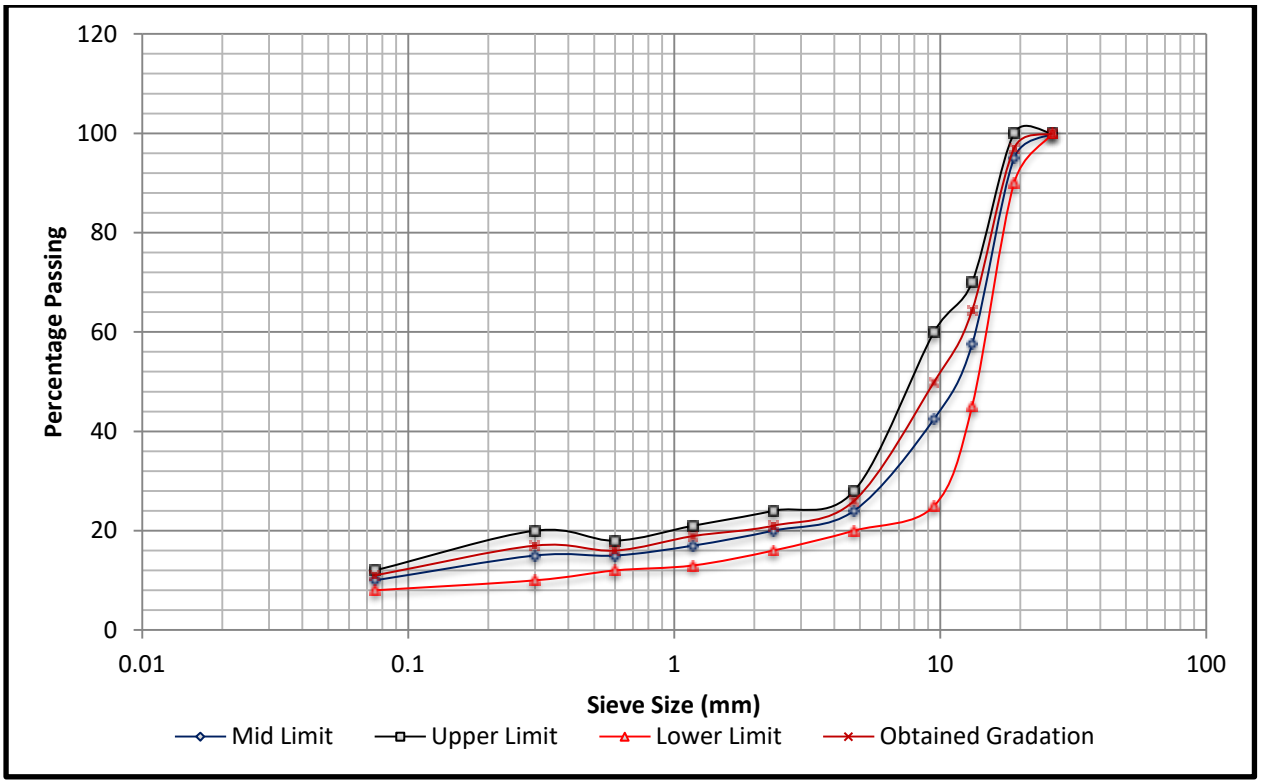

Figure 3.9. Gradation Curve for SMA NMA 19 mm

Table 3.10. Summary of proportioning for different aggregate size for different mix type

Mix Type	Proportioning for Different Aggregate Size			
	20 mm	10 mm	Stone Dust	Lime
BC I	360	324	492	24
BC II	0	636	540	24
DBM I	432	252	492	24
DBM II	420	276	480	24
SMA 13 mm	0	900	252	48
SMA 19 mm	588	324	216	72

3.4 Mix Design Using Marshall Method And Fixation Of Optimum Bitumen Content (OBC)

3.4.1 Preparation of Marshall Samples

Marshall samples were casted as per Marshall method of mix design in the Asphalt Mix Design Method 7th edition (Asphalt Institute MS-2), procedures given in the ASTM D6926 for different bituminous mixes i.e., BC Grade I and II, DBM Grade I and II and SMA NMA 13 and 19 mm using three different binder viz.,VG40, PMB40 and CRMB60. The test

specimen should be prepared for standard size of 63.5 mm in height and 101.6 mm in diameter at a fixed compaction and mixing temperature demonstrated earlier in the section 3.2. Total weight of aggregate taken for each grade of mixture will be 1200 grams and the bitumen was added to it has been calculated using the percentage of bitumen content by the overall weight of the aggregate. The trial mixes were prepared at different percentage of binder content i.e., 4%, 4.5%, 5% and 5.5% for BC Grade I and II and DBM Grade I and II grade whereas for SMA-13 mm and SMA-19 mm grade, the binder content percentage varies i.e., 5.5%, 5.75%, 6.00% and 6.25% to find the OBC. Hence, three identical samples at each trial of bitumen percentage were casted with all three binder mentioned above. So, total 36 (3×4×3) samples should be required for each mix. In total 216 (36×6) marshall samples should be required only to find out the OBC.

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Figure 3.10. Heating of sample before mixing

Figure 3.11. Manual Mixing of sample

Figure 3.12. Mould for casting of samples

Figure 3.13. Compactor Setup

3.4.2 Drain down Test

The drain down test is performed as per the IRC SP: 79 code on loose mix of stone matrix asphalt using the wire basket assembly by keeping the samples in oven at a temperature of 170°C for 1 hour. The maximum limit for draindown is 0.3%. But due to fail in the specified requirement of test, the cellulose fibres are added to the minimum requirement of 0.3% to the SMA mix in order to absorb the asphalt and improves the draindown. The draindown percent is calculated by the mentioned formula below,

$$\text{Draindown percent} = \frac{(D-C)}{(B-A)} \times 100$$

Where,

A = mass of empty wire basket in gram

B = mass of wire basket and sample in gram

C = mass of empty plate for collection of drained material in gram

D = mass of the catch plate and drained material in gram

Figure 3.14. Asphalt mix in basket for drain down

3.4.3 Determination of Bulk Specific Gravity (G_{mb})

Bulk specific gravity of laboratory casted marshall samples is determined using the ASTM D2726 or AASHTO T 166 standard named as saturated-surface dry (SSD) method. In this method first water absorption of the specimen should be checked which need to be less than or equal to 2.0 percent. For the purpose of computation of bulk specific gravity, the three weights are required that are mass of sample in air (A), mass of sample in water using the water bath at 25°C (B) and the mass of SSD sample obtained by quickly blotting the sample (C). Generally, the bulk specific gravity is defined as the mass of the sample divided by the mass of the water it displaces.

$$G_{mb} = \frac{A}{(C-B)}$$

Figure 3.15. Mass of sample in air

Figure 3.16. Mass of sample submerged within the water

Figure 3.17. Blotted samples for SSD weight

3.4.4 Determination of Theoretical Maxixmum Specific Gravity (G_{mm})

The theoretical maximum specific gravity is determined using the rice apparatus method as per the ASTM D2041 and AASHTO T 209 standard. The procedure for performing test includes the preparation of loose mix of asphalt samples which should be warmed and separated into a loose as well as individually coated aggregates. The dry loose mix samples need to be placed in the metal bowl and submerged fully with water. A vaccum lid is fitted over the top of the metal bowl and placed on the vibratory shaker. The vaccum pump need to be started and required to maintain the absolute pressure of 27.5 mmHg. After maintaining this pressure vibratory shaker should be started to provide gentle agitation which will help in the removal of any air between the particles. The theoretical maximum specific gravity is calculated as per the given expression below,

$$G_{mm} = \frac{A}{(A+B-C)}$$

Where,

A = Weight of loose mix in air;

B = Weight of container and water;

C = Weight of loose specimen, container and water.

Figure 3.18. Loose mix of samples in a tray

Figure 3.19. Rice Apparatus Setup

3.4.5 Computation of Air voids, voids filled with mineral aggregates (VMA) and voids filled with bitumen (VFB)

The air voids is the influencial volumetric parameter for the design of the asphalt mixture. It is recommended that the 4 % of air voids should be the desired level after several years of traffic. If the maintained air voids is less than 2% then the chances of rut and shove may occurs due to heavy traffic whereas if the same is more than 5% then, the possibility of development of premature cracking, ravelling and stripping can be seen. As per the MoRT&H guidelines (fifth edition), the amount of air voids can be varied between 3% - 6% of the total mix volume. The air voids is calculated using the value obtained from the bulk specific gravity and the theoretical maximum specific gravity as per the relation given below,

$$V_v = \frac{G_{mm} - G_{mb}}{G_{mm}} \times 100$$

The voids in mineral aggregate (VMA) is defined as the intergranular void space within the aggregate particles in a compacted asphalt mixture which may includes the air voids and the effective asphalt content, expressed as a percent of the total volume. VMA is calculated using the following expression mentioned below,

$$VMA = 100 - \frac{G_{mb} P_s}{G_{sb}}$$

The voids filled with asphalt (VFA) is defined as the percentage by volume of the voids filled with mineral aggregates (VMA) that is filled with the effective binder. VFA is calculated using the following expression mentioned below,

$$VFA = \frac{VMA - V_v}{VMA} \times 100$$

Where,

V_v = Percent Air voids in the mix;

G_{mm} = Theoretical maximum specific gravity of the mix;

G_{mb} = Bulk specific gravity of the mix;

VMA = voids in the mineral aggregate;

Gmb = bulk specific gravity of mixture;

Ps = percentage of aggregate by total mix weight;

Gsb = bulk dry specific gravity of the aggregate;

VFA = voids filled with asphalt.

3.4.6 Marshall Stability Test

The Marshall Stability testing machine is similar to compression testing device conforming to ASTM D6927.

3.4.6.1 Conditioning of Sample

The conditioning of test specimen is to be done by immersing it into the water bath at a temperature of 60°C for 30 to 40 minutes before the testing.

Figure 3.20. Water bath maintaining a temperature of 60 degree celcius

Figure 3.21. Immersed Specimen in water bath

3.4.6.2 Testing of Specimen

The constant loading rate of 51 mm per minute is applied on the sample with the help of cylindrical segment testing head maintaining horizontal position due to two perpendicular guide posts and allow for free vertical movement during the test. The stability value and deformation would be determined using the calibrated proving ring and the Marshall flow meter respectively, which may be equipped within the test setup.

Figure 3.22. Marshall Testing Machine **Setup**

Figure 3.23. Appearance of Marshall Samples after testing

3.4.7 Finding of Optimum Bitumen Content (OBC)

The graphs are plotted between the trial bitumen content and Marshall test results data as well as various volumetric results to reach at the final optimum bitumen content for different grade of mixes with the three different binder as mentioned earlier in section 3.4.1. The OBC is

computed by taking mean of the bitumen content corresponding to the maximum stability value, maximum unit weight and 4 % air voids. The obtained OBC value is also checked with the VFB and VMA which should be within the range of the limit provided in the MoRT&H guidelines (Fifth edition).

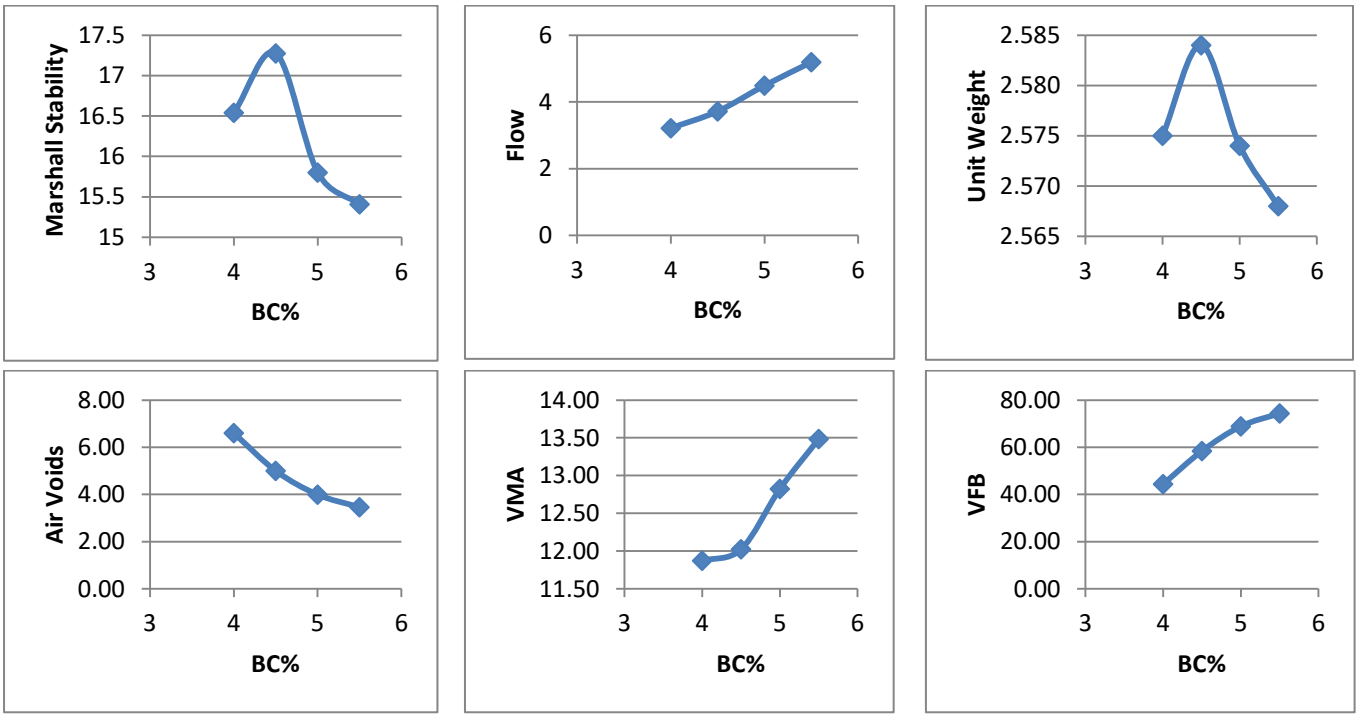

Figure 2.24. Graphs of Marshall Test Result and Volumetric properties for BC I grade with VG40 binder

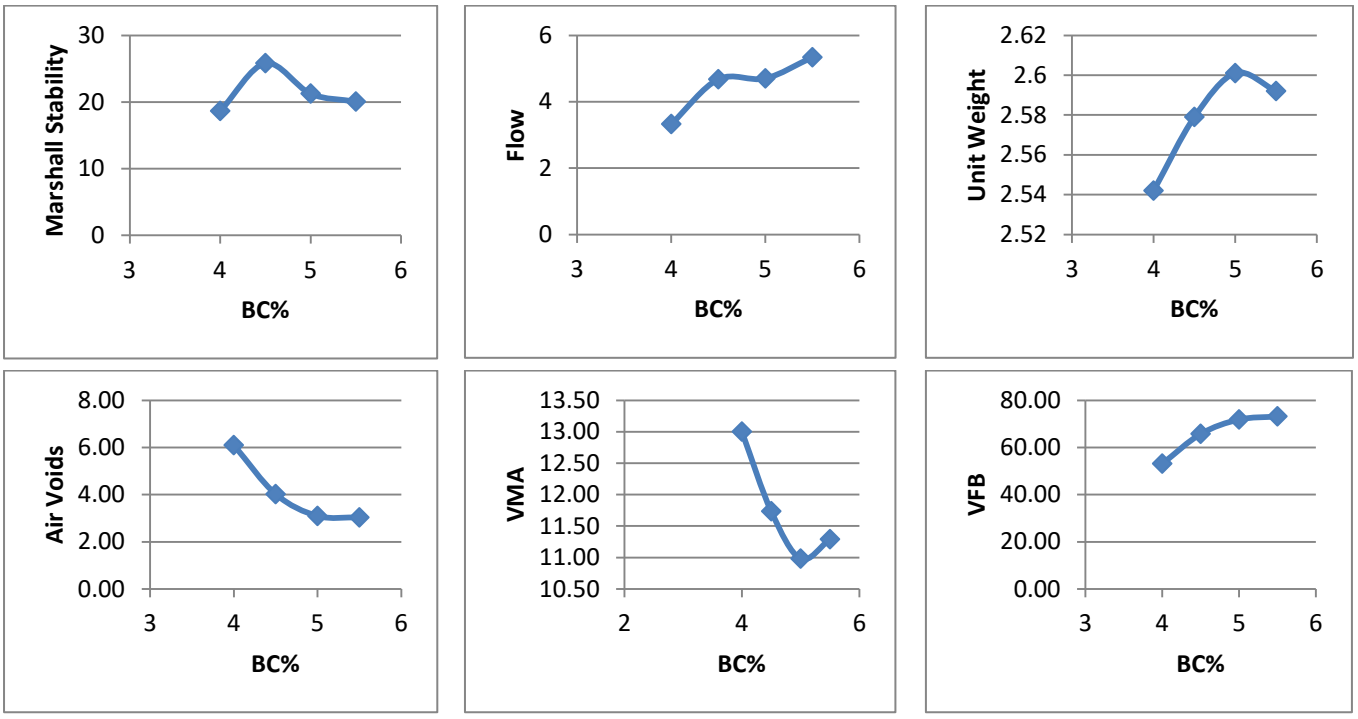

Figure 2.25. Graphs of Marshall Test Result and Volumetric properties for BC I grade with PMB40 binder

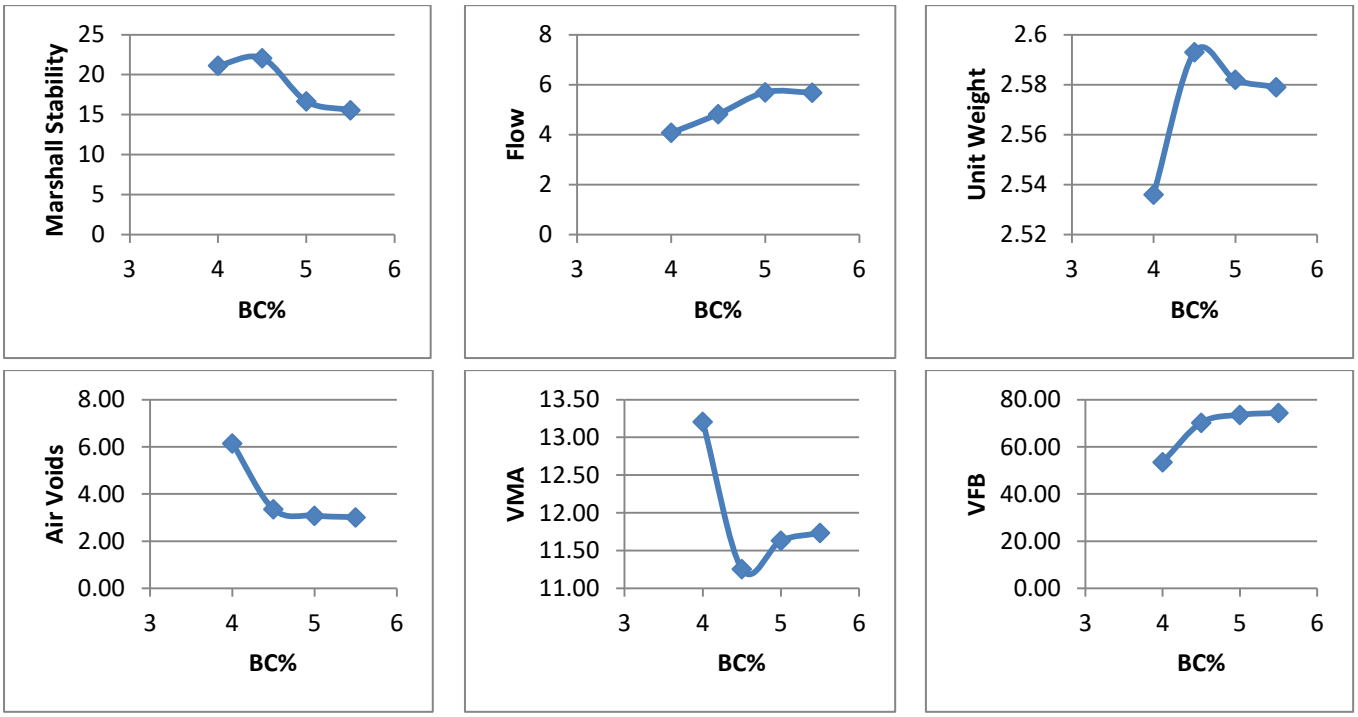

Figure 2.26. Graphs of Marshall Test Result and Volumetric properties for BC I grade with CRMB60 binder

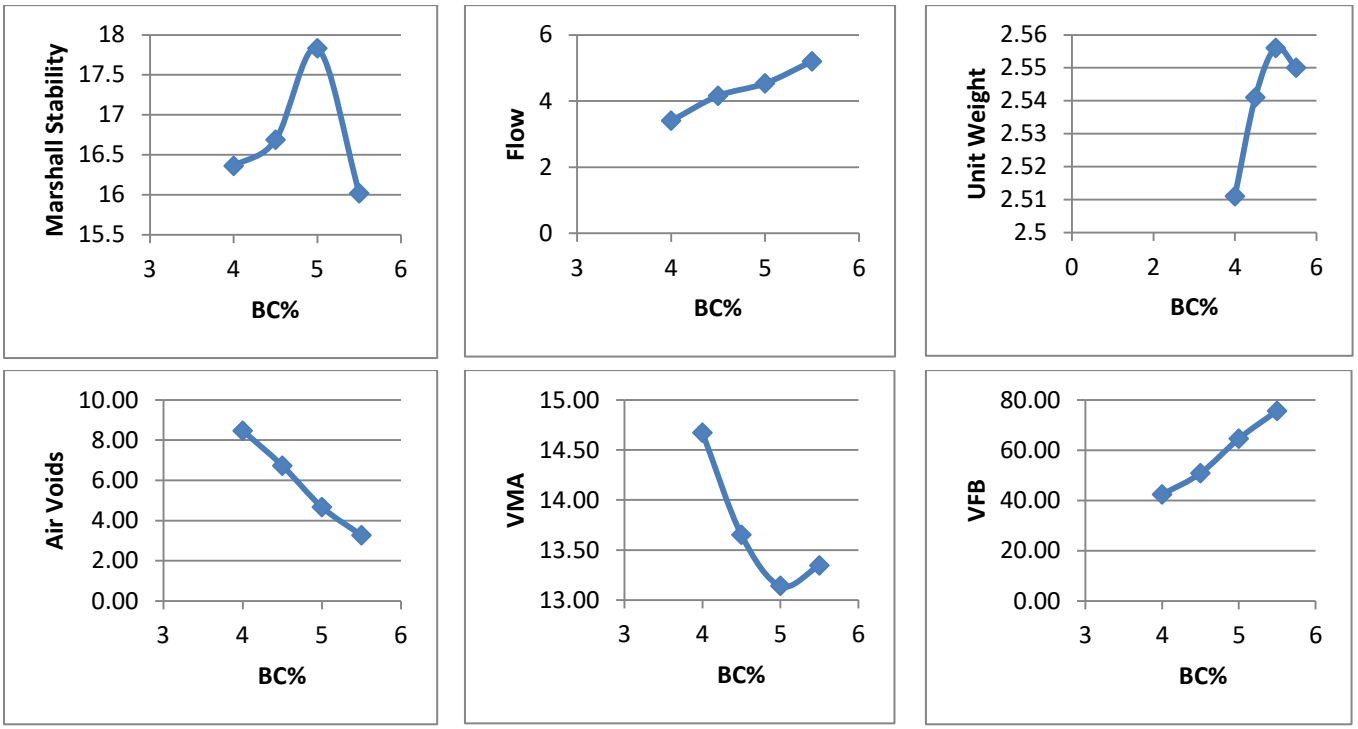

Figure 2.27. Graphs of Marshall Test Result and Volumetric properties for BC II grade with VG 40 binder

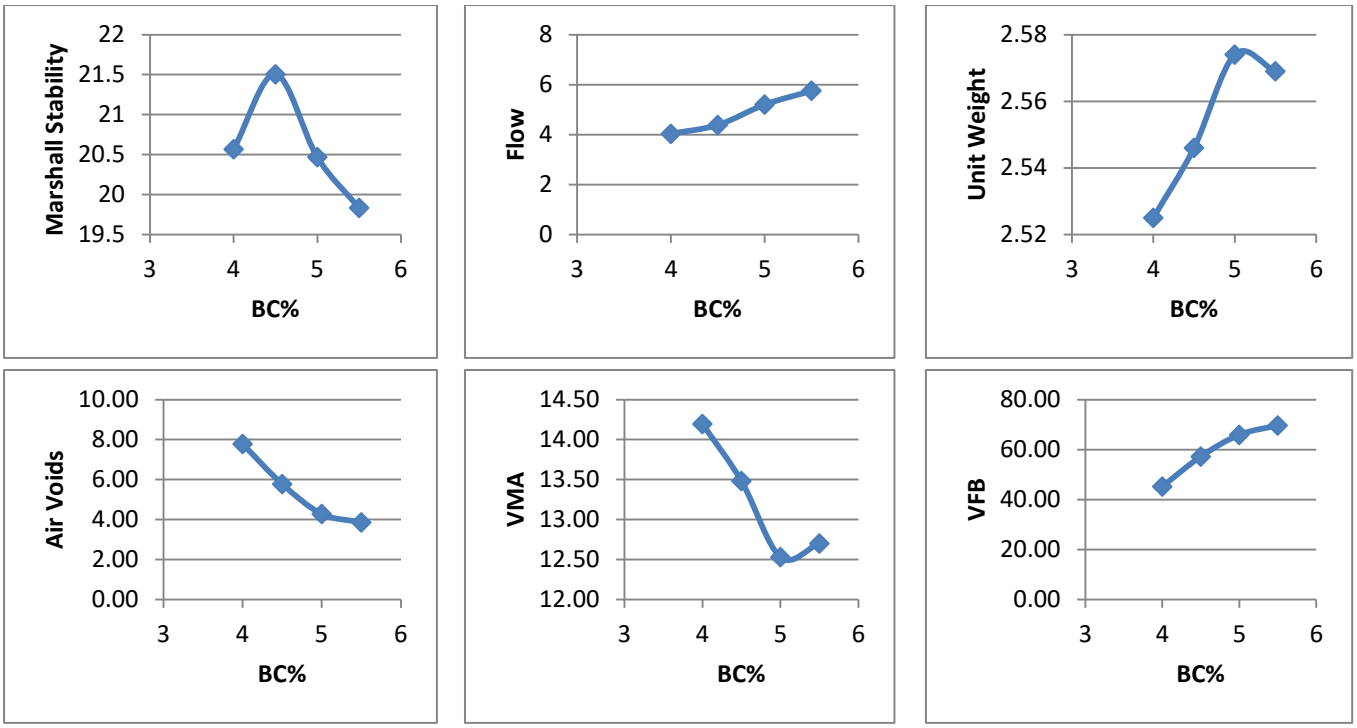

Figure 2.28. Graphs of Marshall Test Result and Volumetric properties for BC II grade with PMB 40 binder

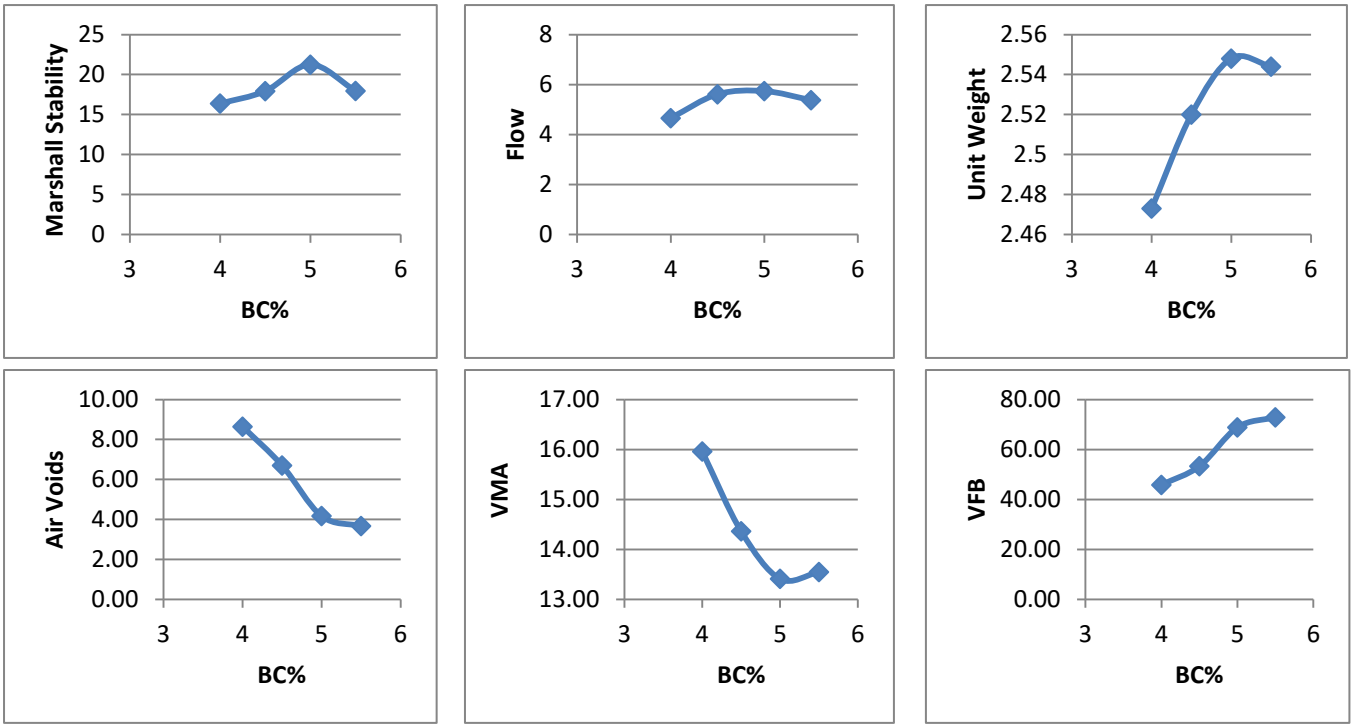

Figure 2.29. Graphs of Marshall Test Result and Volumetric properties for BC II grade with CRMB 60 binder

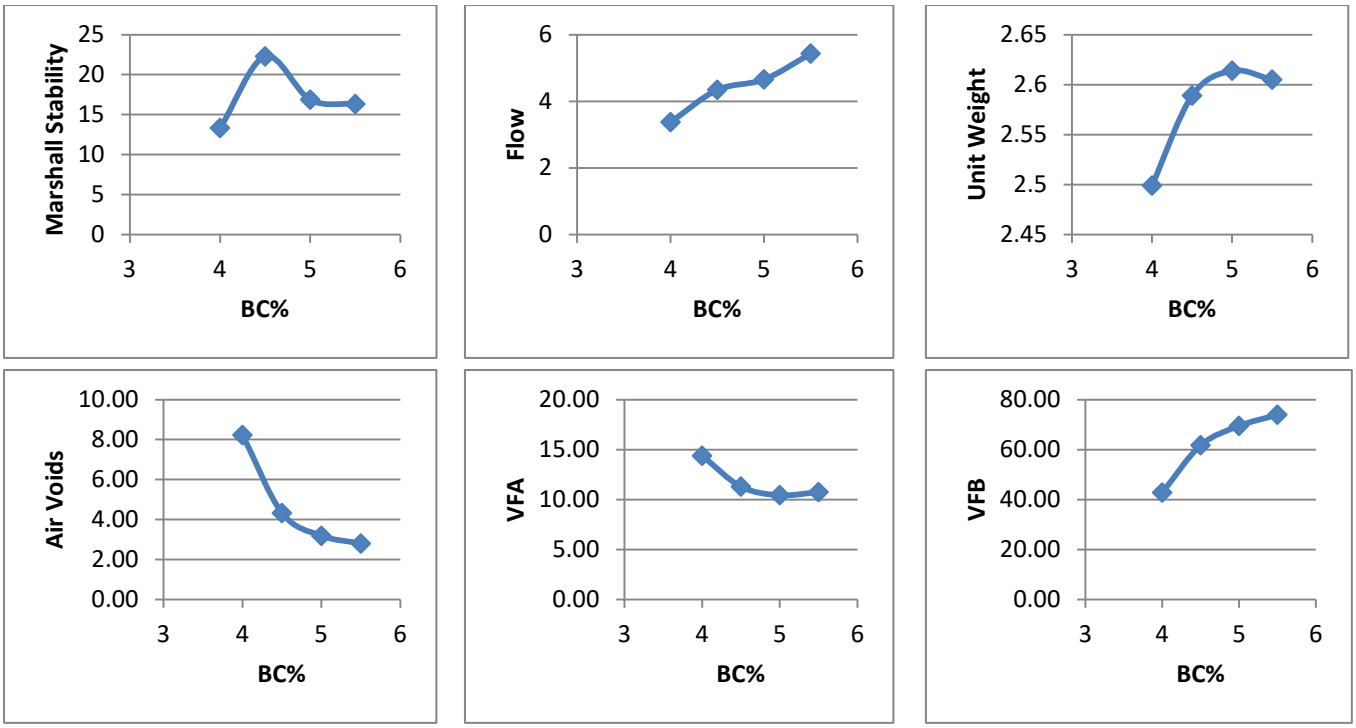

Figure 3.30. Graphs of Marshall Test Result and Volumetric properties for DBM I grade with VG 40 binder

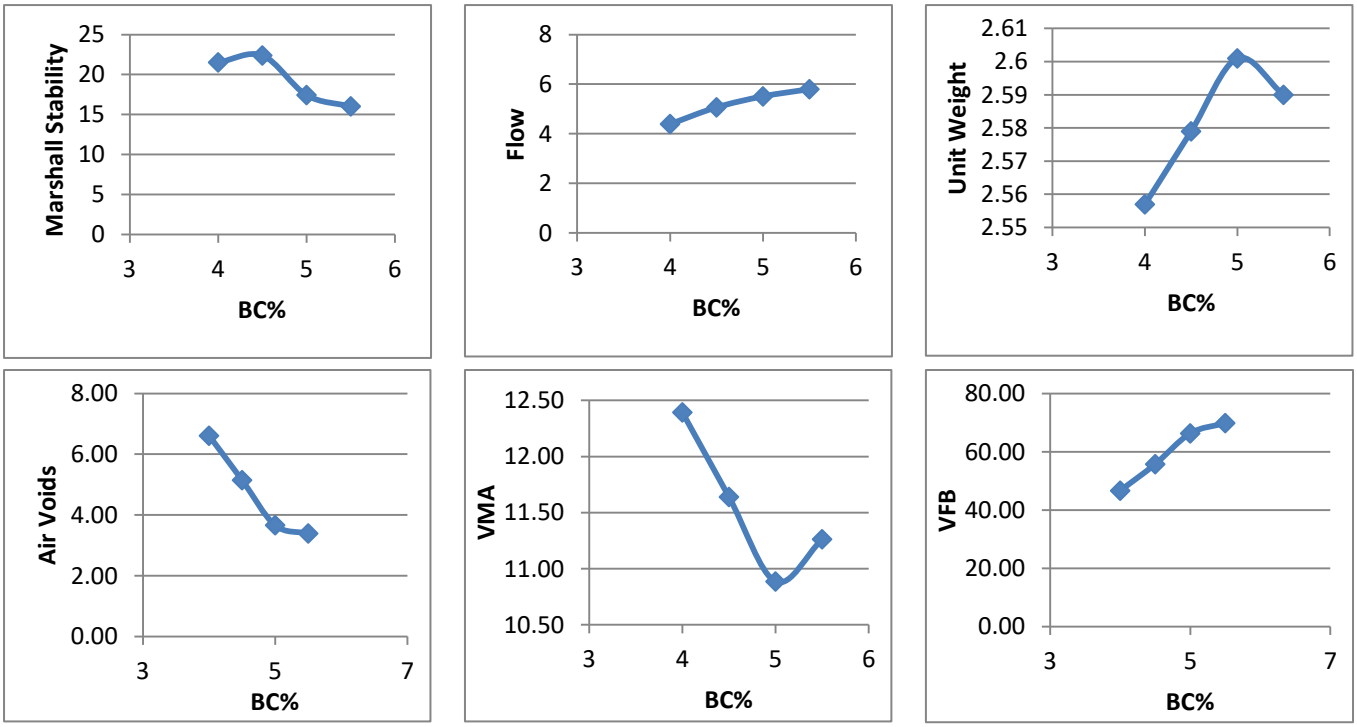

Figure 3.31. Graphs of Marshall Test Result and Volumetric properties for DBM I grade with PMB 40 binder

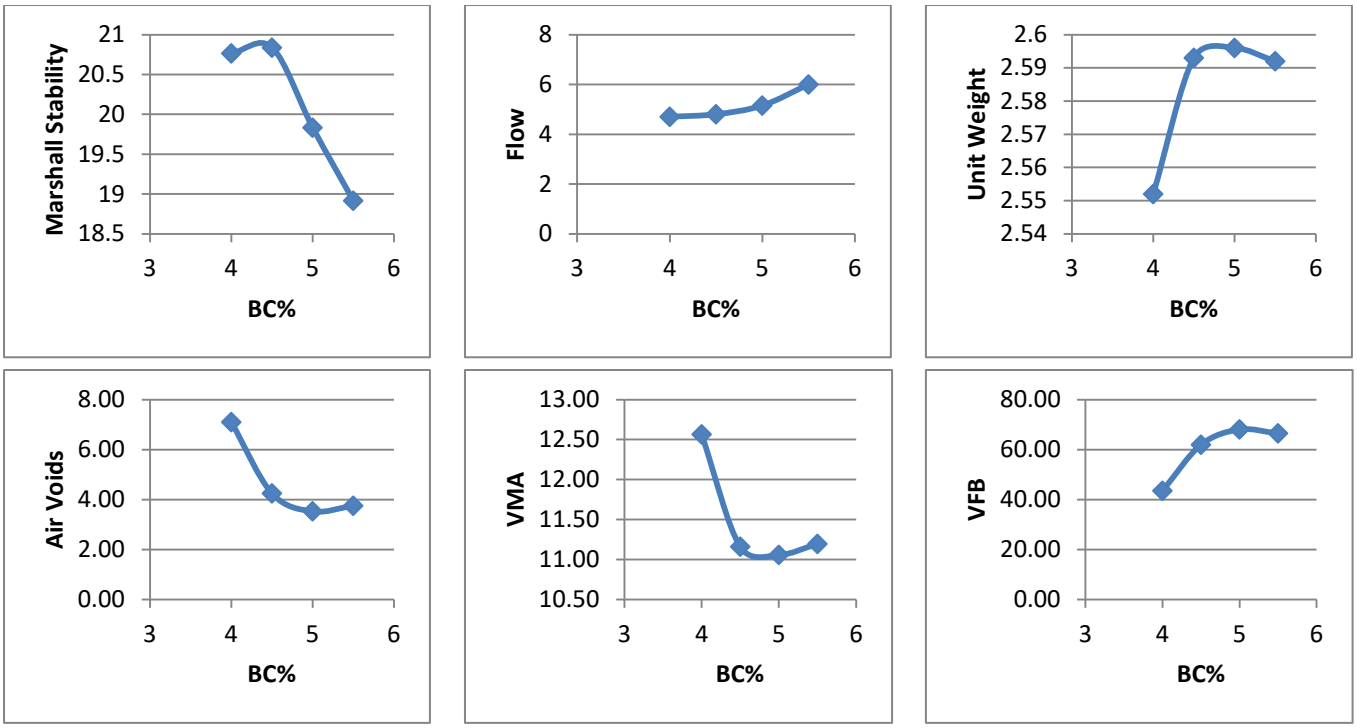

Figure 3.32. Graphs of Marshall Test Result and Volumetric properties for DBM I grade with CRMB 60 binder

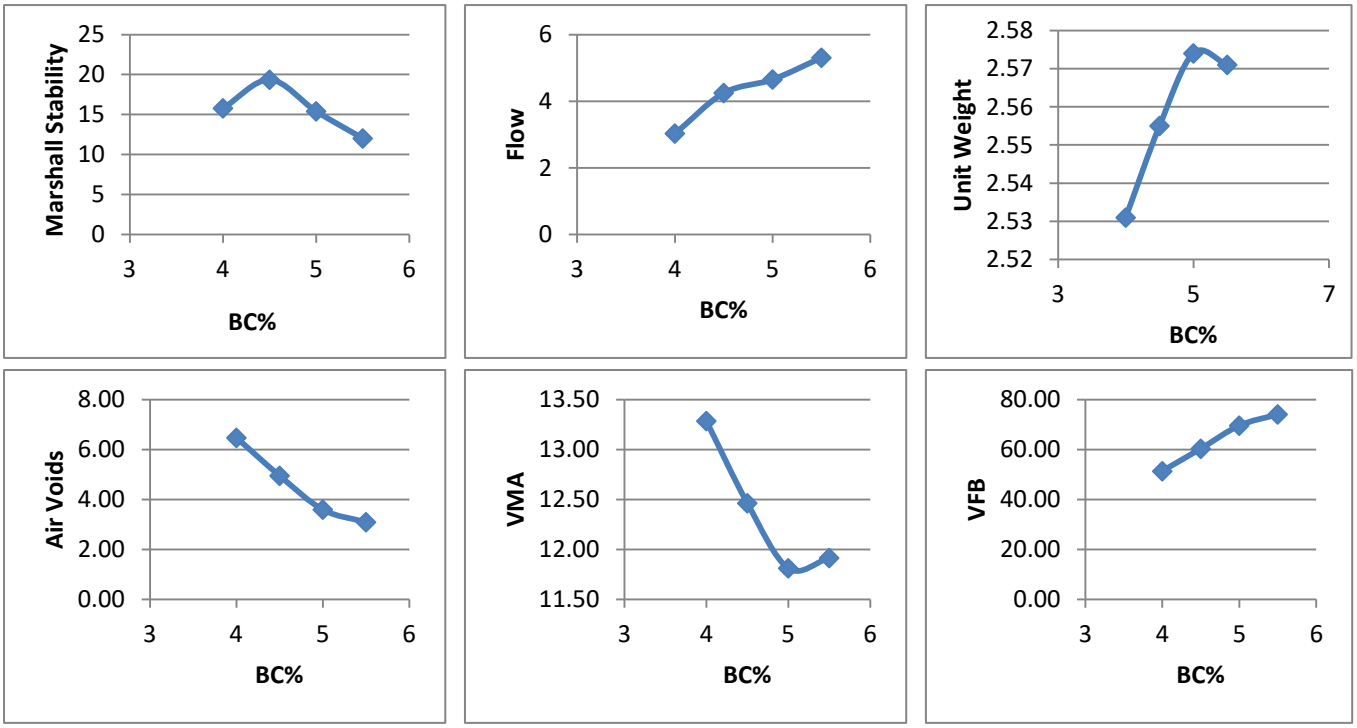

Figure 3.33. Graphs of Marshall Test Result and Volumetric properties for DBM II grade with VG 40 binder

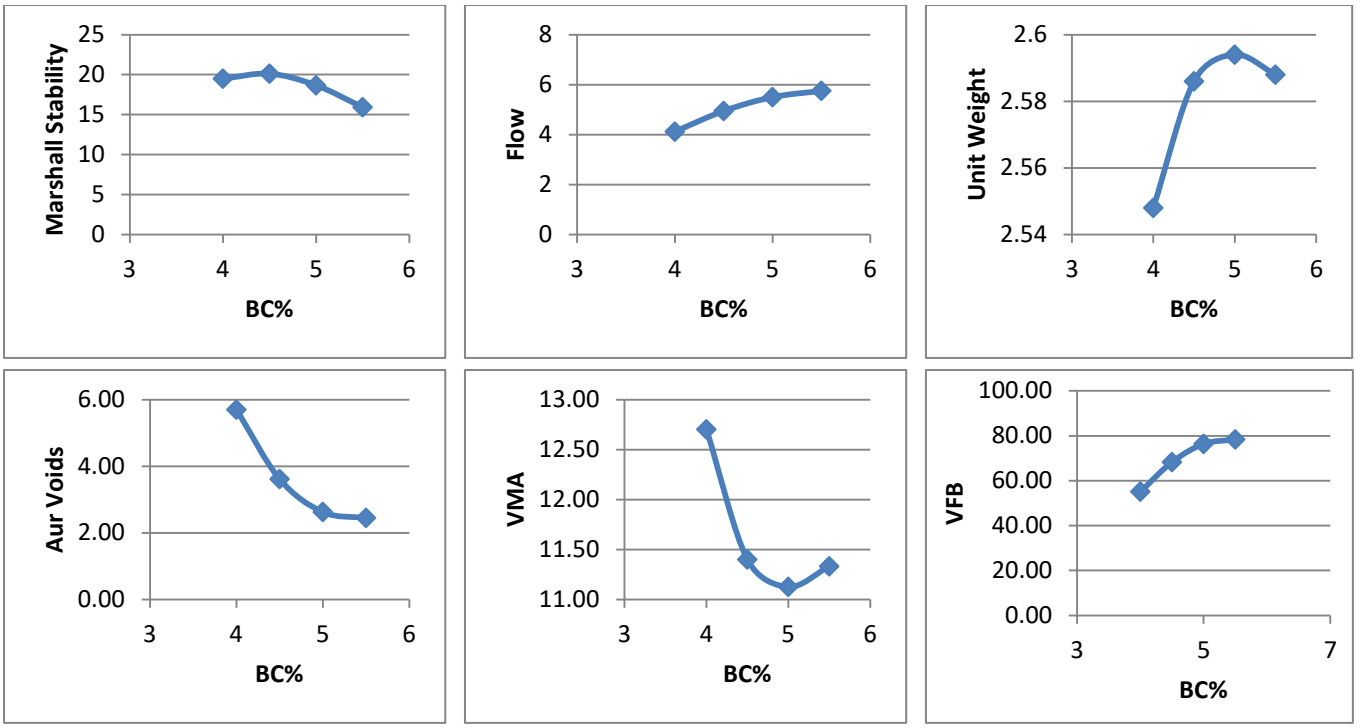

Figure 3.34. Graphs of Marshall Test Result and Volumetric properties for DBM II grade with PMB 40 binder

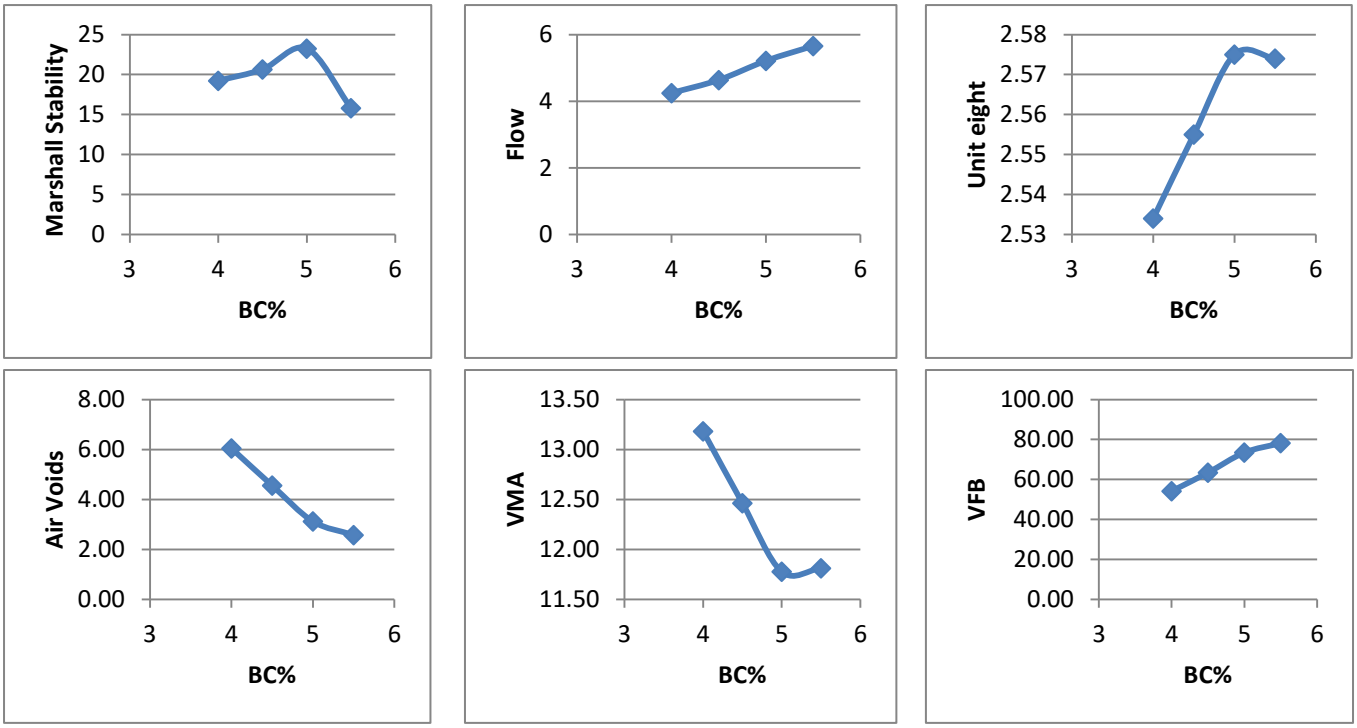

Figure 3.35. Graphs of Marshall Test Result and Volumetric properties for DBM II grade with CRMB 60 binder

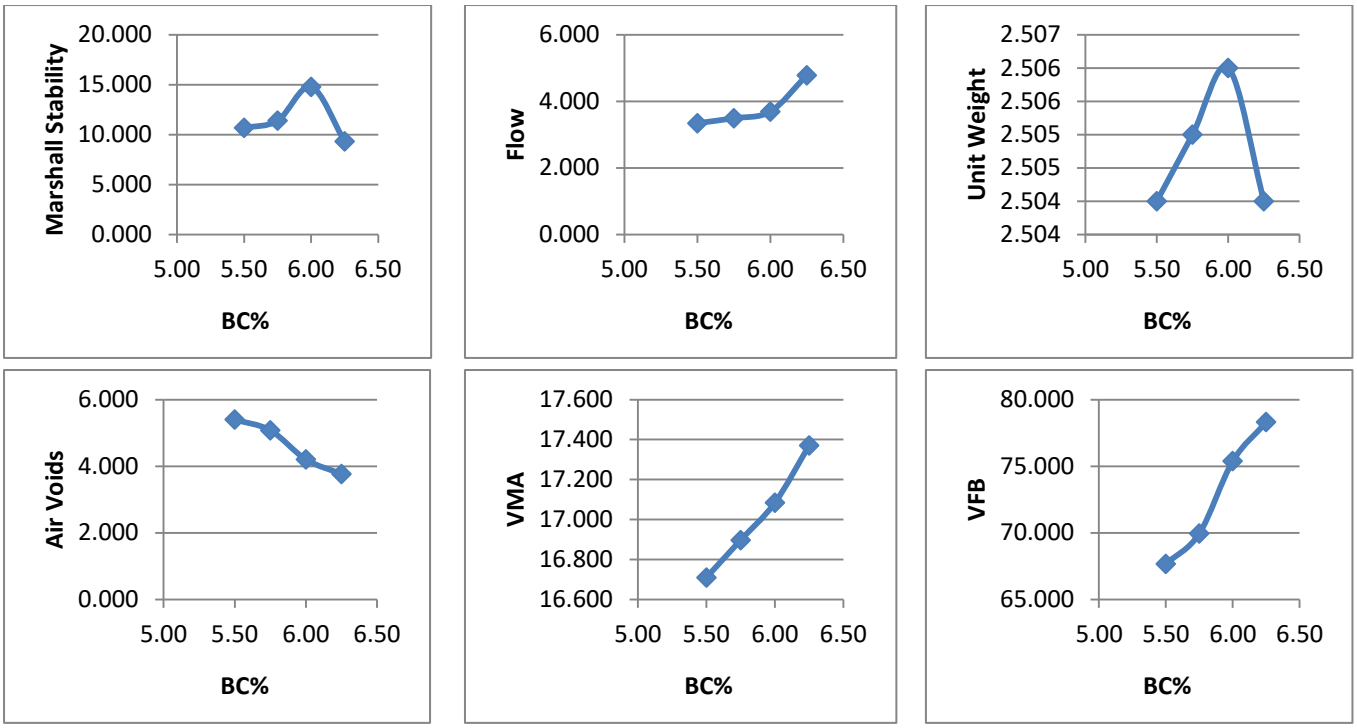

Figure 3.36. Graphs of Marshall Test Result and Volumetric properties for SMA NMA 13 mm grade with VG 40 binder

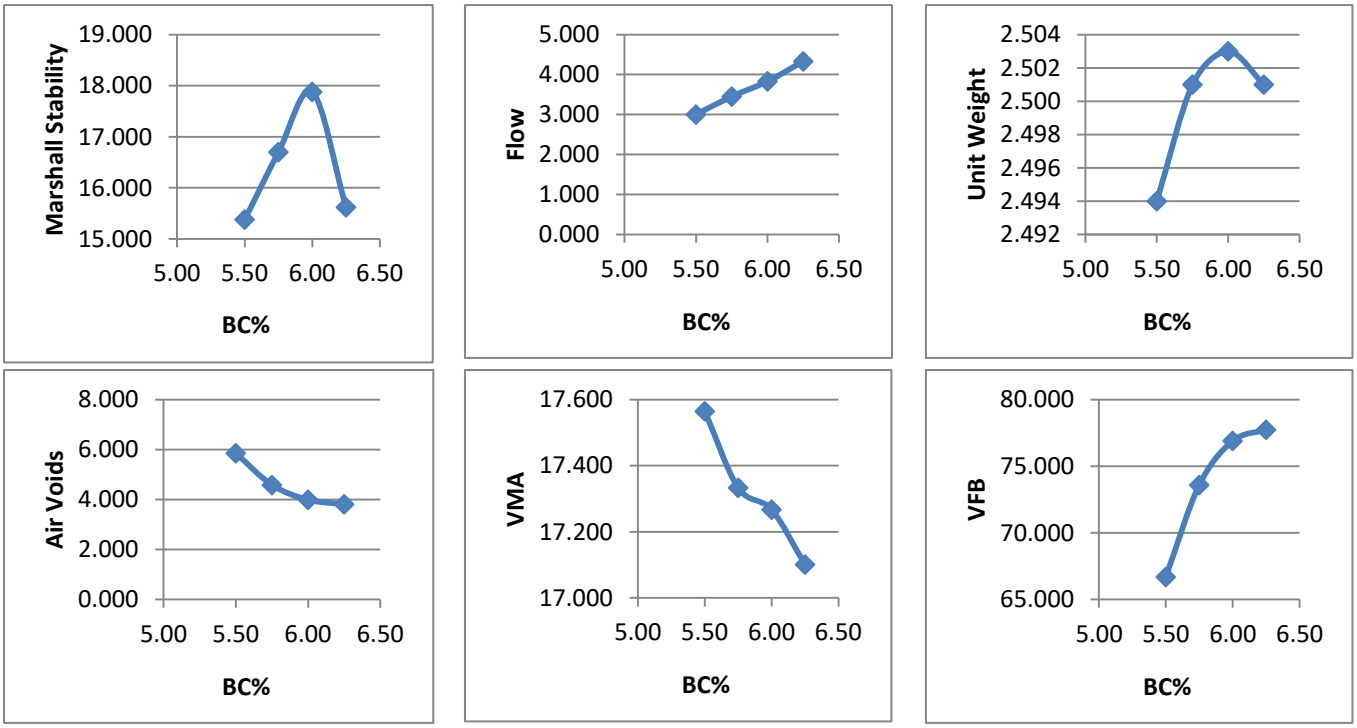

Figure 3.37. Graphs of Marshall Test Result and Volumetric properties for SMA NMA 13 mm grade with PMB 40 binder

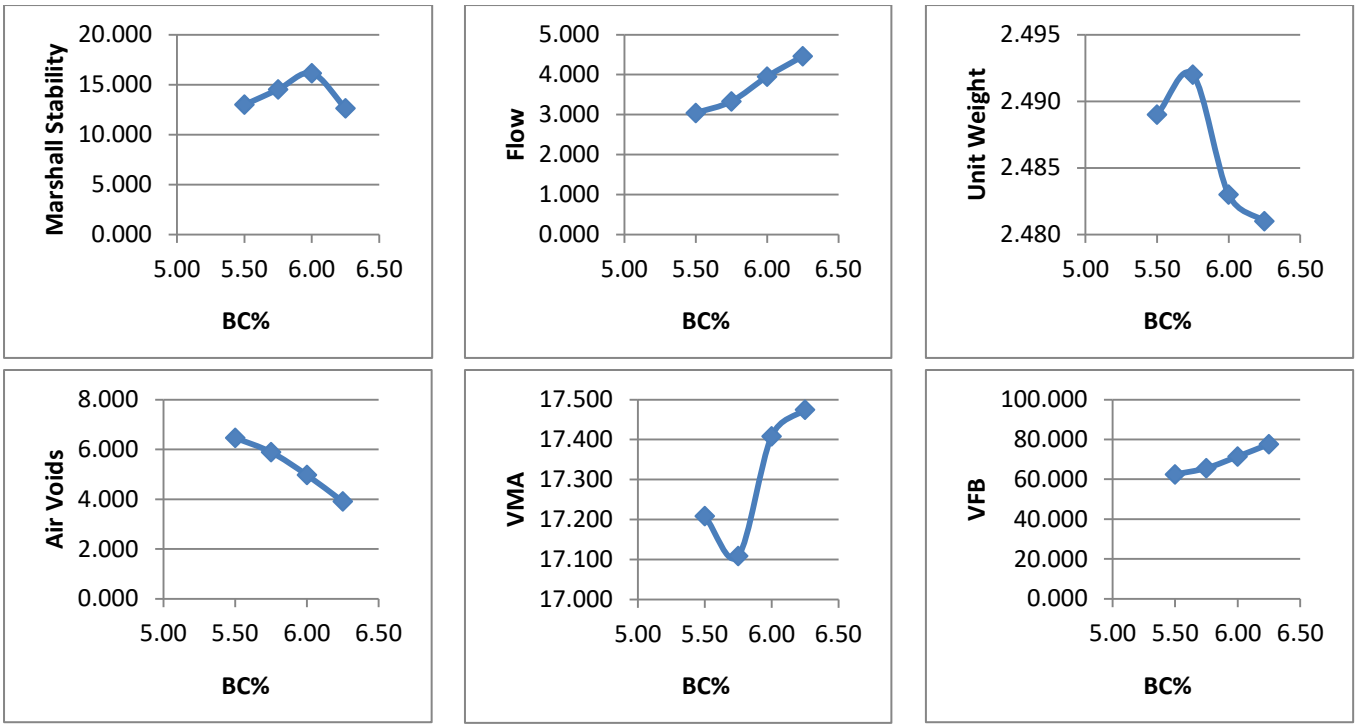

Figure 3.38. Graphs of Marshall Test Result and Volumetric properties for SMA NMA 13 mm grade with CRMB 60 binder

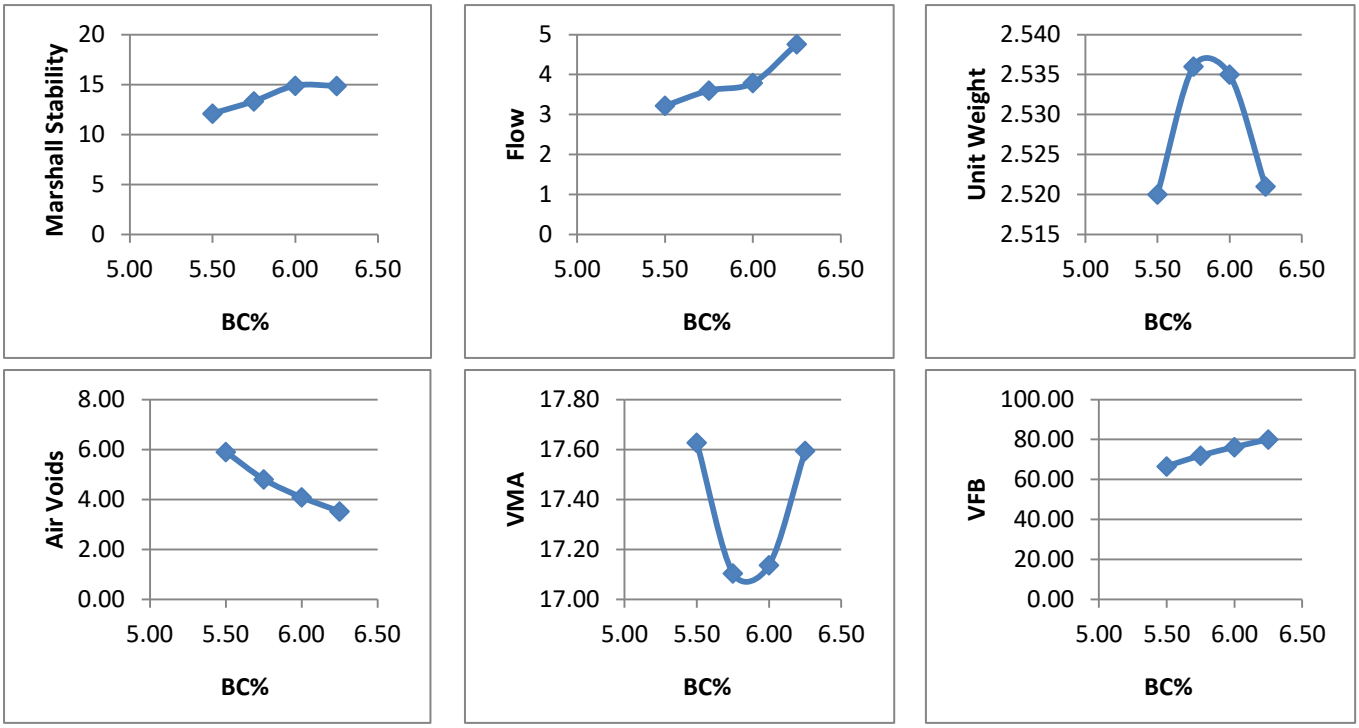

Figure 3.39. Graphs of Marshall Test Result and Volumetric properties for SMA NMA 19 mm grade with VG 40 binder

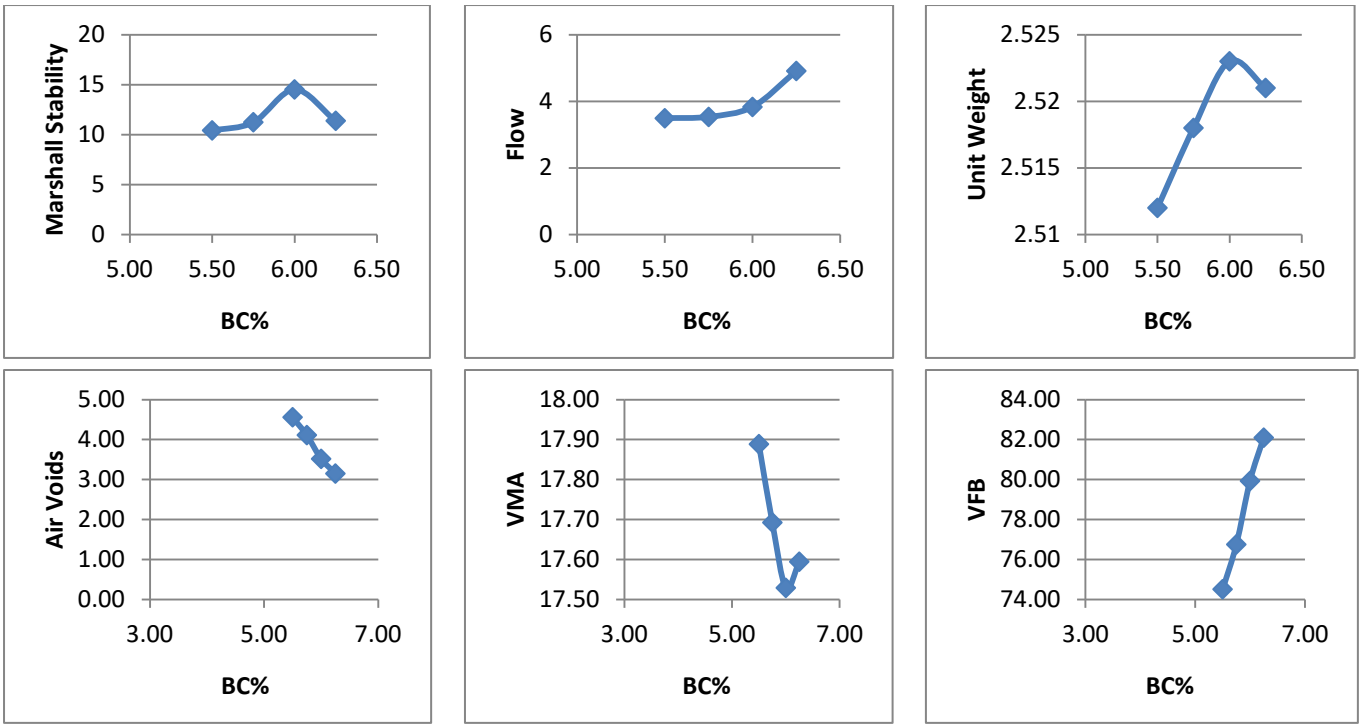

Figure 3.40. Graphs of Marshall Test Result and Volumetric properties for SMA NMA 19 mm grade with PMB 40 binder

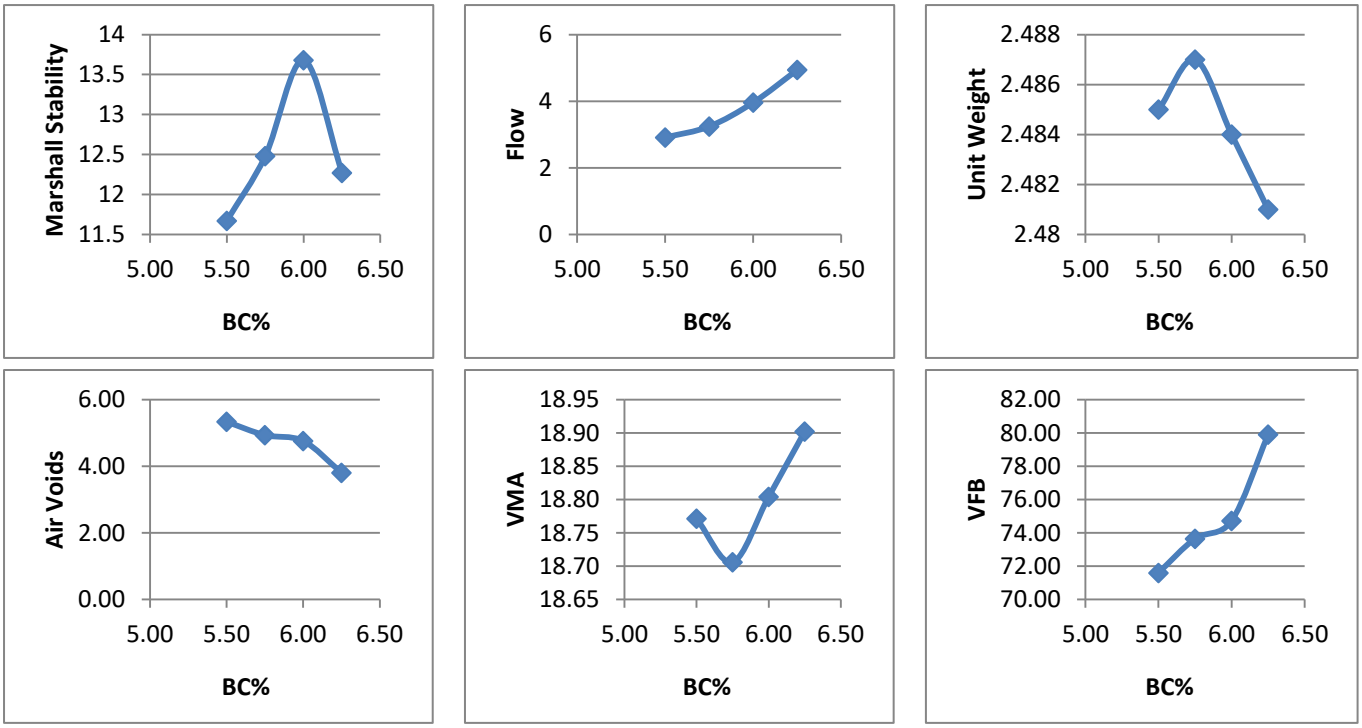

Figure 3.41. Graphs of Marshall Test Result and Volumetric properties for SMA NMA 19 mm grade with CRMB 60 binder

The final results of OBC % and correspnding Marshall test results as well as volumetric properties has been depicted in the table 3.11 for different types of mix.

Table 3.11. Summary of Marshall Test Data and Volumetric Properties Correspond to obtained OBC %

Types of mix @ 4 % air voids	OBC (%)	Bulk Density (g/cc)	Stability (kN)	Flow (mm)	VFB (%)
BC I with VG 40	4.8	2.579	17.274	3.71	69
BC I with PMB 40	4.6	2.689	18.616	3.89	66
BC I with CRMB 60	4.5	2.593	20.065	3.97	70
BC II with VG 40	5.1	2.556	17.831	3.40	67
BC II with PMB 40	5.0	2.574	20.268	4.00	66
BC II with CRMB 60	5.1	2.540	17.239	3.65	69
DBM I with VG 40	4.7	2.594	17.171	3.37	69
DBM I with PMB 40	4.8	2.589	16.344	3.39	66
DBM I with CRMB 60	5.0	2.594	18.833	3.80	67
DBM II with VG 40	4.8	2.564	17.458	3.63	69
DBM II with PMB 40	4.6	2.589	19.162	3.94	69
DBM II with CRMB 60	4.8	2.571	18.222	3.63	73
SMA 13 mm with VG 40	6.0	2.506	14.770	3.69	75
SMA 13 mm with PMB40	6.0	2.503	17.880	3.83	76
SMA 13 mm with CRMB 60	5.9	2.482	16.140	3.95	71
SMA 19 mm with VG 40	6.0	2.535	14.880	3.79	76
SMA 19 mm with PMB 40	5.8	2.522	13.860	3.77	79
SMA 19 mm with CRMB 60	5.9	2.485	13.200	3.67	74

CHAPTER 4 : EXPERIMENTAL INVESTIGATION

This chapter includes the experimental investigation by conducting performance test that is indirect tensile strength test and resilient modulus test performed on the prepared marshall specimen.

4.1 Indirect Tensile Strength Test

The indirect tensile strength (ITS) test is performed as per the ASTM D6931 code on the bituminous cylindrical specimen across its vertical diametral plane at a deformation rate of 50.8 mm per minute and a temperature of 35°C. The peak load at failure is measured and used to compute the value of indirect tensile (IDT) strength. The yield strain is also calculated by using the deformation value of the specimen. The conditioning of specimen is done for minimum 4 hours at a temperture of 35°C before performing the test in an air bath. The IDT strength is calculated with the help of the follwing expression mentioned below,

$$S_t = \frac{2000 \times P}{\pi \times t \times D}$$

Where,

S_t = IDT strength, kPa;

P = peak load, N;

t = specimen height, mm;

D = specimen diameter, mm.

Figure 4.1. Specimen apperarence after ITS test

Figure 4.2. ITS Test Setup

The indirect tensile (IDT) strength data at temperature of 35°C for has been shown in the tabular format for different bituminous mixes i.e., BC Grade I and II, DBM Grade I and II and SMA NMA 13 and 19 mm using three different binder viz.,VG40, PMB40 and CRMB60.

Sample No.	VG 40 @ 35°C					
	BC1	BC2	DBM1	DBM2	SMA 13	SMA 19
1	0.773	0.834	0.818	0.871	0.716	0.691
2	0.755	0.818	0.784	0.790	0.694	0.613
3	0.742	0.808	0.717	0.773	0.677	0.602
4	0.722	0.771	0.662	0.748	0.669	0.577
5	0.677	0.759	0.643	0.724	0.651	0.576
6	0.676	0.759	0.641	0.703	0.587	0.535
7	0.646	0.759	0.638	0.669	0.536	0.513
8	0.604	0.748	0.636	0.633	0.530	0.509
9	0.594	0.733	0.561	0.630	0.505	0.501
10	0.548	0.718	0.553	0.623	0.494	0.488
11	0.541	0.718	0.534	0.584	0.483	0.487
12	0.533	0.649	0.531	0.578	0.448	0.465
13	0.531	0.649	0.519	0.575	0.438	0.459
14	0.529	0.621	0.512	0.492	0.419	0.417

Table 4.1. Indirect tensile strength data at temperature of 35 degree celcius for various grade of mix with VG 40 binder

Sample No.	PMB 40 @ 35°C					
	BC1	BC2	DBM1	DBM2	SMA 13	SMA 19
1	1.584	1.158	0.959	1.175	0.901	0.824
2	1.370	1.042	0.903	1.111	0.843	0.791
3	1.176	1.040	0.888	1.016	0.828	0.778
4	1.052	0.993	0.825	0.992	0.789	0.762
5	1.031	0.993	0.813	0.983	0.782	0.727
6	0.980	0.921	0.805	0.947	0.779	0.715
7	0.896	0.889	0.794	0.946	0.769	0.701
8	0.872	0.870	0.771	0.922	0.748	0.677
9	0.827	0.865	0.752	0.890	0.705	0.673
10	0.809	0.822	0.751	0.840	0.675	0.666
11	0.753	0.764	0.737	0.815	0.658	0.663
12	0.732	0.755	0.648	0.783	0.612	0.640
13	0.719	0.704	0.640	0.771	0.575	0.592
14	0.697	0.676	0.609	0.764	0.559	0.586

Table 4.2. Indirect tensile strength data at temperature of 35 degree celcius for various grade of mix with PMB 40 binder

Sample No.	CRMB 60 @ 35°C					
	BC1	BC2	DBM1	DBM2	SMA 13	SMA 19
1	0.964	0.882	0.923	1.082	0.768	0.841
2	0.943	0.880	0.919	0.960	0.748	0.804
3	0.907	0.855	0.881	0.950	0.738	0.804
4	0.893	0.849	0.836	0.927	0.700	0.789
5	0.851	0.760	0.803	0.907	0.640	0.777
6	0.836	0.759	0.787	0.847	0.608	0.766
7	0.817	0.755	0.743	0.833	0.604	0.759
8	0.809	0.740	0.721	0.831	0.593	0.755
9	0.808	0.712	0.686	0.809	0.572	0.750
10	0.776	0.704	0.685	0.809	0.558	0.656
11	0.690	0.696	0.677	0.790	0.554	0.652
12	0.690	0.674	0.622	0.788	0.548	0.599
13	0.678	0.639	0.617	0.788	0.518	0.586
14	0.648	0.632	0.613	0.762	0.513	0.550

Table 4.3. Indirect tensile strength data at temperature of 35 degree celcius for various grade of mix with CRMB 60 binder

4.2 Resilient Modulus Test

The resilient modulus (M_r) test is performed as per the ASTM D4123 code on the bituminous cylindrical specimen at temperature of mainly 35°C provided with the test input of peak load as 10 % of the ITS maximum load and the contact load as 4% of peak load. The loading time of 0.1 second and rest period of 0.9 second is also feeded as input. The conditioning of sample in the universal testing machine (UTM) chamber was done for duration of 6 hours before the start of test.

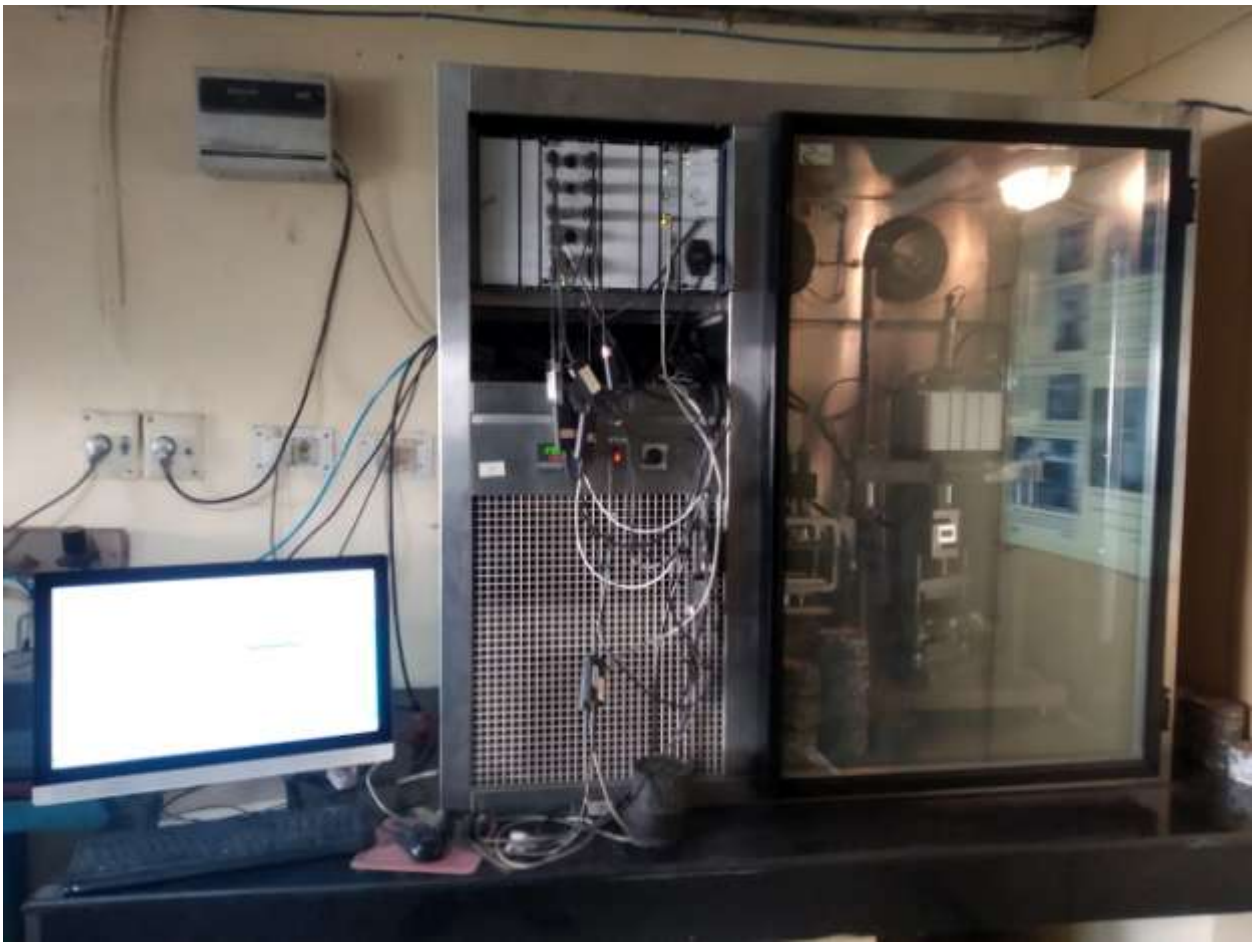

Figure 4.3. Universal Testing Machine (UTM) Setup for performing Mr test

Figure 4.4. Resilient Modulus Testing Apparatus

Figure 4.5. Operation of UTM system for Mr testing

After the preconditioning cycle of 100,the mean of last 5 cycle of test should be noted as resilient modulus value. The resilient modulus value was determined at different percentage of binder as well as at the calculated OBC % content is shown in table 4.4 to 4.6 .

Sample No.	VG 40 @ 35°C					
	BC1	BC2	DBM1	DBM2	SMA 13	SMA 19
1	1944	1899	2075	2000	1712	1760
2	1927	1883	1931	1748	1711	1707
3	1906	1871	1921	1731	1707	1694
4	1899	1860	1917	1730	1705	1518
5	1878	1857	1906	1686	1701	1482
6	1800	1841	1760	1672	1697	1451
7	1619	1839	1730	1661	1692	1435
8	1532	1757	1645	1656	1685	1397
9	1423	1751	1472	1655	1684	1335
10	1416	1740	1412	1627	1680	1259
11	1257	1397	1410	1599	1672	1234
12	1252	1360	1404	1594	1547	1210
13	1198	1316	1404	1567	1513	1204
14	1072	1206	1388	1558	1492	1201

Table 4.4. Mr value at 35 °C for various grade of mix with VG 40 binder

Sample No.	PMB 40 @ 35°C					
	BC1	BC2	DBM1	DBM2	SMA 13	SMA 19
1	2100	2336	1916	2616	2227	1835

2	1862	2097	1889	2515	2145	1587
3	1849	1996	1888	2458	2140	1383
4	1663	1967	1811	2441	2132	1540
5	1649	1953	1799	2438	2129	1373
6	1645	1876	1733	2377	2127	1684
7	1643	1634	1673	2371	2052	1554
8	1638	1622	1639	2343	2046	1842
9	1541	1615	1632	1895	1985	1826
10	1432	1610	1624	1882	1975	1597
11	1395	1574	1600	1878	1972	1599
12	1389	1474	1530	1868	1963	1577
13	1388	1324	1462	1863	1948	1586
14	1382	1227	1253	1835	1917	1578

Table 4.5. Mr value at 35 °C for various grade of mix with PMB 40 binder

Sample No.	CRMB 60 @ 35°C					
	BC1	BC2	DBM1	DBM2	SMA 13	SMA 19
1	1692	1485	1785	1643	1684	1387
2	1594	1484	1559	1699	1670	1378
3	1527	1481	1512	1689	1652	1378
4	1513	1398	1375	1687	1644	1377
5	1505	1387	1374	1685	1641	1324
6	1409	1384	1369	1673	1638	1221
7	1200	1379	1363	1655	1636	1185
8	1170	1378	1219	1623	1618	1172
9	1163	1373	1215	1607	1518	1147
10	1160	1361	1214	1603	1435	1142
11	1155	1287	1210	1601	1268	1142
12	1155	939	1208	1516	1262	1141
13	1093	934	1195	1486	1259	1132
14	1038	843	1169	1135	1255	1395

Table 4.6. Mr value at 35 °C for various grade of mix with CRMB 60 binder

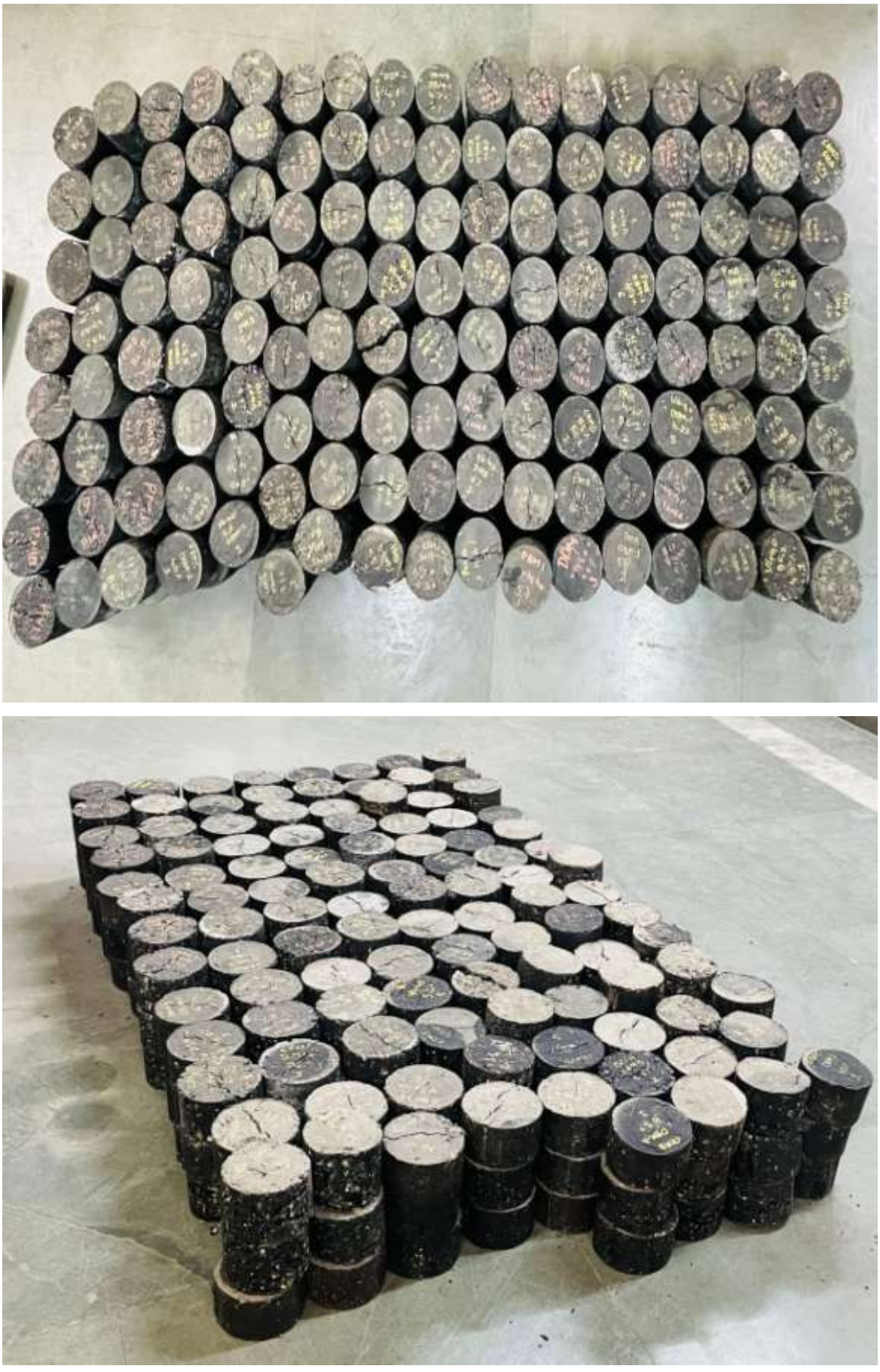

Figure 4.6. Total Samples prepared during reseach work

CHAPTER 5 : ANALYSIS AND VALIDATION OF RESULTS

This chapter includes the analysis of ITS test data, which is used to correlate with the modulus of resilience value obtained from the laboratory testing.

5.1 Analysis of ITS Test Data

The analysis of ITS test data has been carried out for 14 individual samples of each grade i.e., BC1, BC2, DBM1, DBM2, SMA NMA 13 mm, SMA NMA 19 mm with three different types of binder viz.,VG40, PMB40, CRMB60 in the Microsoft Office Excel software. The following steps were followed to analyse the data;

STEP 1 : The obtained ITS test data consists of load and deformation values, would be used for the computation of ITS strength and lateral strain values respectively using the formula mentioned below,

$$S_t = \frac{2000 \times P}{\pi \times t \times D}$$

$$\varepsilon_l = \frac{\Delta D}{D}$$

Where,

S_t = IDT strength, kPa;

P = peak load, N;

t = specimen height, mm;

D = specimen diameter, mm;

ε_l = Lateral strain, unitless;

ΔD = change in specimen diameter,i.e., deformation value,mm.

STEP 2 : The ITS strength and lateral strain data was taken upto 10% of the yield strain value corresponding to the peak load at failure.

STEP 3 : The graph plotting for 10% of the data has to be done between ITS strength along the Y-axis and lateral strain along the X-axis. The trendline need to be observed using the

two degree polynomial equations and the coefficient of determination. Also, the reason behind the consideration of only 10% of data was to get the constant slope within the linear elastic region of the stress versus strain curve.

STEP 4 : The 10% of the analysed data for each 14 individual samples were compiled together and scattered on a graph for all 18 mix types supported with a two degree polynomial equation and the coefficient of determination which are presented in figure 5.1 to 5.18.

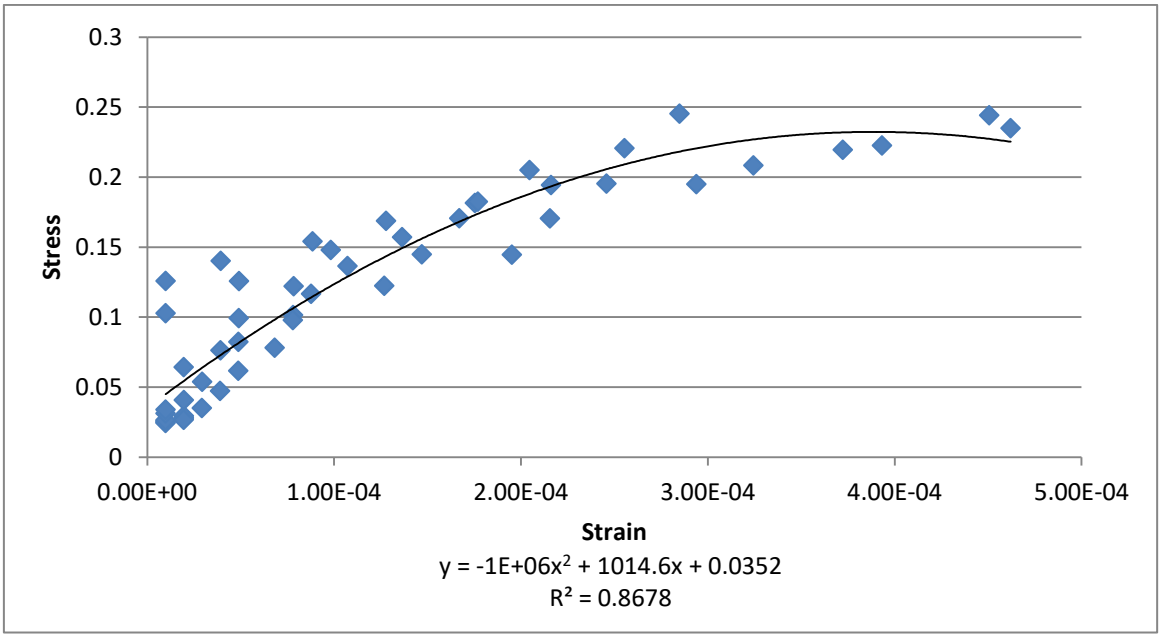

Figure 5.1. Analysed dataset for BC1 with VG40

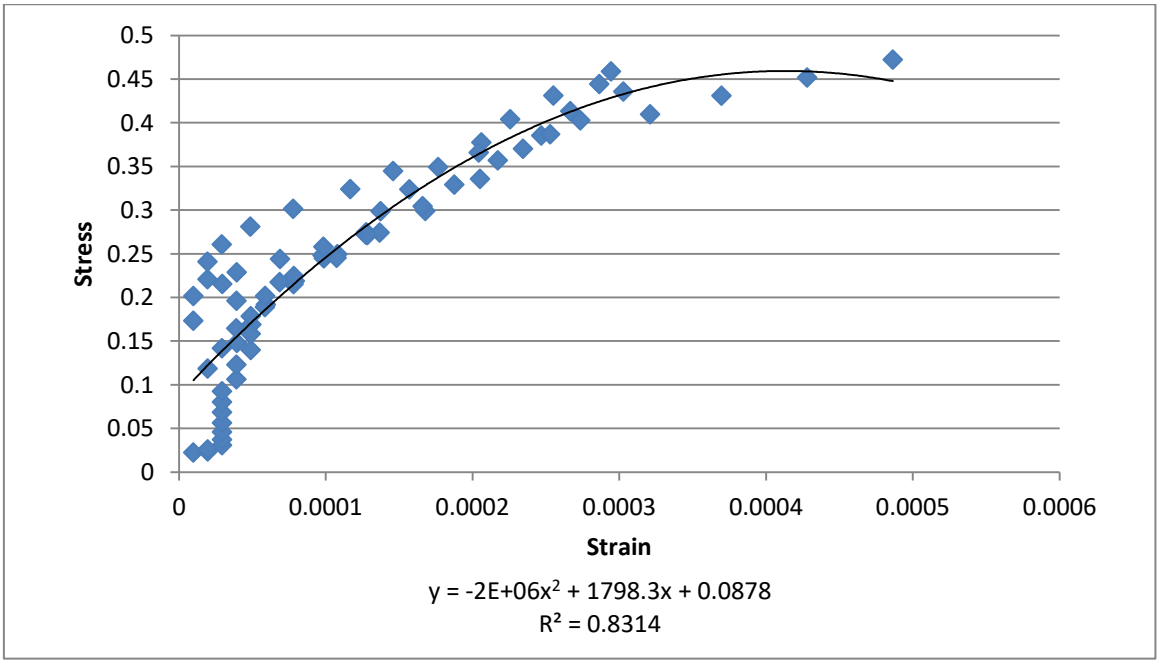

Figure 5.2. Analysed dataset for BC1 with PMB40

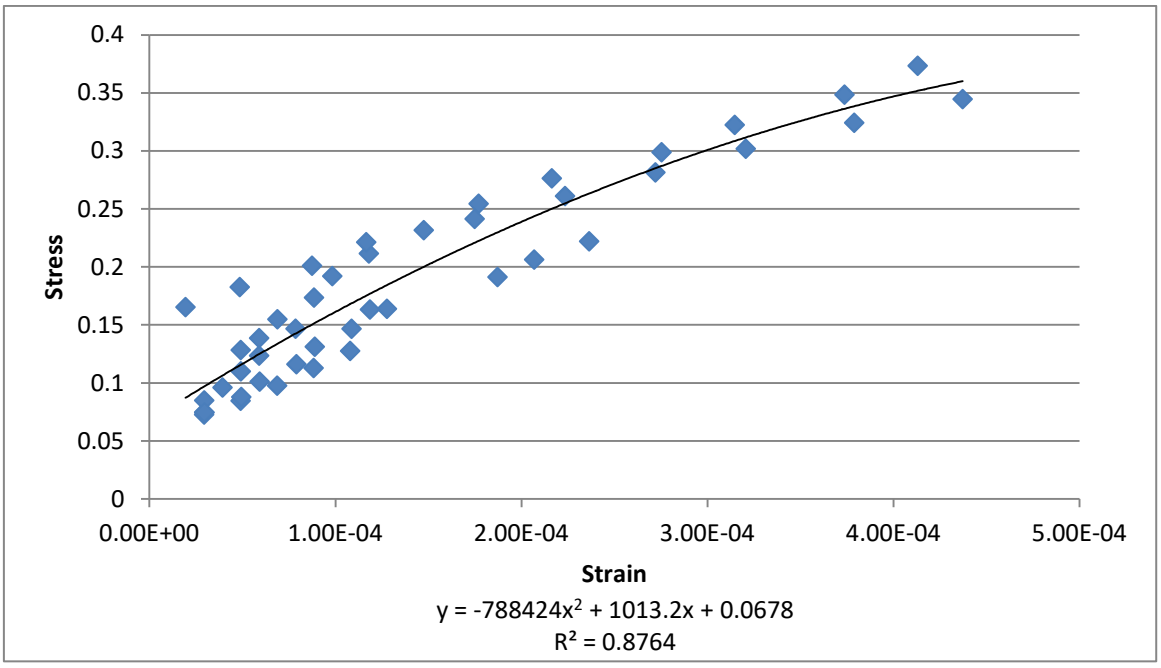

Figure 5.3. Analysed dataset for BC1 with CRMB60

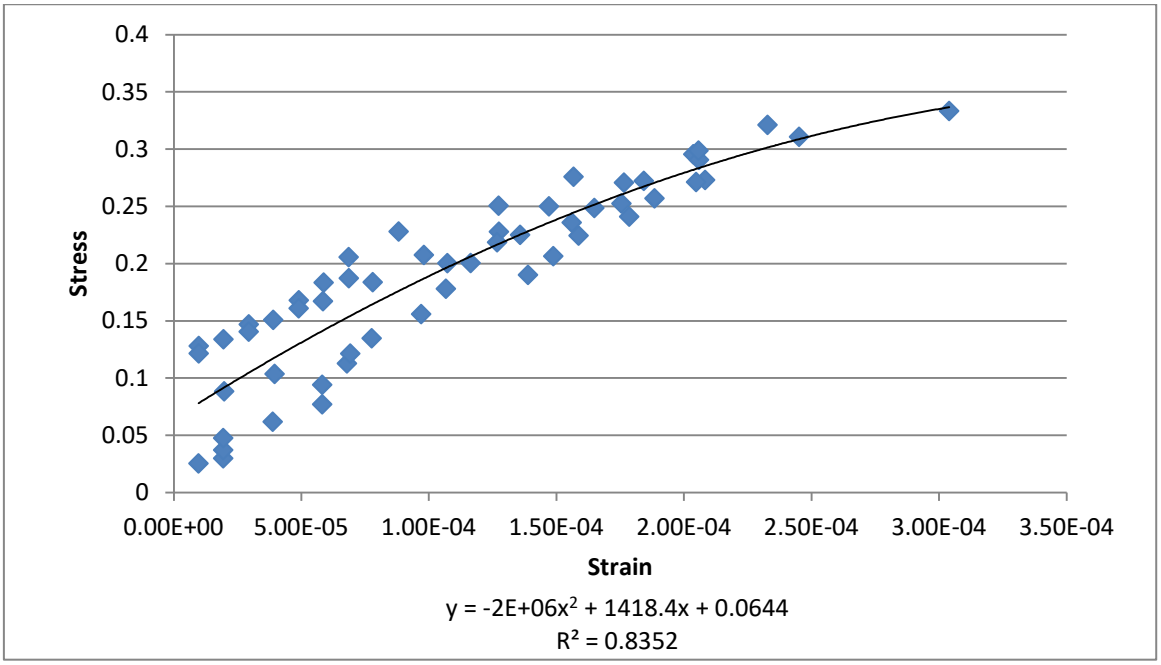

Figure 5.4. Analysed dataset for BC2 with VG40

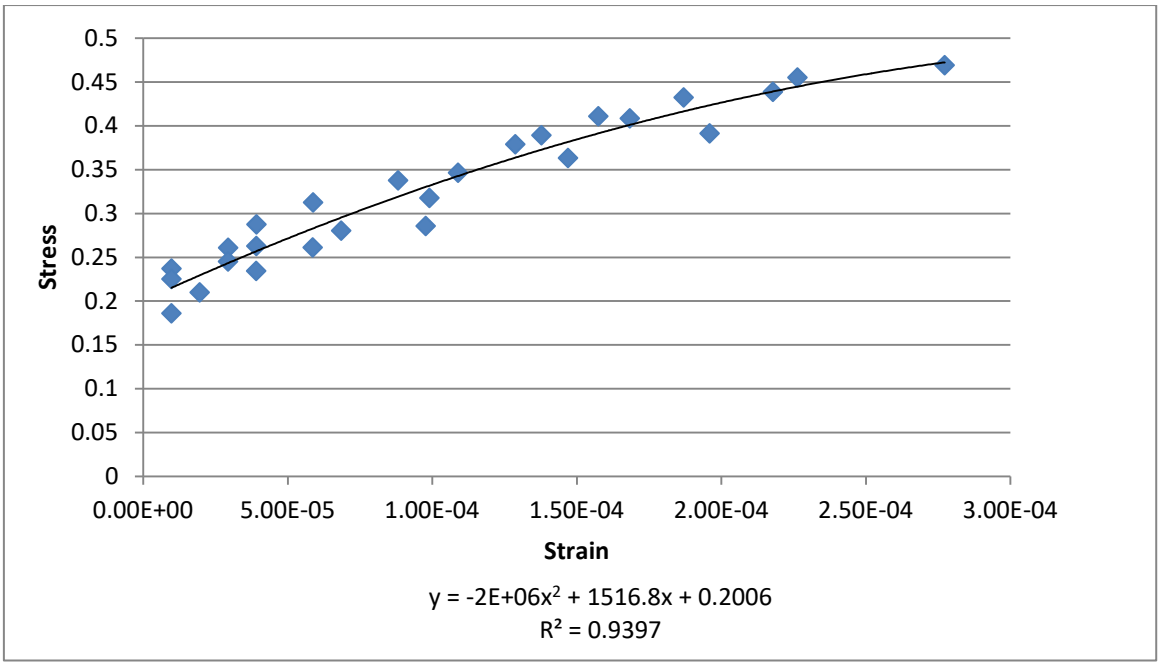

Figure 5.5. Analysed dataset for BC2 with PMB40

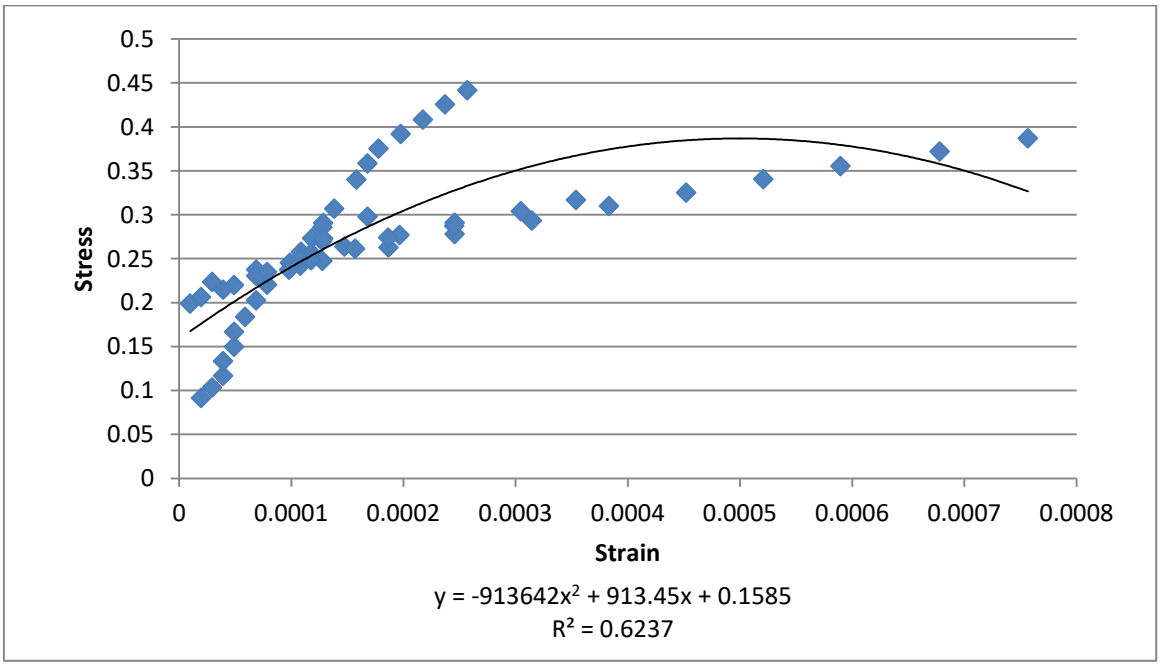

Figure 5.6. Analysed dataset for BC2 with CRMB60

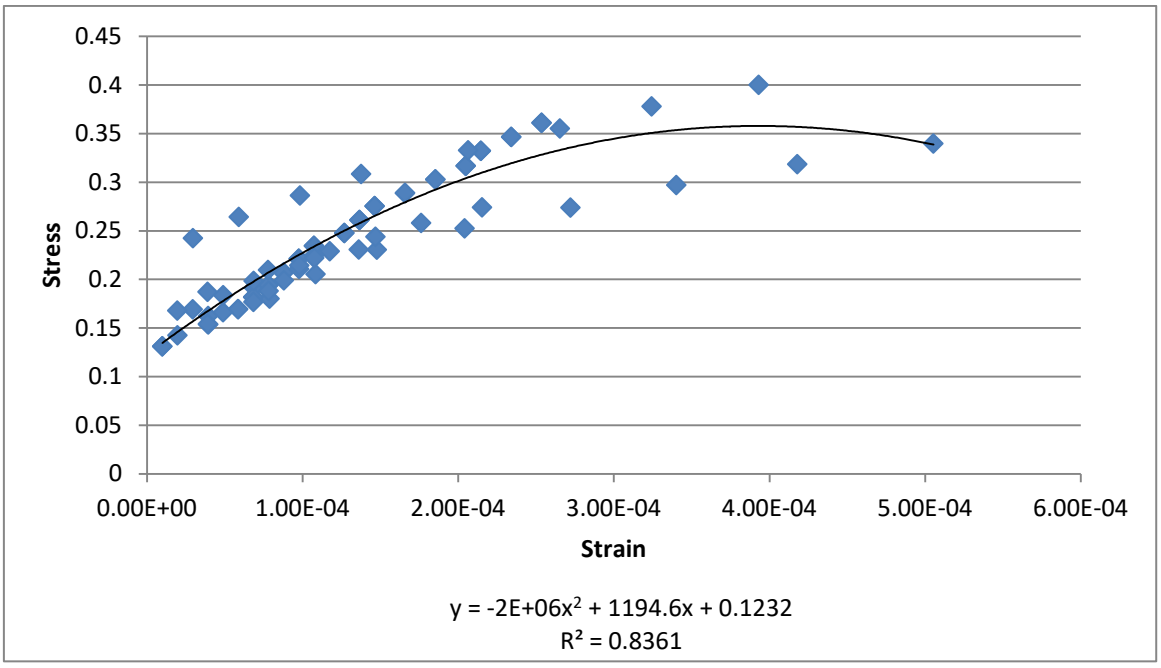

Figure 5.7. Analysed dataset for DBM1 with VG40

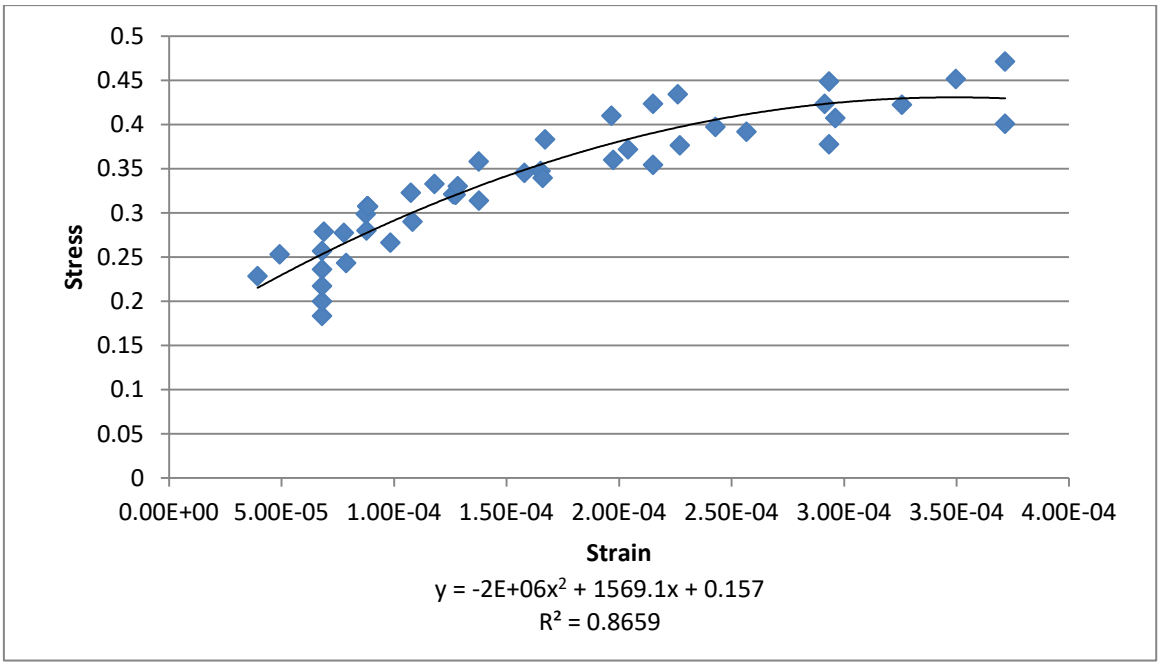

Figure 5.8. Analysed dataset for DBM1 with PMB40

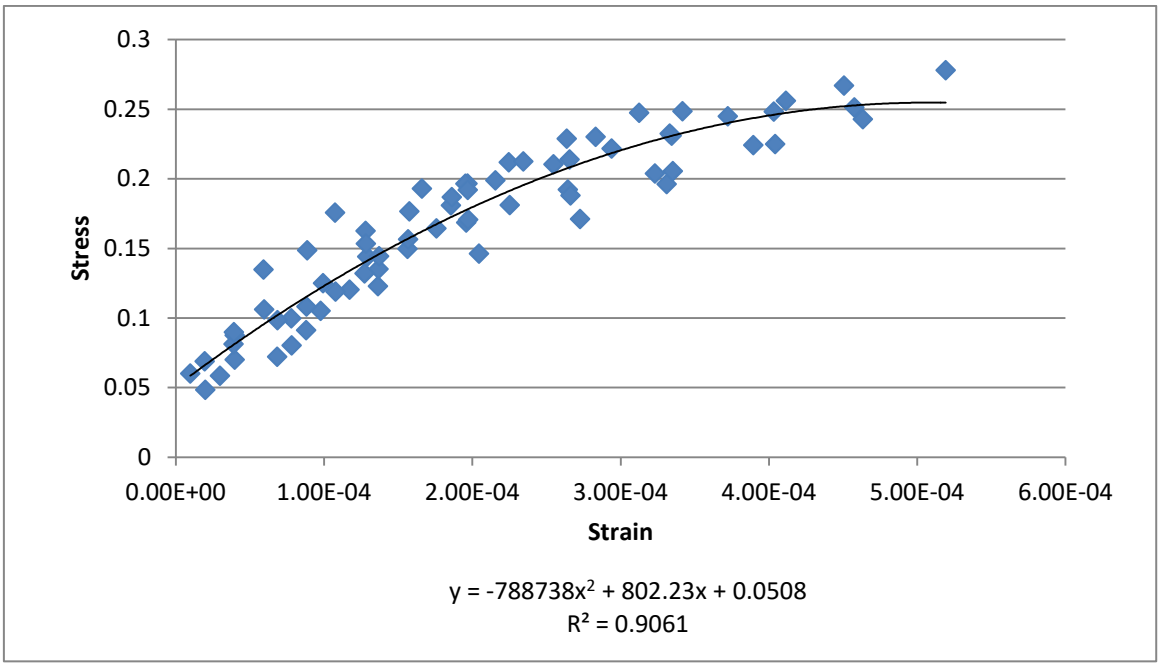

Figure 5.9. Analysed dataset for DBM1 with CRMB60

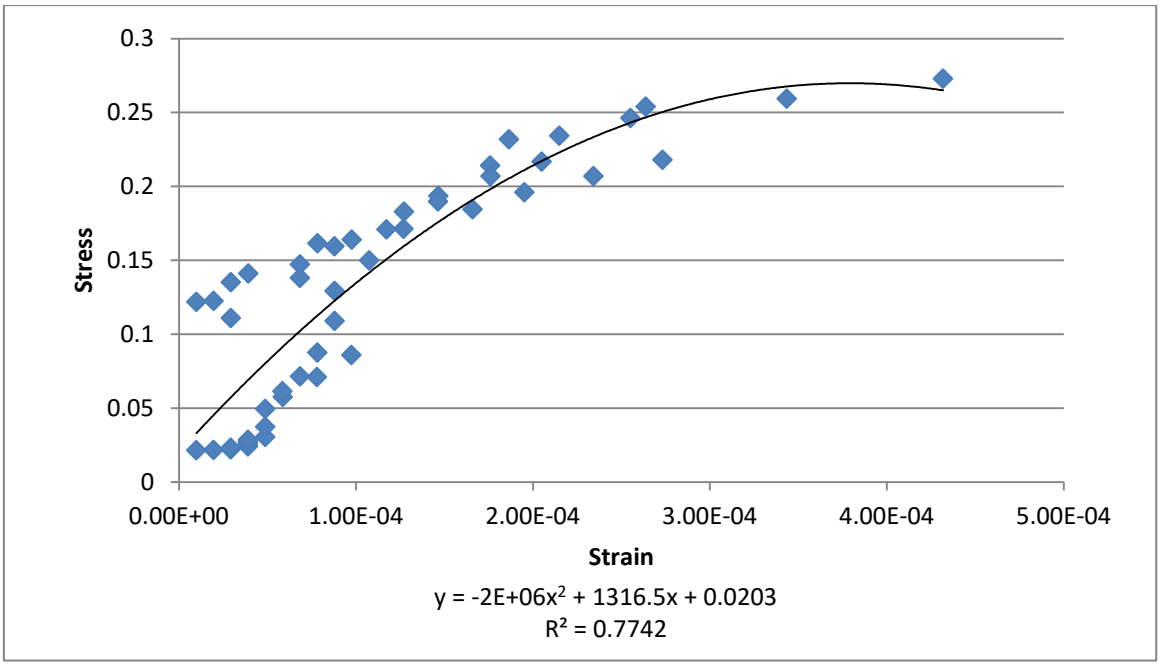

Figure 5.10. Analysed dataset for DBM2 with VG40

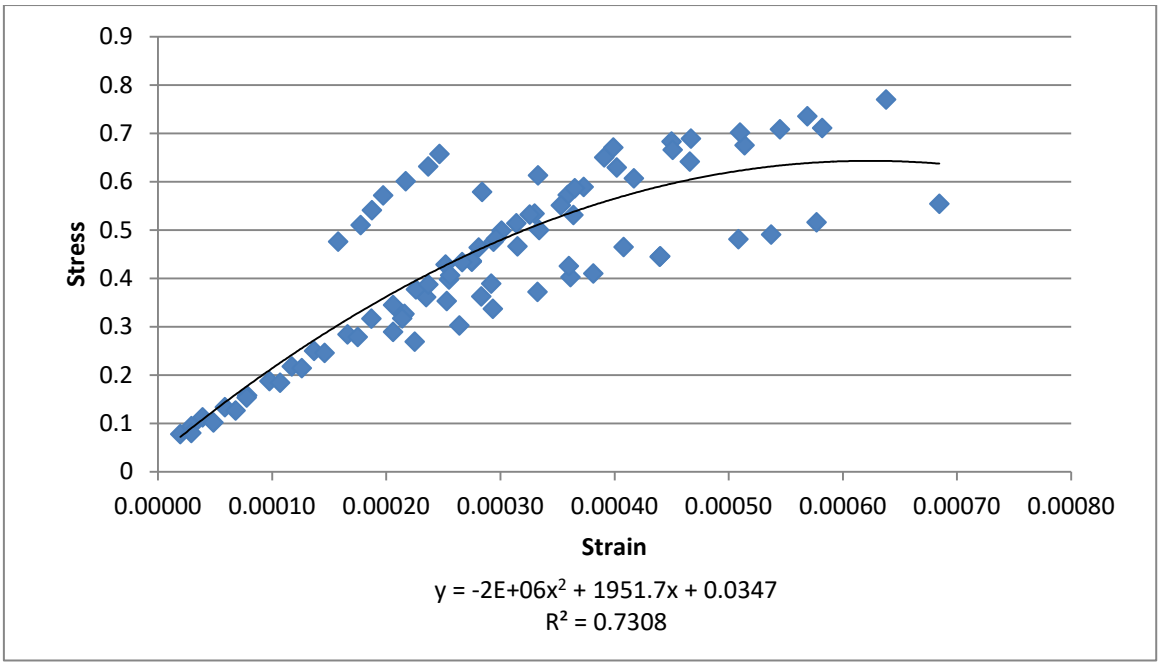

Figure 5.11. Analysed dataset for DBM2 with PMB40

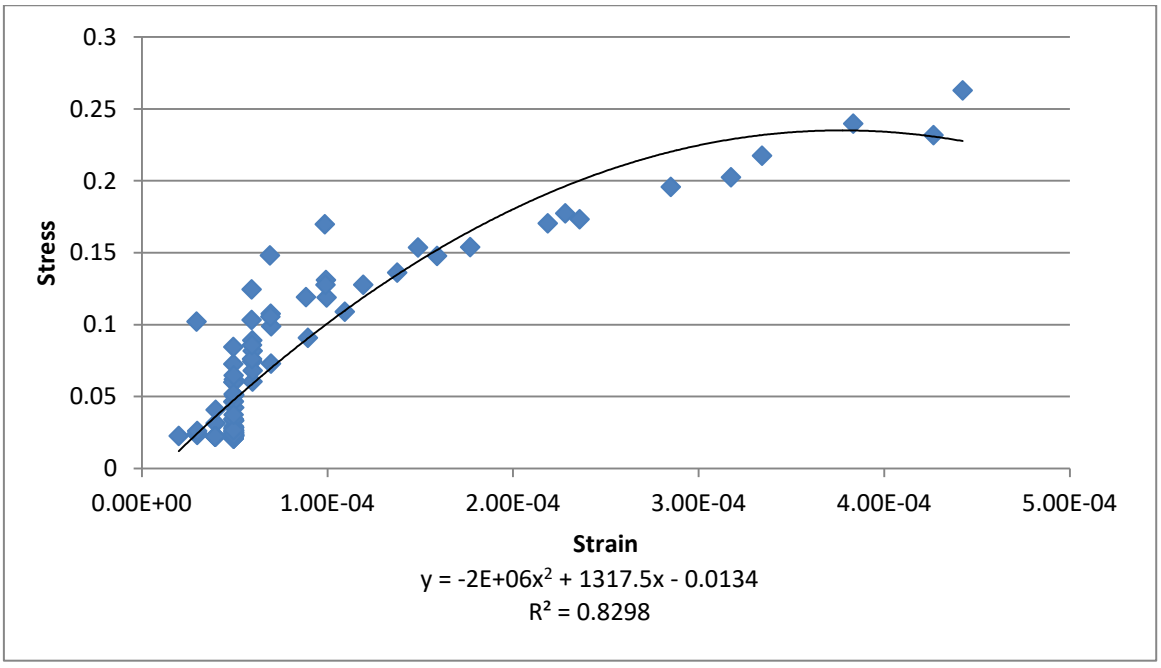

Figure 5.12. Analysed dataset for DBM2 with CRMB60

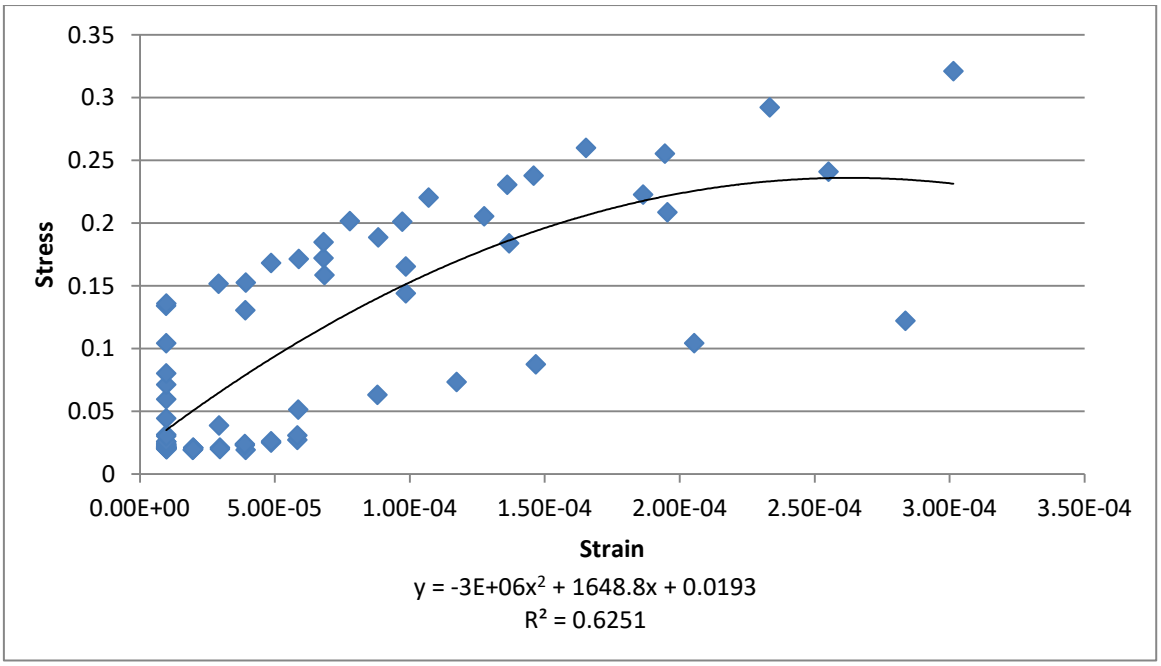

Figure 5.13. Analysed dataset for SMA NMA 13 mm with VG40

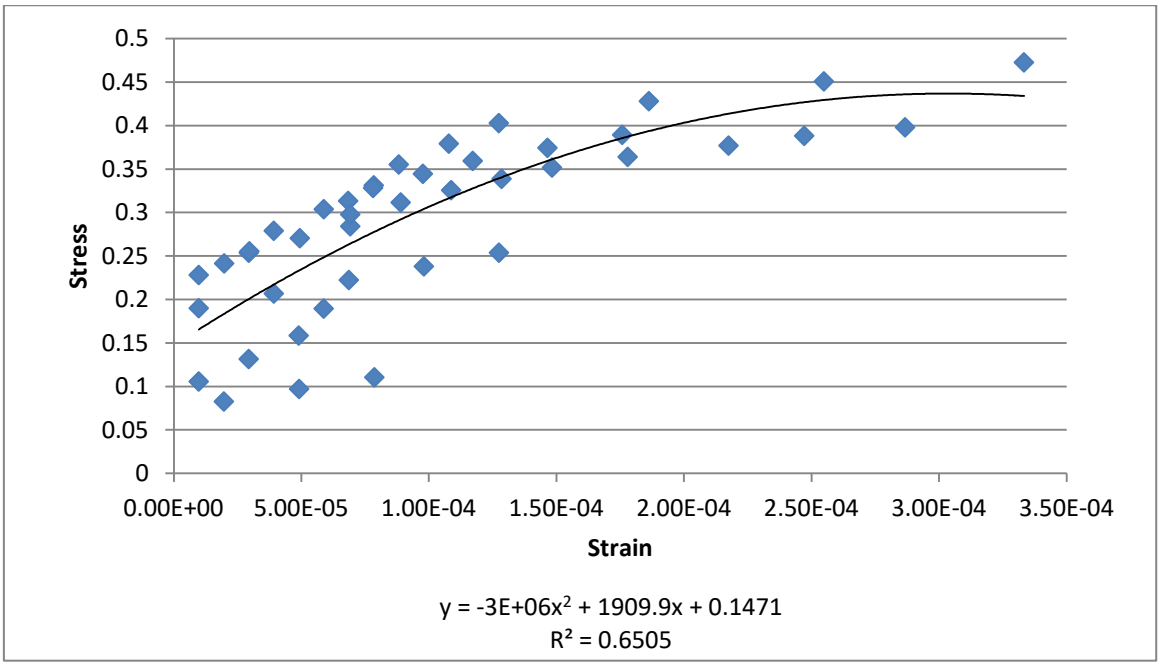

Figure 5.14. Analysed dataset for SMA NMA 13 mm with PMB40

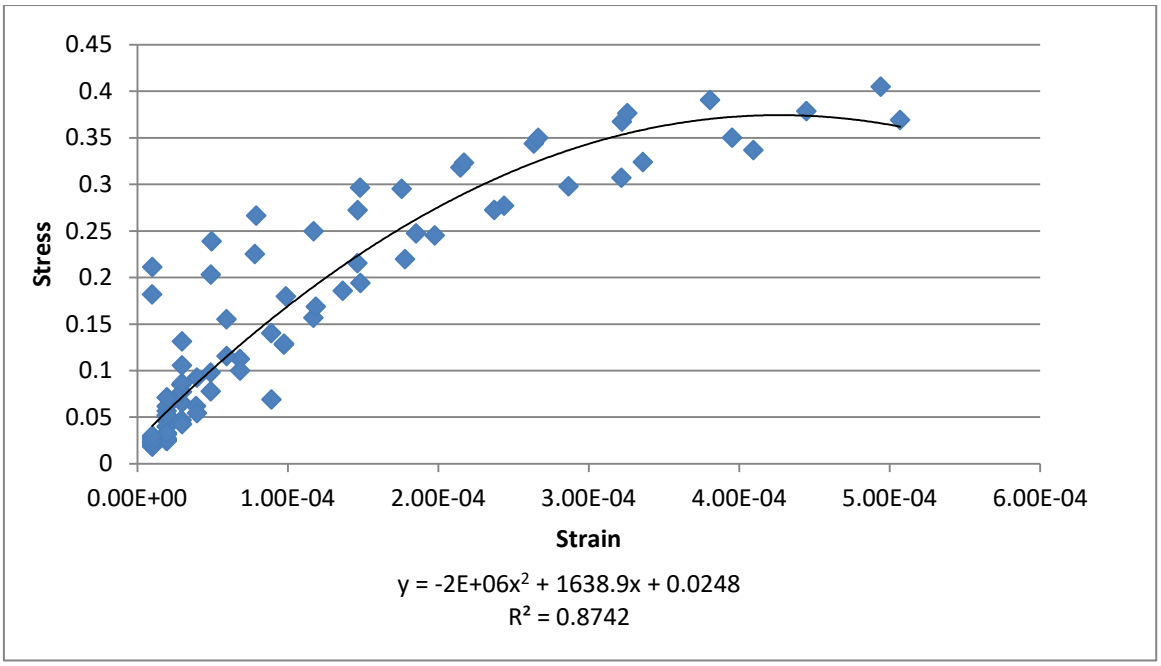

Figure 5.15. Analysed dataset for SMA NMA 13 mm with CRMB60

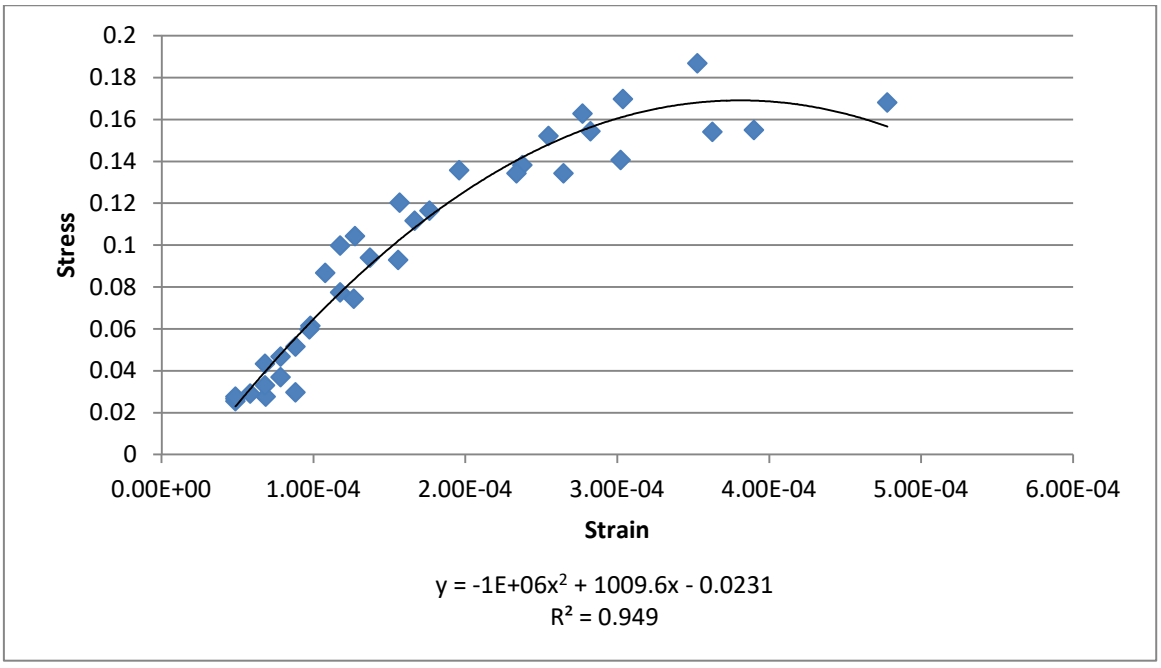

Figure 5.16. Analysed dataset for SMA NMA 19 mm with VG40

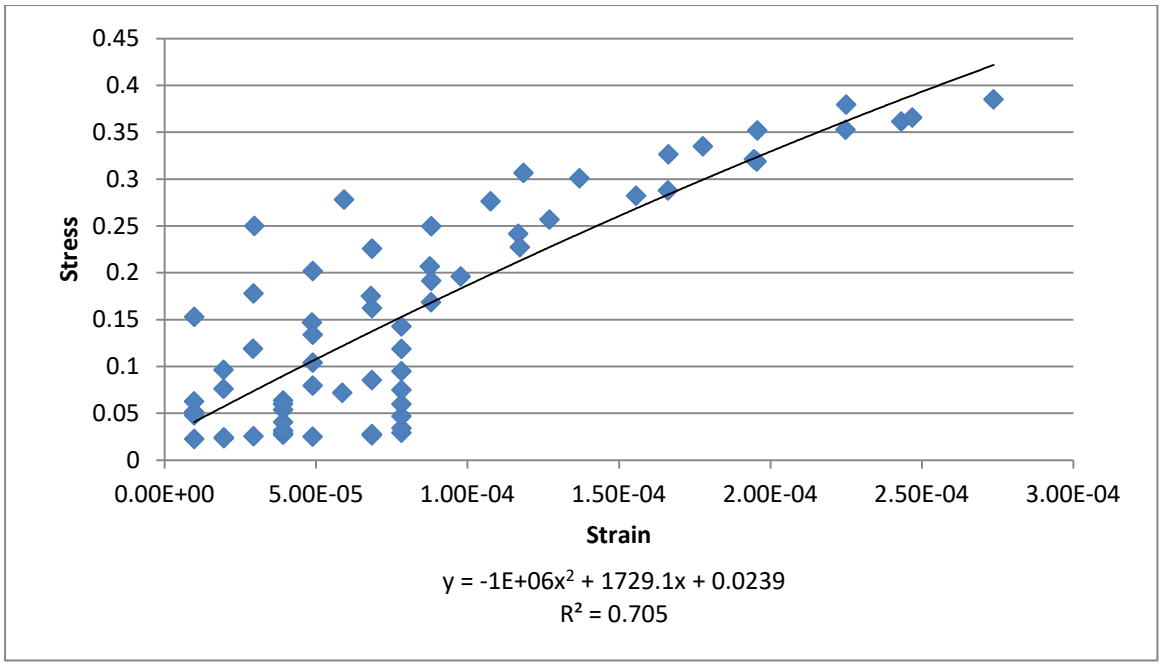

Figure 5.17. Analysed dataset for SMA NMA 19 mm with PMB40

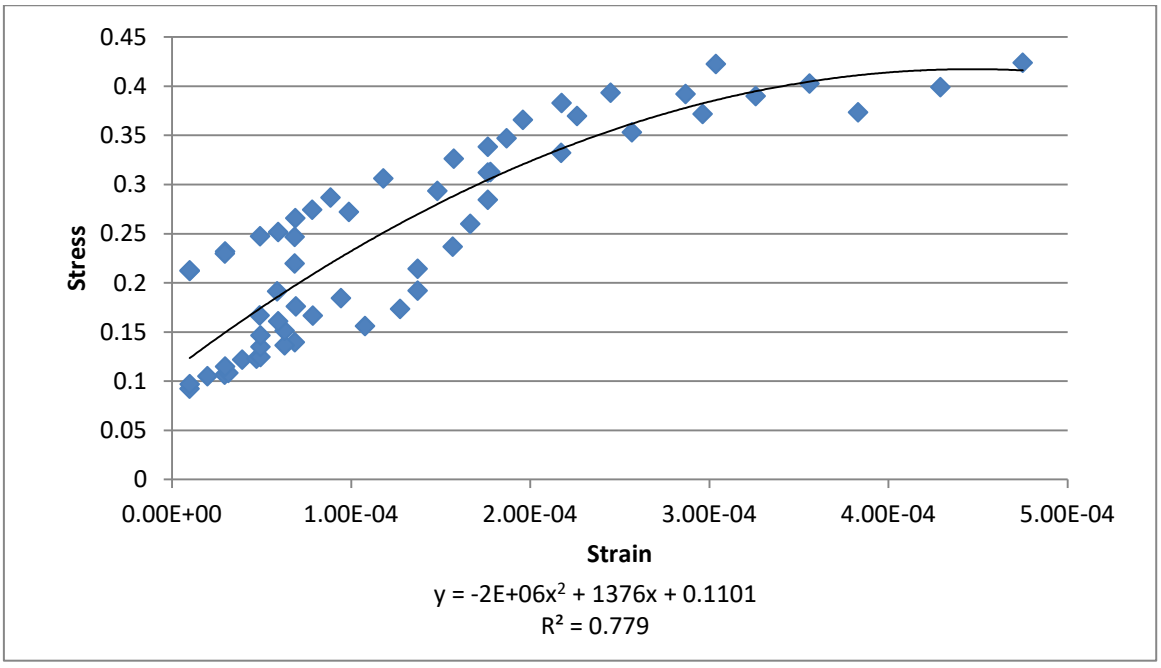

Figure 5.18. Analysed dataset for SMA NMA 19 mm with CRMB60

5.2 Validation of the results

The first derivative of the stress-strain equation will give the slope which is equal to the modulus value. The below table indicates the modulus value obtained from the first derivative of stress-strain equation and coeffient of determination(R^2). The ITS value and modulus value obtained from UTM laboratory testing is also shown in the table for all 18 mix types. It has been seen that both the modulus values,i.e., modulus value obtained from the ITS test data and resilient modulus value from the laboratory testing were nearly same.

Table 5.1. Validation Table

Mix Grade	Binder Typer	ITS Value(MPa)	Stress-Strain equation	R^2	Modulus (ITS data)	Mr (lab test)
BC1	VG40	0.757	$y = -1E+06x^2+1014.6x+0.0352$	0.869	1015	1580
	PMB40	1.335	$y = -2E+06x^2+1798.3x+0.0878$	0.831	1798	1613
	CRMB60	0.803	$y = -788424x^2+1013.2x+0.0678$	0.876	1013	1312
BC2	VG40	0.740	$y = -2E+06x^2+1418.4x+0.0644$	0.835	1418	1684
	PMB40	0.943	$y = -2E+06x^2+1516.8+0.2006$	0.940	1517	1736
	CRMB60	0.870	$y = -913642x^2+913.45x+0.1585$	0.624	913	1294
DBM1	VG40	0.648	$y = -2E+06x^2+1194.6x+0.1232$	0.836	1195	1670
	PMB40	0.815	$y = -2E+06x^2+1569.1x+0.157$	0.866	1569	1675
	CRMB60	0.695	$y = -788738x^2+802.23x+0.0508$	0.906	802	1341
DBM2	VG40	0.791	$y = -2E+06x^2+1316.5.6x+0.0203$	0.774	1317	1677
	PMB40	1.061	$y = -2E+06x^2+1951.7+0.0347$	0.731	1952	2199
	CRMB60	0.934	$y = -2E+06x^2+1317.5x-0.0134$	0.830	1318	1593
SMA NMA 13 mm	VG40	0.681	$y = -3E+06x^2+1648.8.5x+0.0193$	0.625	1649	1657
	PMB40	0.837	$y = -3E+06x^2+1911.1x+0.1471$	0.722	1911	2054
	CRMB60	0.695	$y = -2E+06x^2+1638.9x+0.0248$	0.874	1639	1513
SMA NMA 19 mm	VG40	0.581	$y = -1E+06x^2+1009.6x-0.0231$	0.949	1010	1421
	PMB40	0.749	$y = -1E+06x^2+1729.1x+0.0239$	0.705	1729	1612
	CRMB60	0.650	$y = -2E+06x^2+1376x+0.1101$	0.779	1376	1252

CHAPTER 6 : CONCLUSIONS

6.1 General

The modulus of resilience value is a prominent variable, used to demonstrate the material behaviour and to antedate the pavement performance in the design of pavement. There are many empirical approaches developed by the researcher to determine the resilient modulus based upon the factor which is affecting it mostly or the parameters which is showing good dependency. Eventhough, there is a machine named as universal testing machine (UTM), which is available to direct the resilient modulus value to simulates the field conditions, but due to its limitation of having high - price, this equipment is availabe in few institute across the world. Also, testing in this equipment is time taking task as well as required a trained personel to execute the test due to sensitivity of various components like linear variable differential transducers (LVDTs), actuator, etc. Hence, it is necessary to develop an alternative approach to closely reach towards the resilient modulus value which replicate the same actual field condition for the accurate design of the pavement. In the present study of research, an attempt has been made by implementation of indirect tensile strength test data analysis to arrive at the modulus value which is nearly same as the resilient modulus value obtained from the laboratory testing. The load and lateral deformation values are used to convert into the ITS strength and lateral strain value. Only 10% of the yield strain data are utilized corresponding to the maximum load at the time of failure and the graph needs to be plotted for this 10% data between the stress and strain. A second degree polynonial equation for stress-strain curve has been finds out to be fit with the satisfactory coeffiecient of determination value (R^2). The first derivative of the stress-strain equation will corresponds to the modulus value and holds validation analogously with the obtained resilient modulus value from the laboratoty testing.

Various conclusions that were drawn from this research study are mentioned below,

i. Resilient modulus value determined from the laboratory testing is nearly same as the modulus value obtained from the first derivative of stress-strain equation of 10% analysed ITS data.
ii. The mixes prepared with PMB40 binder is showing higher value of modulus of resilience (Mr) followed by that the VG40 and then CRMB60 binder. But in case of

indirect tensile strength (ITS) value, the mixes casted with PMB40 binder indicates the higher magnitude followed by that the CRMB60 and then VG40 binder.

iii. Mr value will lead to get increase with rise in magnitude of ITS in case of mixes prepared with VG40 and PMB40 binder. But in case of mixes prepared with CRMB60 binder, Mr value will get decreased even if there is increase in the ITS value.

CHAPTER 7 : REFERENCES

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