CHAPTER 6

CHARACTERIZATION OF ASPHALT MIXTURES

6.1 Preface

Ensuring satisfactory performance of asphalt binders has a direct relation to the performance at the mix level. In general, asphalt mixture is a heterogeneous composite material; its mechanical performance (such as rutting, fatigue, and moisture) is expected to change drastically with the variation in mix attributes (asphalt binder and aggregate source). On other hand, production temperature is an essential factor influencing the performance of asphalt mixtures. It is expected that the asphalt mixtures produced at lower mixing and compaction temperature, such as WMA mixtures, may have an increased tendency towards rutting, fatigue, and moisture failure. Therefore, similar to the characterization of WMA binders (as detailed in Chapter 5), the evaluation of WMA mixtures is indeed essential to facilitate the application of WMA technology for pavement construction.

WMA mixtures must be evaluated and screened based on different failure aspects such as rutting, fatigue, and moisture failure [13]. As different WMA technologies have been chosen in the present study, it was observed that the extent of reduction in production temperature is a function of aggregate type, base asphalt binder, and WMA additive (Chapter 4). However, the mechanical performance of WMA considering the effect of base binder (used to prepare WMA) and aggregate type has not been explored to a satisfactory extent. Therefore, there is a need to study the effect of lower production temperatures on the performance of different WMA and compare its results with the response of conventional HMA. This will help in selecting appropriate WMA technology for the construction of asphalt pavements. Also, researchers [536,558,569] attempt to correlate the response of asphalt binder and mixtures to understand the role of asphalt binders in predicting the behavior of asphalt mixtures. However, very limited studies [254,277,570] have considered the effect of WMA technologies on the correlation between asphalt binder and mixtures. In addition, establishing the limiting/threshold values for the acceptance/rejection of asphalt binders and mixtures depending on their performance parameters is yet to be explored. This forms the motivation for this work.

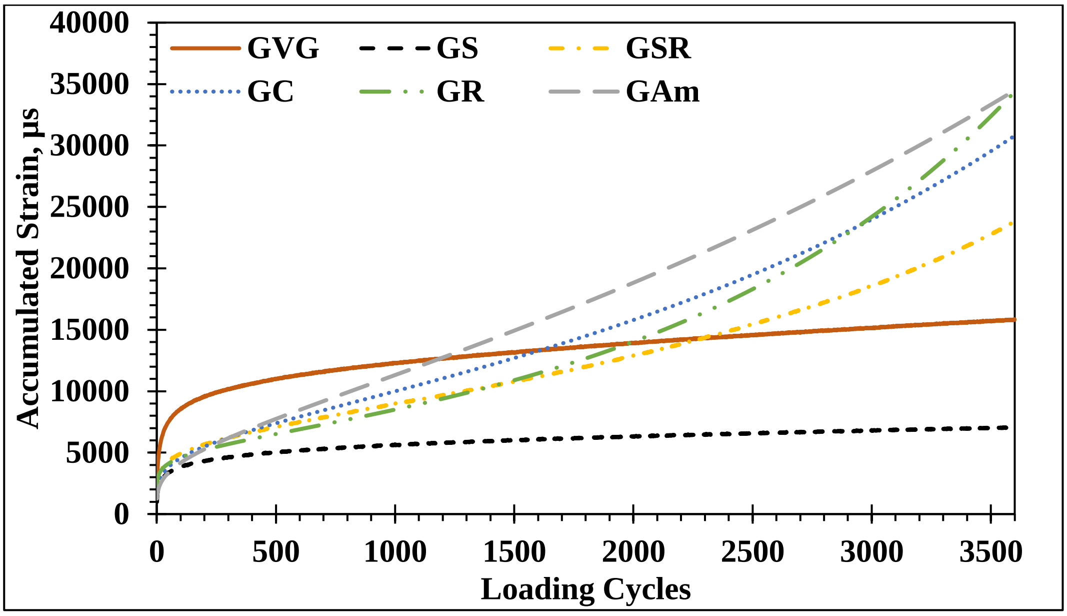
This chapter is an extension of Chapter 5, wherein the performance of WMA binders was evaluated and compared with the corresponding values of base asphalt binders. The present chapter assesses the effect of WMA technologies on the performance of asphalt mixtures. Different tests, such as the cyclic compression test (CCT) was conducted at 60°C to determine the rutting performance, while the Indirect tensile cracking test (Ideal CT) was performed at 20°C to evaluate the fatigue behavior. The moisture damage in asphalt mixture was evaluated based on three different test approaches. The boiling water test (BWT) was performed on a loose asphalt mixture, whereas Tensile strength ratio (TSR) and retained Marshall stability (RMS) were considered as parameters for evaluating the moisture damage in a compacted specimen. These approaches were chosen based on their popularity in the past literatures [169,571,572]. Further, correlations between the attributes of asphalt binders and mixtures have been established, and the limiting values of different performance predictors were proposed based on these correlations. Overall, this chapter covers the performance of WMA mixtures based on different aspects, incorporating two different base asphalt binders (VG30 and PMB40), two aggregate sources (granite and dolomite), and five different

WMA additives at their optimum dosages (Sasobit, Sasobit Redux, Cecabase, Rediset, and Aspha-Min).

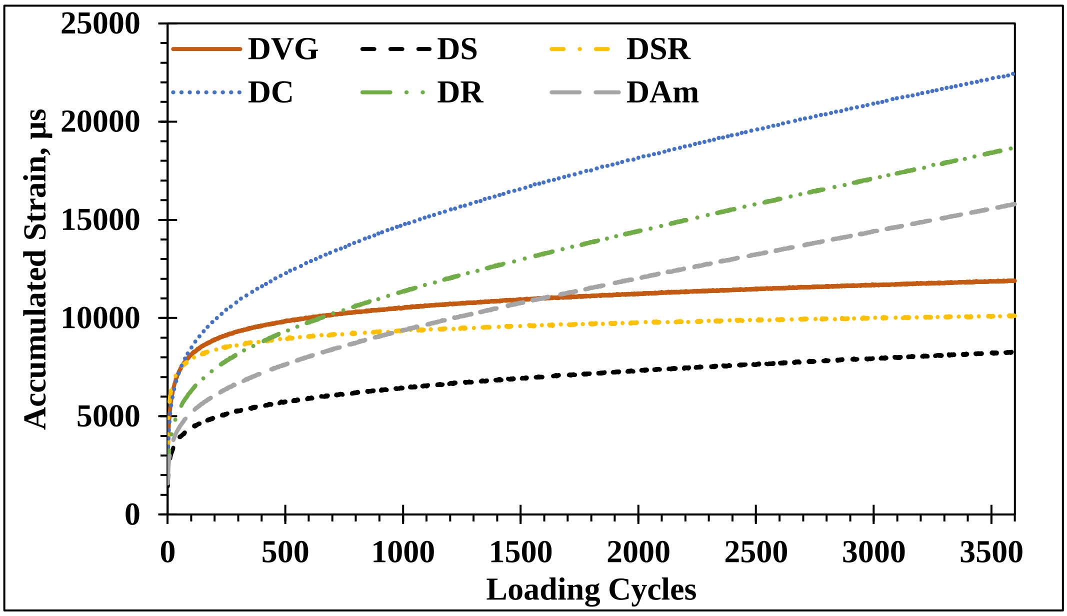
6.2 Rutting Performance

Asphalt pavements are generally subjected to heavy traffic loading and environmental conditions, leading to longitudinal depressions under the wheel path. These depressions are typically defined as rutting or permanent deformation [573]. It is a time-dependent strain accumulation phenomenon, occurring primarily at high temperatures (>40°C). Most researchers [280,535,574–577] have used static creep, dynamic creep or cyclic compression test (CCT), wheel tracking test, and triaxial repeated load test to evaluate rutting resistance. Among the available tests, CCT is considered one of the most practical tests that simulate the concept of axial compression on asphalt pavements [406,576,578]. This test was developed in the mid-1970s by Monismith et al. [579], and it was estimated that CCT could be appropriate to investigate and compare the rutting behavior of unmodified as well as modified asphalt mixtures. The following section discussed the effect of WMA on the rutting behavior of asphalt mixtures determined by conducting CCT at 60°C. The working procedure, sample preparation, evaluated rutting parameters, and other test conditions were demonstrated in Chapter 3.

Figure 6.1 presents the variation of accumulated strain with the loading cycles for different asphalt mixtures (HMA and WMA). The variation in the rutting behavior of WMA mixtures is a function of the ageing of asphalt binder/production temperature of asphalt mixtures (which inherently depends on the aggregate source, base asphalt binder, WMA technology, and the interaction between them). It was found that all the asphalt mixtures (HMA and WMA) sustained 3600 loading cycles, irrespective of the aggregate source and base asphalt binder. It is well known that the lower production temperature of WMA leads to less oxidation of asphalt binders and thus exhibits lower rutting resistance [242,280]. However, few WMA mixtures performed relatively better than their respective HMA mixtures, as indicated by lower strain response. Among different WMA, the addition of Sasobit in any combination of asphalt binder and aggregate source offered less accumulated strain values. This behavior is attributed to the crystallization of wax particles at the test temperature (60°C), which is lower than the melting point of Sasobit [246,268,580]. The crystallization effect increased the stiffness and thereby led to high rutting resistance. It can be perceived that the influence of Sasobit is more pronounced in VG30 than PMB40. Similar observations were found in a previous study [356]. The addition of chemical and foaming agents in VG30 showed higher strain values throughout the loading cycles. However, their behavior was not significant in the case of PMB40 mixtures; instead, no definite trend was observed. Some of the WMA mixtures (such as GSR, GC, GR, GAm, DAm, DR, and DC) had strains lower than HMA mixtures, but below a certain number of loading cycles (Figure 6.1); nevertheless, the rate of strain accumulation increased rapidly after a further increase in loading cycles. For instance, using Aspha-Min with VG30 and dolomite indicated better rutting resistance (accumulated strain is 9375 µs) at 1000 loading cycles compared to the reference HMA mixture (accumulated strain is 10527 µs), whereas it showed relatively poor performance than HMA at 3600 loading cycles. These inconsistent trends/observations were also noted for other WMA mixtures considered in this study. Overall, the strain curve results were inconclusive in revealing the effect of aggregate source, base asphalt binder, and WMA technology owing to the large variations in the strain response throughout the loading cycles.


GVG
GS
GSR
GC
GR
GAm
Accumulated Strain, μs
Loading Cycles
40000
35000
30000
25000
20000
15000
10000
5000
0
0
500
1000
1500
2000
2500
3000
3500

(a)


DVG
DS
DSR
DC
DR
DAm
Accumulated Strain, μs
Loading Cycles
25000
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3000
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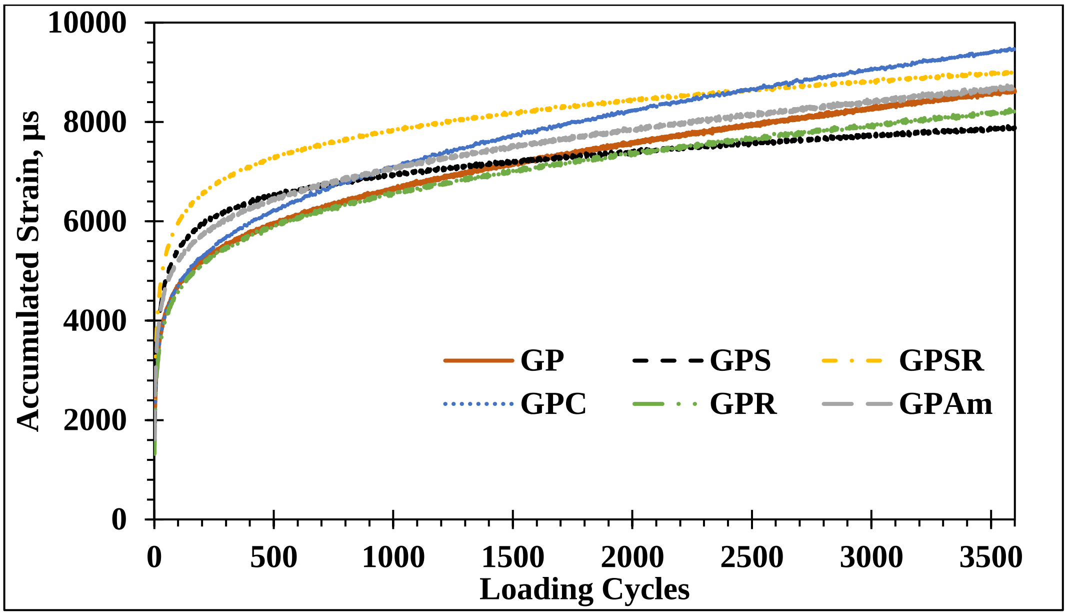
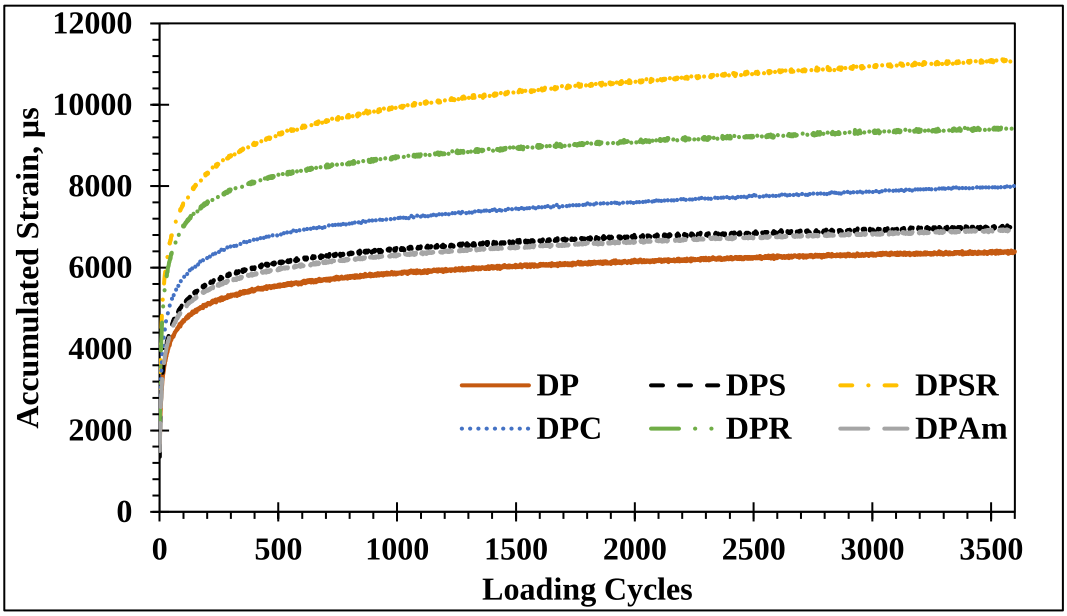
(b)



(c)

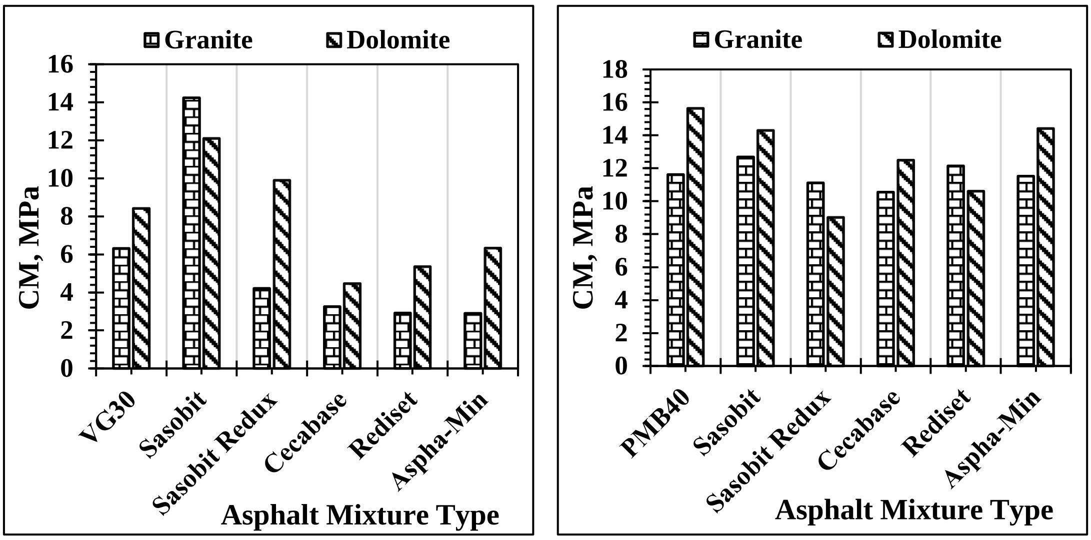


(d)

Figure 6.1. Strain response with change in loading cycles for different groups of asphalt mixtures (a) GVG, (b) DVG, (c) GP, and (d) DP

EN12697-25 [506] recommended another parameter, typically known as creep modulus (CM), to compare the rutting behavior of asphalt mixtures considering different aggregate sources and base asphalt binders. This parameter considers the effect of stress (along with the accumulated strain) and is defined as the ratio of stress to the accumulated strain at the end of loading cycle. A lower CM indicates high permanent deformation and vice-versa.

Figure 6.2 (a-b) shows the CM value corresponding to different asphalt mixtures. As can be seen, the value of CM clearly differentiates the effect of base asphalt binder and aggregate source on the rutting behavior of WMA mixtures. On an average, the asphalt mixtures prepared with PMB40 as base asphalt binder or dolomite as aggregate source led to comparatively better rutting resistance than VG30 or granite, respectively. For example, the addition of Rediset with VG30 and granite (GR) resulted in slightly lower CM values than conventional HMA, whereas in the case of PMB40 (i.e., GPR), it showed a relatively higher value of CM, indicating better resistance against rutting failure. It has to be noted that the effectiveness of WMA additives primarily depends on the interaction between different combinations of base asphalt binder, aggregate source, and WMA technology [181,288,581]. Interestingly, the rutting behavior of Sasobit with VG30 and granite aggregates was found to be promising, while with PMB40, it showed better compatibility (higher CM) with dolomite aggregates. A similar observation was noted in the case of Sasobit Redux and Rediset-based WMA mixtures. Based on the CM value, it can be stated that organic-based WMA additives performed better against rutting failure, followed by foaming and chemical agents. These observations were independent of aggregate source and base asphalt binder.



(a) (b)

Figure 6.2. Creep modulus for different WMA mixtures (a) with VG30 and (b) with PMB40

6.3 Fatigue Performance

As already stated in the preceding chapters, fatigue cracking is one of the major distresses at intermediate temperatures due to the repetition of traffic load. Since 1960, the concept of fracture mechanics has been extensively used to assess the fracture/fatigue properties of asphalt mixtures. Use of fracture mechanics and the interpretation of their testing parameters have been widely discussed in past studies [534,582,583]. Based on this phenomenon, the fatigue performance of asphalt mixtures is often evaluated in terms of fracture energy (FE), defined as the energy required to initiate or propagate the crack on a sample of asphalt mixture under a given loading condition. Typically, FE is the ratio of area under the load-deformation curve divided by the width and thickness of the cylindrical compacted specimen. Any asphalt mixture with higher FE has a better resistance against fatigue cracking [584,585].

Several test methods, such as Semi-circular bending (SCB), Disc shape tension test (DCT), Indirect tensile test (IDT), Single edge-notched beam (SENB), and Double edge-notched tension test (DENT), have been developed to determine the fatigue resistance in terms of FE [585–589]. Although FE can be determined using any of these test methods, the magnitude of FE calculated from each test may differ. This is attributed to the difference in loading condition, stress/strain value, shape of the sample, and the working principle of an individual test. Few studies, including Radeef et al. [590], Zhou et al. [587,591], and Newcomb et al. [592], presented a critical review of popular methods that can be used in this direction. Though the above-mentioned tests deliver a reliable process for determining fatigue resistance, with excellent field performance, the complexity/difficulty in sample preparation and sophisticated instrumentation lead to their poor candidature for fatigue characterization. To overcome these drawbacks, Zhou et al. [587] introduced a simple cracking test, i.e., Indirect tensile cracking test (Ideal CT), which allows the use of a cylindrical specimen without cutting or notching (as required in SENB, DENT, or SCB). The test can be performed with simple instrumentation and high repeatability [587,591]. The loading in the Ideal CT test is continued even after the achievement of peak load to fully capture the post-peak behavior of the asphalt mixture. Buttlar et al. [593] reported an appreciable correlation between results obtained from Ideal CT and in-situ fatigue cracking. Considering these advantages, an Ideal CT test was carried out in the present research work to adjudge the fatigue behavior of HMA and WMA mixtures. The details of sample preparation, test temperature, and working procedure of this test can be found in Chapter 3.

Various studies conducted Ideal CT test at different temperatures to determine the resistance against fatigue cracking in terms of FE. While analyzing the fatigue performance, researchers [594,595] reported the inadequacy of the FE parameter to differentiate two asphalt mixtures with distinctly diverse behavior (depending on the material and their intrinsic characteristics). For example, as shown in Figure 6.3, the FE of two different samples were similar, but there is a considerable difference in their peak load and post-peak behavior. This resulted in similar fatigue resistance for both the asphalt mixtures (P and Q); however, the actual behavior of these asphalt mixtures may differ. It indicates the inadequacy/inaptness of FE parameter for distinguishing the asphalt mixtures with high peak load and brittle post-peak characteristics (as displayed by P), and asphalt mixtures having lower peak load and flexible/ductile post-peak behavior (as shown by Q). Similar discrepancies were reported by the previous literatures [594,596]. Thus, a new parameter would be needed to assess fatigue performance more generically and logically.

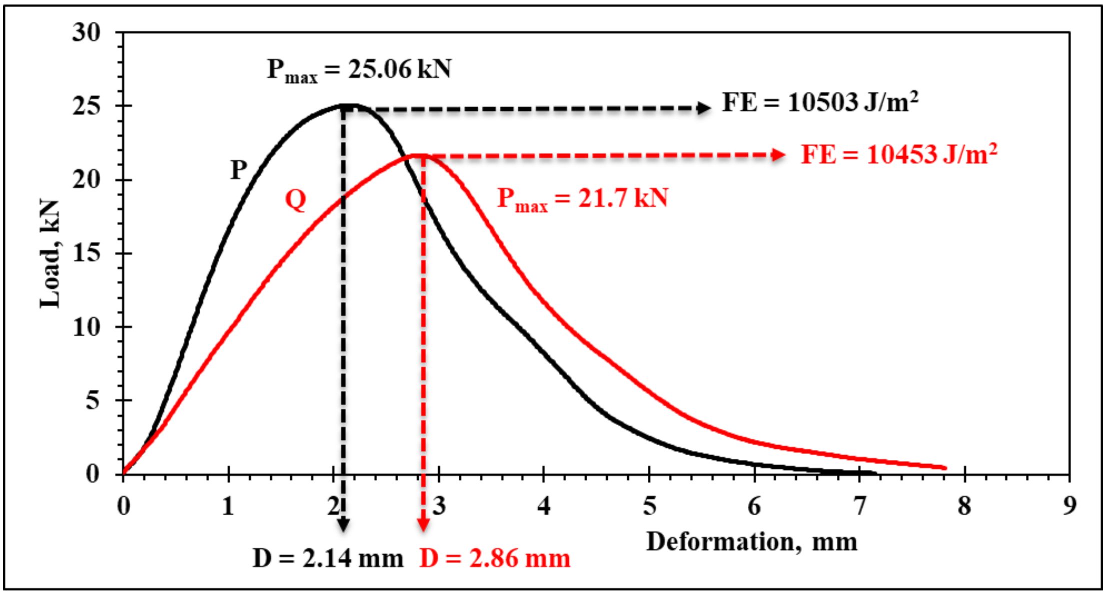


Figure 6.3. Representative example showing the inadequacy of FE parameter

Few studies [595,597,598] recommended the normalization of the FE parameter to better differentiate the asphalt mixtures with similar FE. The normalization can be done using peak load and the parameter that defines the shape of the post-peak portion (i.e., slope). In this regard, Ashish et al. [599] stated that post-peak characteristics are essential to capture the overall fracture properties, but are often ignored in FE analysis. Although the normalization using the slope value offers more information on the brittle and ductile nature of asphalt mixtures, its use has been opposed due to the higher variability in results [595]. On the contrary, another characteristic parameter, i.e., peak load, imparts relatively lower variability and thus, is considered for the analysis in the present study. A new fatigue parameter, defined as Fatigue Index (FI), was introduced to estimate the fatigue performance of asphalt mixtures. FI identifies the response of an asphalt mix based on the fracture energy and the mechanical strength (peak load). Mathematically, FI is written as:

$$FI = \frac{FE}{P_{Max}} \tag{6.1}$$

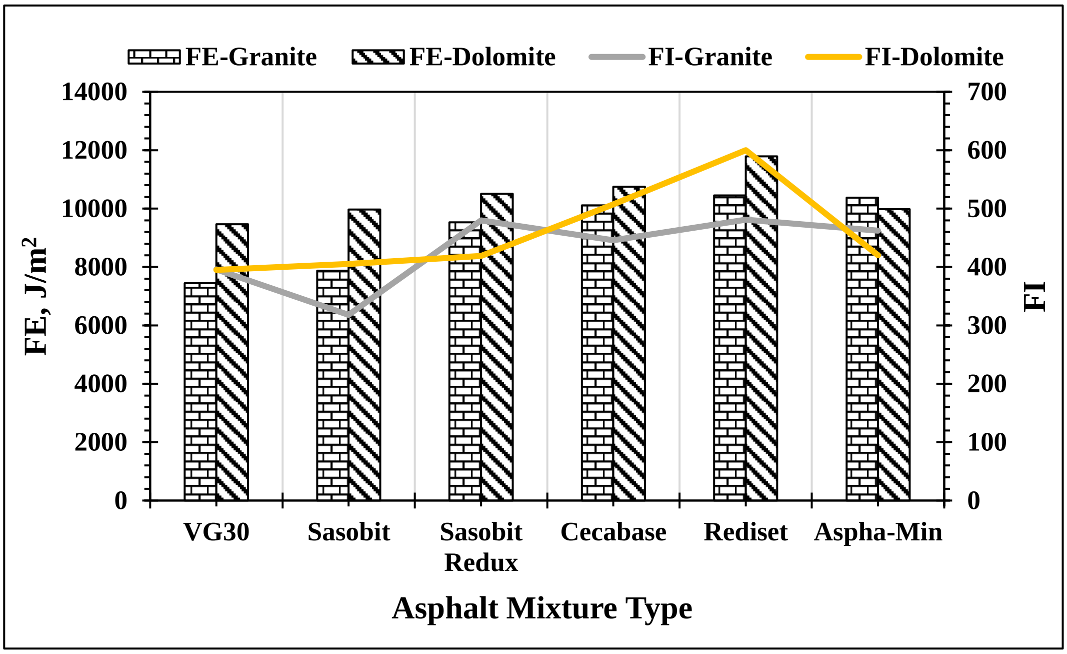
As per equation 6.1, the asphalt mixture with high P_{Max} and low FE is expected to undergo a brittle failure (as shown by P in Figure 6.3), indicating steep post-peak characteristics compared to a ductile asphalt mixture (Q in Figure 6.3). As FI considers the failure pattern (brittle/ductile), it is logical to use FI instead of FE to determine the fatigue resistance in a more conceptual way. A higher value of FI would indicate higher resistance to fatigue cracking and vice-versa. FI was introduced with the scope of discriminating the asphalt mixtures with the same FE but different peak load values. However, the inferences of both FE and FI are presented in this study to assess the effect of different test parameters on the fatigue characteristics of asphalt mixtures. The correlations between these two parameters were also determined to understand their dependency on each other.

6.3.1 Discussion on FE and FI of WMA mixtures

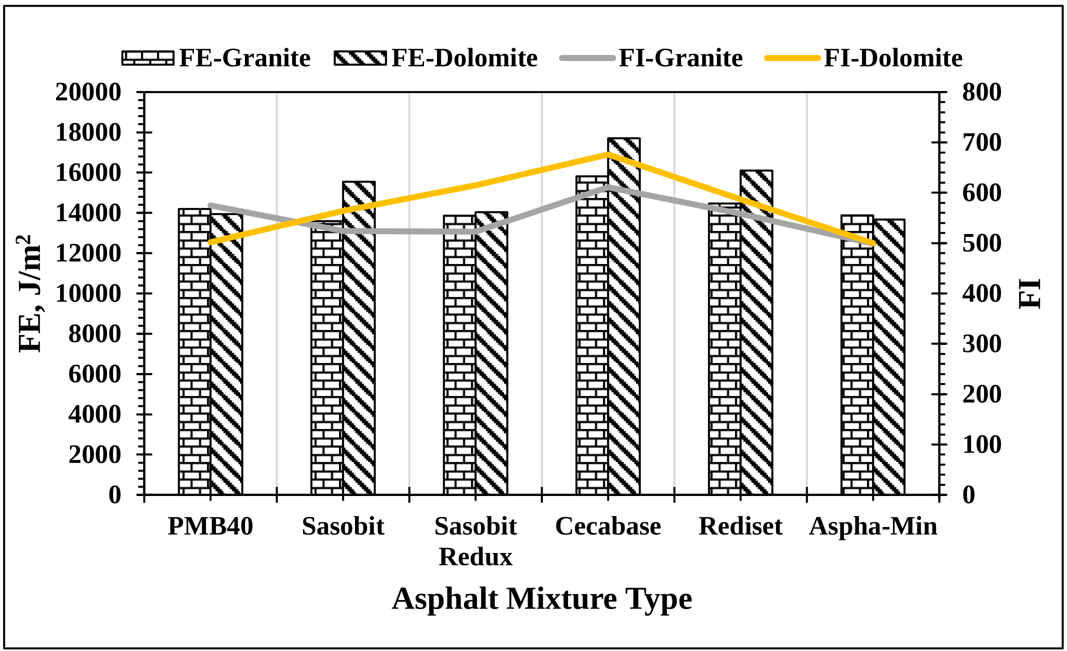
Figures 6.4a and Figure 6.4b show the variation in FE and FI for different asphalt mixtures (HMA and WMA) prepared with VG30 and PMB40 as base asphalt binders, respectively. The effect of WMA was found to be more dominant in the case of VG30 as compared to PMB40. This is attributed to the predominance of polymer networks in the case of PMB40 (as demonstrated using SEM in Chapter 3). This behavior was found to be consistent for both FE and FI parameters. In addition, the dominating nature of the polymeric network was identified in organic and foaming-based WMA mixtures, as indicated by the lower percent change in Table 6.1. A comparison of selected WMA technologies appreciated the application of chemical-based WMA agents due to their superior fatigue performance. This may be attributed to the emulsification of base asphalt binder with the addition of chemical agents [168,281,600]. Irrespective of fatigue parameters, the combination such as DPC, GPC, DR, and GR showed improved performance among their respective group (which is categorized based on the aggregate and base asphalt binder, i.e., DP, GP, DVG, and GVG, respectively). On an average, compared to granite, the asphalt mixture comprising dolomite aggregates exhibited higher improvement in FE and FI, resulting in higher resistance to fatigue cracking. This is attributed to the strong interfacial bond between the dolomite aggregate and WMA binders, which does not break under the action of repetitive load [601–603]. Thus, it can be stated that aggregate source, base asphalt binder, and WMA technology significantly influence fatigue performance.

As can be seen in Table 6.1, each combination of asphalt mixture reflects variation in fatigue resistance in comparison to their respective HMA mixture. The addition of Sasobit Redux in VG30 improved the fatigue resistance by 11% (in terms of FE) and

6% (in terms of FI) when the aggregate source was dolomite (DSR), whereas, in the case of granite aggregate (GSR), fatigue performance was increased by 28% (in terms of FE) and 21% (in terms of FI) compared to their respective HMA mixtures. Considering constant aggregate source, the use of a foaming agent, i.e., Am, in combination with VG30 and granite improved the FE and FI by around 39% and 17%, respectively, whereas changing the base asphalt binder to PMB40 led to a lower impact of Am, as shown by a negative percent change (Table 6.1). Similar observations were made for other combinations of asphalt mixtures, hence not detailed here for brevity. However, it was identified that along with the above-mentioned variables (aggregate source, base asphalt binder, and WMA technology), fatigue performance inevitably varied with the change in fatigue parameters (FE and FI).



(a)



(b)

Figure 6.4. Variation in fatigue performance with the inclusion of WMA additives in different asphalt mixtures (a) with VG30 and (b) with PMB40

Table 6.1. Percent change in FE and FI of WMA mixtures with respect to their respective HMA mixtures

WMA Additive	Asphalt Mixture Group							
	GVG		DVG		GP		DP	
	FE, %	FI, %	FE, %	FI, %	FE, %	FI, %	FE, %	FI, %
Sasobit	5.80	-19.70	5.37	2.53	-4.24	-8.87	11.50	12.35
Sasobit Redux	28.04	21.21	11.04	6.08	-2.40	-9.04	0.70	22.51
Cecabase	35.77	12.63	13.68	28.35	11.40	6.26	27.04	34.66
Rediset	40.37	21.46	24.65	51.90	1.83	-2.96	15.47	16.93
Aspha-Min	39.25	16.67	5.48	6.33	-2.27	-13.22	-1.93	-0.40

6.3.2 Discrepancies between FE and FI

Although FE analysis indicated comparable fatigue performance for GS (Sasobit with granite and VG30) as conventional HMA, the same asphalt mixture had poor fatigue resistance based on the FI parameter. This discrepancy may arise due to the ignorance of post-peak behavior in FE analysis (i.e., brittle or flexible characteristics). One such example is shown in Figure 6.5, where GSR displayed flexible nature with a gradual decrease in post-peak slope, indicating higher load-bearing capacity than DSR, which showed brittle characteristics (steep post-peak slope). However, FE parameter indicated higher fatigue resistance for DSR combination even though it yields brittle failure (Figure 6.5). On the other hand, FI parameter successfully differentiates both the asphalt mixtures and displayed lower fatigue performance of DSR than GSR combination. A similar type of discrepancy was observed while analyzing other tested asphalt mixtures.

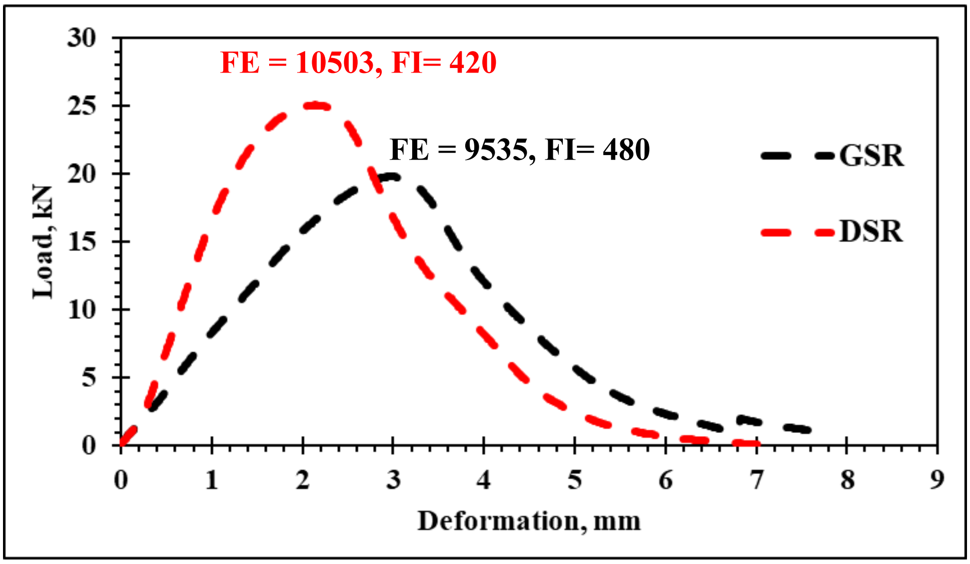


Figure 6.5. Comparison between FE and FI

Figure 6.6 presents the correlation between both the fatigue parameters. An increasing trend was observed between the two parameters, indicating a strong correlation with an R^2 of 0.78. Despite the appreciable correlation and similar trend, the effect of brittleness and the flexible nature of any infrastructure cannot be overlooked. Thus, the present study recommends the use of FI rather than FE parameter for better estimation of the fatigue resistance of asphalt mixtures.

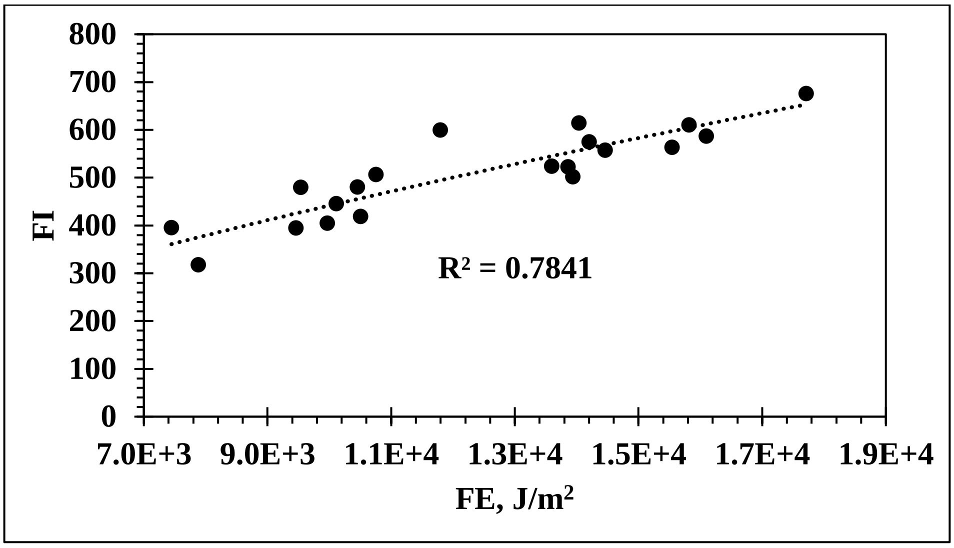


Figure 6.6. Correlation between fatigue parameters

6.4 Moisture Resistance

Moisture damage is one of the premature and complex forms of failure common in asphalt pavement [604]. In the literature various test methods are available for assessing the resistance of the mixture against moisture damage [324,326,327]. Mehrara and Khodaii [605] and Kakar et al. [606] have provided a comprehensive review about these test methods. In general, the laboratory evaluation of moisture damage can be grouped into two test domains: (a) test on loose asphalt mixture or mixture component and (b) tests on compacted asphalt mixture. Test methods considered in this study were chosen considering their popularity and ease of testing under each category of testing. For loose mixture, boiling water test (BWT) and pneumatic adhesion test (PAT) (discussed in Chapter 5) were chosen. Tensile strength ratio (TSR) and retained Marshall stability (RMS) were considered as parameters for evaluating the moisture damage in compacted asphalt mixture. Table 6.2 highlights the details of the test methods used in this study to evaluate moisture susceptibility of asphalt mixtures.

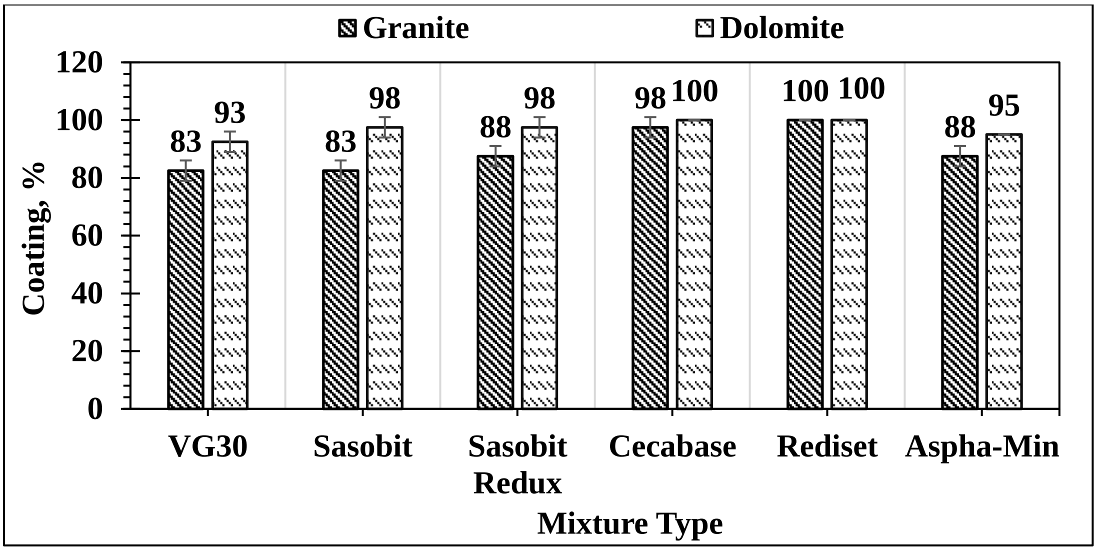
Table 6.2. Overview of the popular test methods used for assessing the moisture susceptibility

Test	Specification	Description	Moisture damage indicator (specified limit)
PAT*	AASHTO T361 [331]	Evaluating the bond strength (BS) between the asphalt binder and mineral aggregates under dry and wet condition.	Bond strength ratio (BSR) $= \frac{BS_{Wet}}{BS_{Dry}}$ (No specified limit)
BWT	ASTM D3625 [509]	Subjecting the loose asphalt mixture to boil under water at a specified temperature for 10 minutes.	Visualization of percentage coating or stripping over the aggregates (% coating ≥85%) [510,607]
Modified Lottman	AASHTO T283 [344]	Measuring the Indirect tensile strength (ITS) of compacted asphalt mixtures in unconditioned and moisture conditioned state.	Tensile strength ratio (TSR) $= \frac{ITS_{Conditioned}}{ITS_{Unconditioned}}$ (TSR ≥80%)
Retained Marshall stability	ASTM D6927 [511]	Determining the Marshall stability (MS) of asphalt mixtures before and after moisture conditioning.	Retained Marshall stability (RMS) $= \frac{MS_{Conditioned}}{MS_{Unconditioned}}$ (RMS ≥80%)

**Discussion on PAT has been detailed in Chapter 5*

6.4.1 Discussion on Boiling Water Test

Figure 6.7 presents the results of percentage coating obtained for different samples. Irrespective of the base binder and type of aggregates, WMA binders showed better performance than the base asphalt binders. The qualitative inferences from the analysis of BWT are in agreement with the PAT results (as shown in Chapter 5), where chemical WMA agents outperformed other additives. Though the effect of base binder was not found to be significant, dolomite aggregates showed higher resistance to moisture damage in comparison to granite. The disagreement in the ranking of the binders between BWT and PAT is attributed to the differences in simulation procedure of moisture ingress. Moreover, the subjective assessment of percentage coating in BWT may lead to variation in the results.



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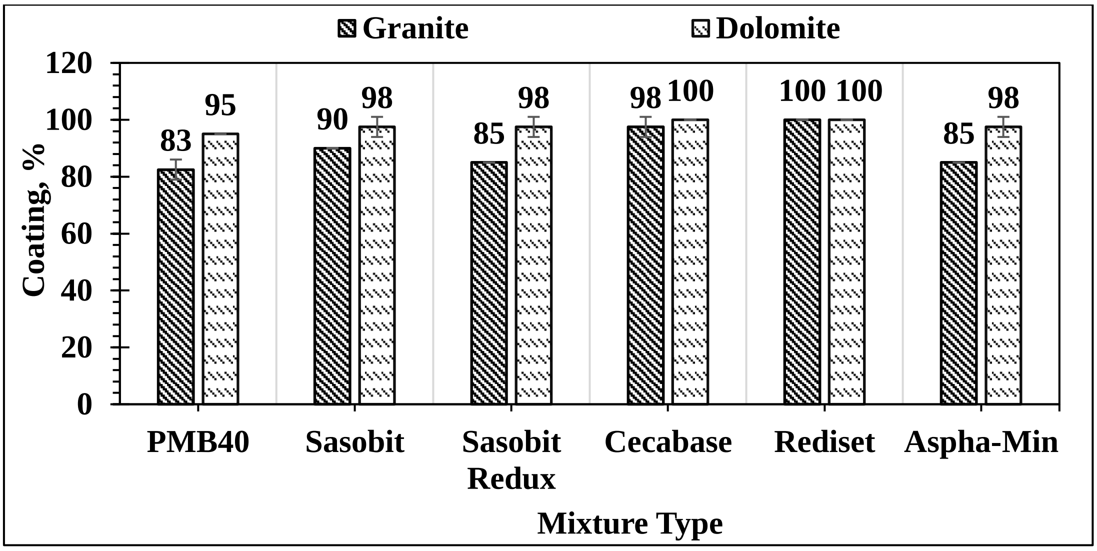
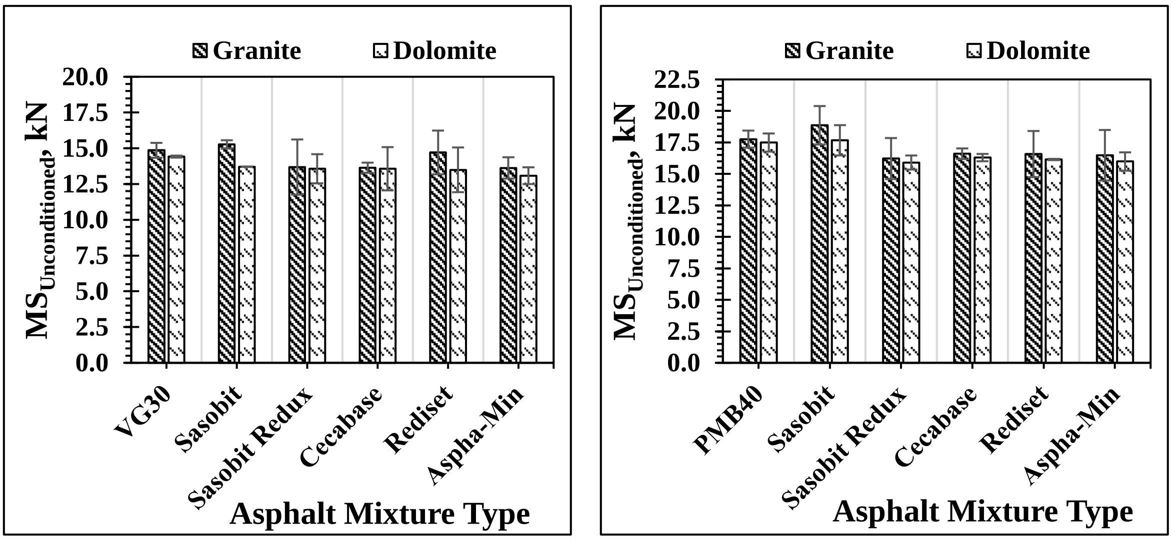


(b)

Figure 6.7. Variation in coating over the aggregates with the addition of WMA additives in different base asphalt binders (a) VG30 and (b) PMB40

6.4.2 Discussion on Retained Marshall Stability

The unconditioned and conditioned Marhsall stability along with the RMS values are presented in Figure 6.8. It should be noted that unlike PAT and BWT, RMS is obtained by testing compacted mixtures. Hence the results are expected to be dependent on other parameters such as air-voids and aggregate gradation in the mixture. Owing to the higher stiffness, unconditioned and conditioned Marshall stability of PMB samples were higher than VG 30 samples. Except Sasobit in granite, Marshall stability of WMA samples were under ±10% of the value of controlled mixtures. Thus, no strong inference could be made related to interaction between aggregate and binder using this method. RMS values which quantify the moisture damage in asphalt mixture indicated that all the mixtures provided sufficient resistance against moisture damage, with value > 90%. Cecabase with PMB40 and dolomite showed the highest RMS (99.11%), while Sasobit with granite and VG30 displayed the lowest value (92.55%). The relative values between different samples were found to be in close proximity with no significant difference.



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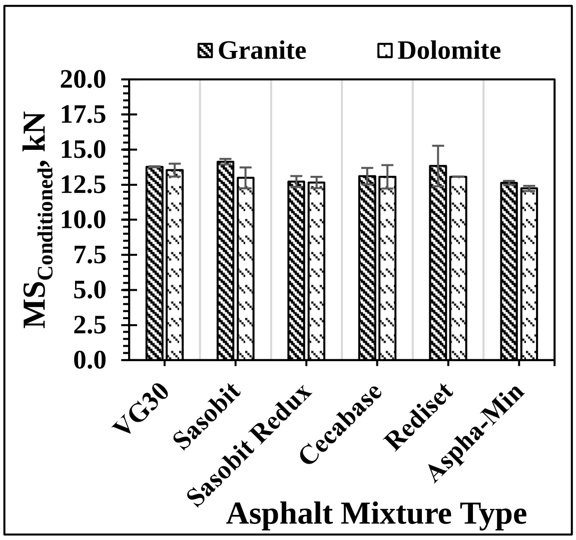

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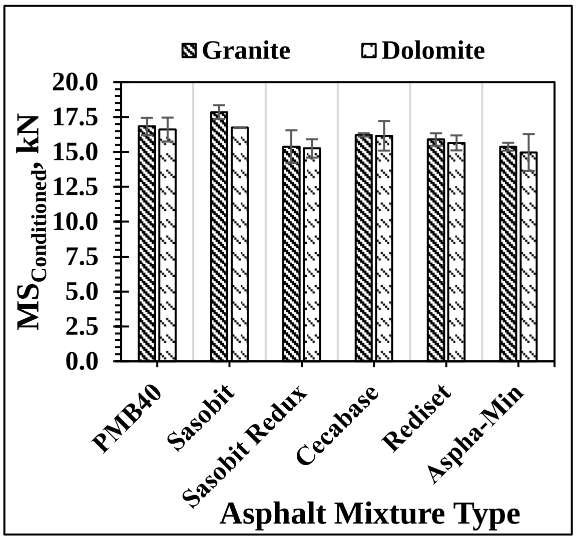

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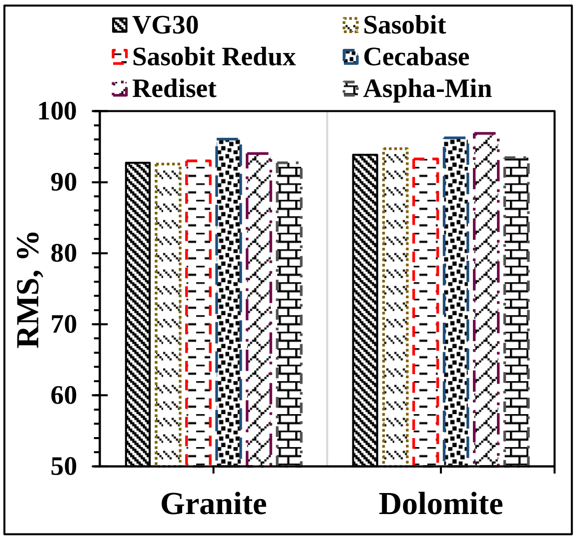

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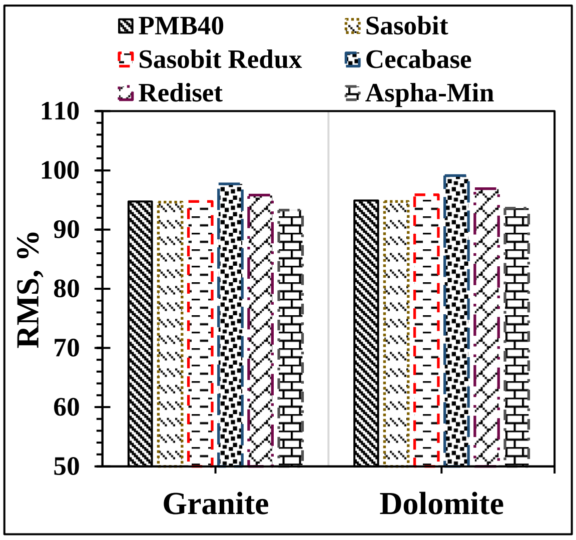


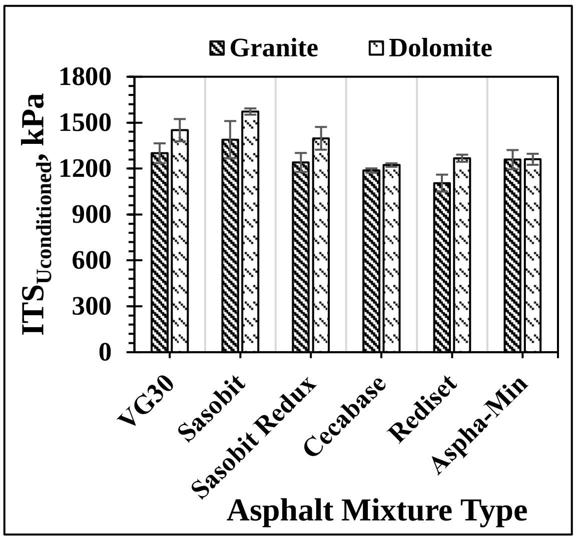
(f)

Figure 6.8. Variation in MS for WMA mixtures prepared using different base asphalt binders (a) VG30 in unconditioned state (b) PMB40 in unconditioned state, (c) VG30 in conditioned state, (d) PMB40 in conditioned state, (e) RMS when VG30 is base asphalt binder, and (f) RMS when PMB40 is base asphalt binder

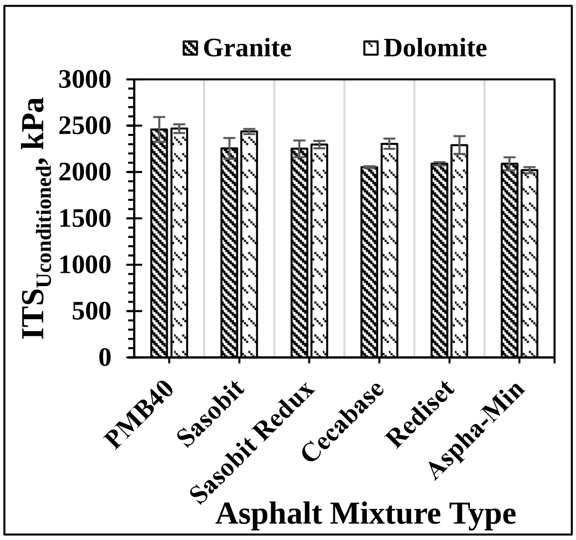
6.4.3 Indirect Tensile Strength and Tensile Strength Ratio

As indicated previously, TSR ratio is calculated using the unconditioned and conditioned values of ITS. Apart from quantifying the moisture sensitivity, ITS value also describes the tensile strength properties in asphalt mixtures. Figure 6.9 presents the variation in unconditioned and conditioned ITS values of different asphalt mixtures along with the calculated TSR. In contrast to Marshall stability, the ITS values of mixtures produced with dolomite aggregates was found to be higher than with granite aggregates. Better adhesive bond offered by dolomite increases the strain tolerance in asphalt mixtures, which must have contributed to the increase in ITS. As is expected, ITS of PMB mixtures were higher than VG 30 mixtures. This is attributed to the higher stiffness of PMB in comparison to VG 30. While the ITS values of organic WMA mixtures were comparable with the control mixtures, ITS (unconditioned as well as conditioned) of chemical and foaming WMA mixtures were found to be lower. Considering the variability, this reduction in the average values is not very significant. The effect of Sasobit was found to be binder specific. Addition of Sasobit in VG 30 increased the ITS value; however, this effect was not similar in PMB 40. The crystallized wax in Sasobit increased the stiffness of VG 30 at ambient temperature. However, in PMB 40, the polymeric network was more dominant and the effect of crystallized wax was not evident.

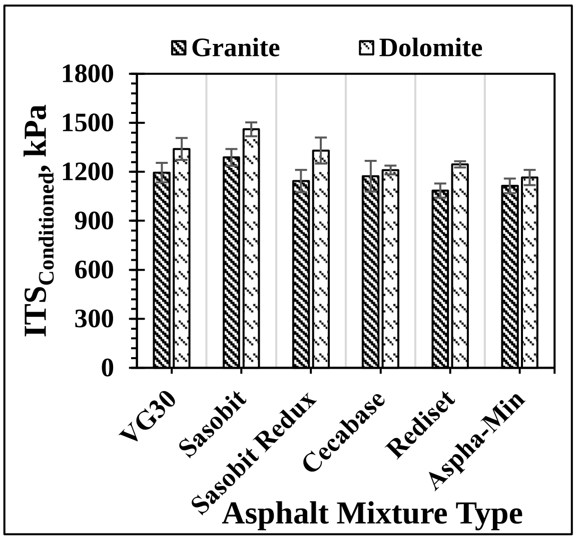
The TSR value of all the tested mixtures was found to be greater than 80%, which is usually considered as a specification limit to control moisture damage in asphalt mixtures. The TSR in chemical WMAs were found to range between 98-100%, while organic additives showed comparable results with control mixtures with an average TSR of 92%. Foaming technology led to reduction in moisture resistance in the control mixtures, though the value were higher than the minimum specification requirement.


Granite
Dolomite
ITSUconditioned, kPa
0
300
600
900
1200
1500
1800
VG30
Sasobit
Sasobit Redux
Cecabase
Rediset
Aspha-Min
Asphalt Mixture Type

(a)


Granite
Dolomite
ITSUconditioned, kPa
0
500
1000
1500
2000
2500
3000
PMB40
Sasobit
Sasobit Redux
Cecabase
Rediset
Aspha-Min
Asphalt Mixture Type

(b)


Granite
Dolomite
ITSConditioned, kPa
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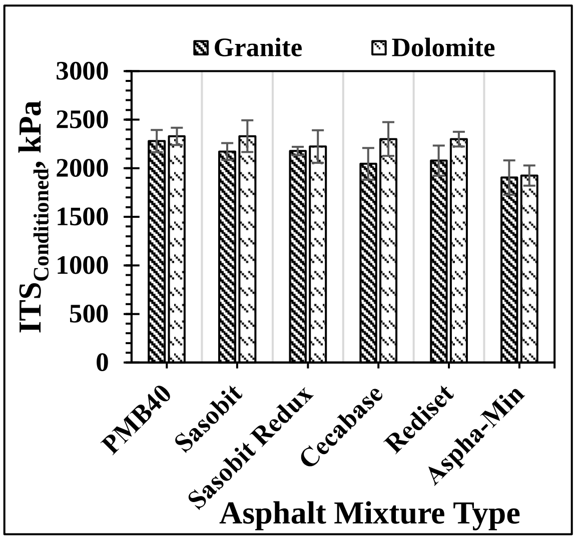
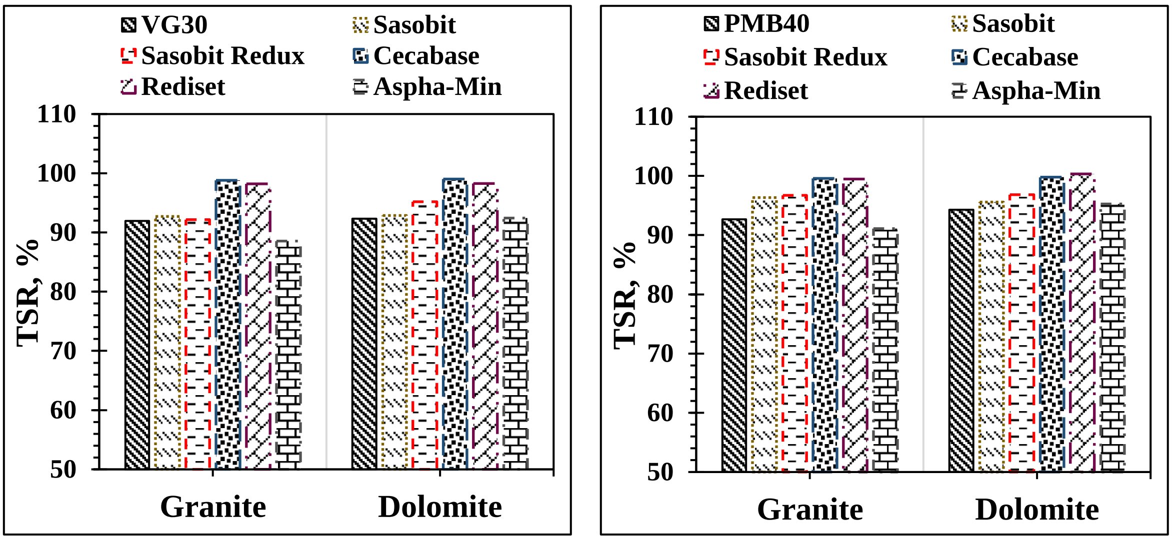
300
600
900
1200
1500
1800
VG30
Sasobit
Sasobit Redux
Cecabase
Rediset
Aspha-Min
Asphalt Mixture Type

(c)


Granite
Dolomite
ITSConditioned, kPa
0
500
1000
1500
2000
2500
3000
PMB40
Sasobit
Sasobit Redux
Cecabase
Rediset
Aspha-Min
Asphalt Mixture Type

(d)



(e) (f)

Figure 6.9. Variation in ITS for WMA mixtures prepared using different base asphalt binders (a) VG30 in unconditioned state (b) PMB40 in unconditioned state, (c) VG30 in conditioned state, (d) PMB40 in conditioned state, (e) TSR when VG30 is base asphalt binder, and (f) TSR when PMB40 is base asphalt binder

6.5 Correlation Analysis

An attempt has been made on establishing the correlations between the response of asphalt binders and mixtures considered in the present study. The correlations were defined and are shown individually corresponding to each distress (rutting, fatigue and moisture). Different parameters were used in this study to define the correlations between the asphalt binders and mixtures. However, there are no available specifications on the minimum desired value of various test parameters (such as CM from CCT, FI from Ideal CT, and BSR from PAT). Therefore, the results from other test methods along with the developed correlations were used to establish a specification limit in the present study. The results of Aspha-Min are not shown as it was not evaluated at binder's level (since the additive was directly added in the asphalt mixture).

The variation between different test parameters were modelled using a power law. For obtaining 90% reliability, the correlation curve was shifted vertically downwards such that 10% of the data (2 out of 20 here) are below this curve. It should be noted that the limiting criteria established in this study are independent of WMA technology, aggregate type, and base asphalt binder and were developed based on the data collected during the laboratory investigation. Further, more variations in chosen materials (aggregate, asphalt binder, WMA technology, or any other modifier/additive) are required to confirm the use of the developed criteria.

6.5.1 Correlating Rutting Performance

Figure 6.10 shows the relation between the CM determined at 60°C using CCT and J_{nr} value obtained using the MSCR test at 60°C. An inverse correlation exists between J_{nr} and CM values, which indicates that a lower J_{nr} and higher CM contribute to better rutting resistance. Although the CM was determined at the mixture level, it showed appreciable correlations with the J_{nr} values with an R^2 ranging from 0.68 to 0.77. This range of R^2 depends on the stress level at which the J_{nr} was measured and correlated. Though four different stress levels (0.1, 3.2, 5, and 10 kPa) were considered in this study, J_{nr} at 3.2 kPa offered the highest correlation (R^2 = 0.77) with the CM, as shown in Figure 6.11. Thus, J_{nr} at 3.2 kPa can be taken as a predictor of the rutting performance of asphalt mixtures, provided that the test temperatures at binders and mixtures levels are the same (60°C here). These interpretations are in line with the previous studies [608–610], which correlated the rutting performance of asphalt binder and mixtures.

However, a change in test temperature or the stress level at the binder's level causes severe discrepancies in the established correlations and leads to inappropriate interpretations [500,608]. For this reason, the novel rutting parameter (as described in Chapter 5), derived from the MSCR test by combining the effect of stress levels and temperature, was correlated with the CM values. As can be seen in Figure 6.12, RP of asphalt binder and CM of asphalt mixture exhibit a good correlation with an R^2 of 0.70. While both the parameters at the binder's level (J_{nr} and RP) displayed strong correlations with the rutting performance of asphalt mixtures, the RP, rather than J_{nr}, was anticipated to be a better predictor for rutting behavior. This is attributed to the independency of RP on stress and test temperature.

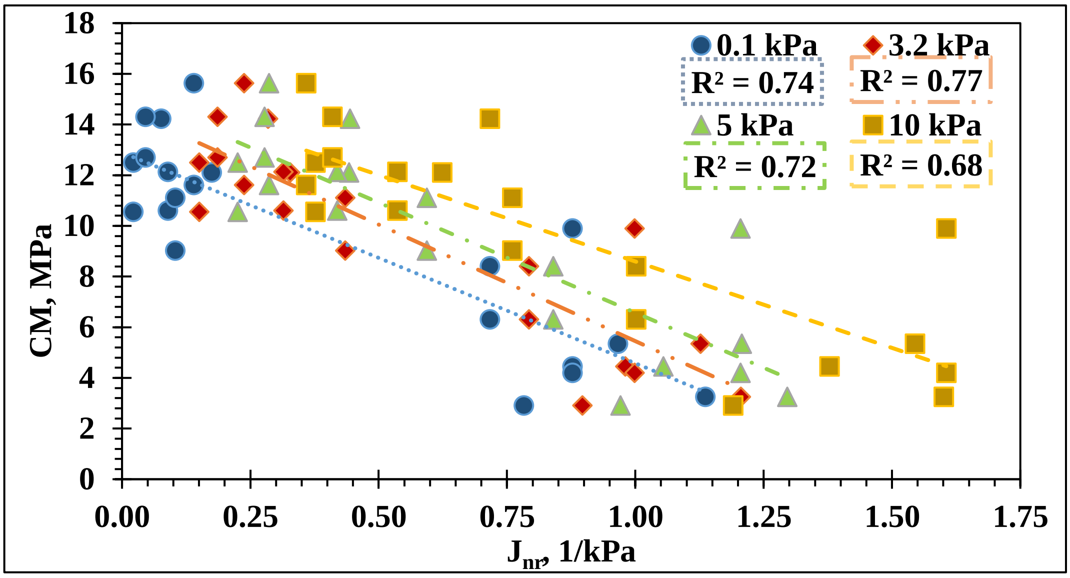


Figure 6.10. Correlation between J_{nr} at different stress levels and CM of asphalt mixtures

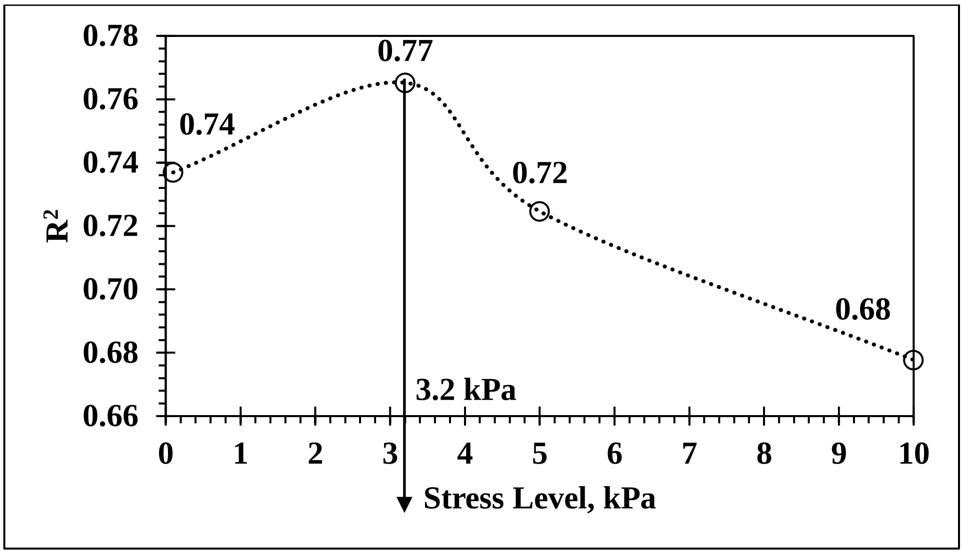


Figure 6.11. Variation in R^2 (CM-J_{nr}) with the change in stress levels

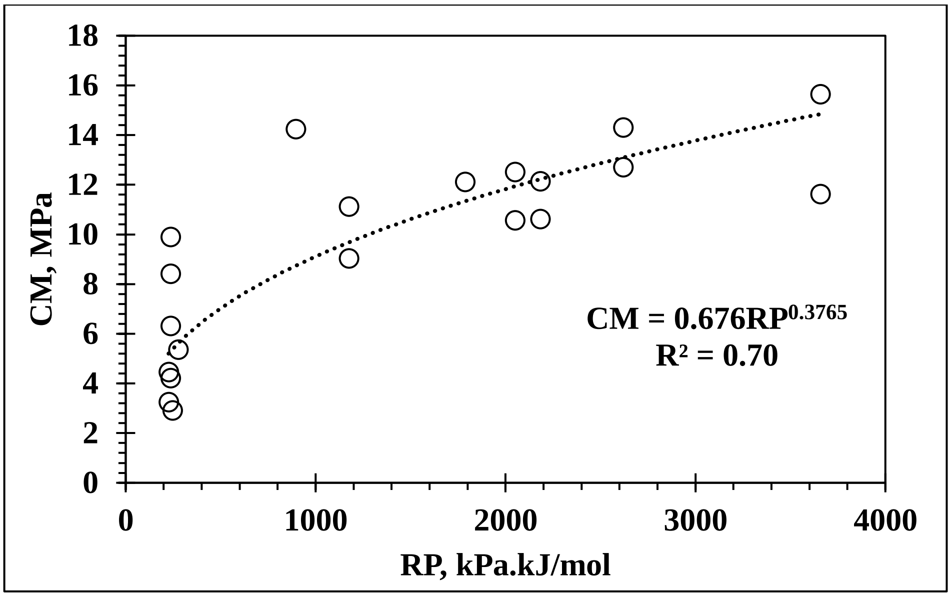


Figure 6.12. Correlation between RP and CM

6.5.1.1 Limiting Values of Rutting Test Parameters

In general, there is no reference for defining the limiting values of RP and CM required to accept any asphalt binder and mixture in terms of rutting resistance, respectively. Therefore, an effort has been made to provide their limiting values based on these correlations and specifications of AASHTO M332 [611]. As per the specifications, the asphalt binder is categorized under different traffic grades (i.e., S, H, V, and E) depending on the J_{nr} at 3.2 kPa. The specified minimum values of J_{nr} (as shown in Table 6.3) were further used to establish the limiting value of RP based on the correlation between RP and J_{nr} (Figure 6.13). A very strong correlation ($R^2 = 0.91$) exists between the J_{nr} (determined at 60°C and 3.2 kPa) and RP (independent of stress and temperature). Table 6.3 shows the predicted RP values corresponding to different traffic grades.

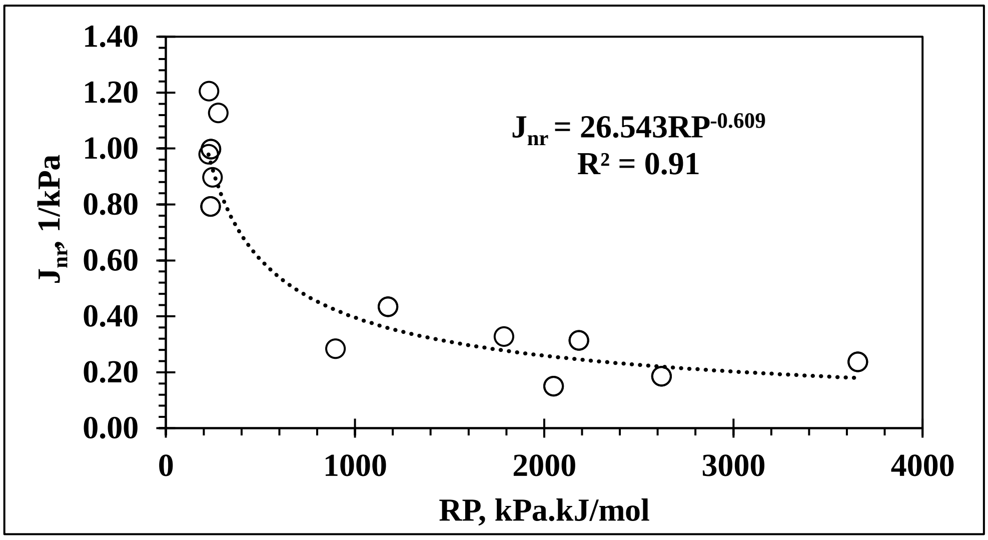


Figure 6.13. Correlation between J_{nr} and RP

The predicted value of RP was then used to define the limiting values of CM based on the correlation between them. Based on the shifted data, as shown in Figure 6.14, the limiting values of CM for S, H, V, and E were found to be 2, 3.5, 7, and 13, respectively. Table 6.3 shows the compiled form of limiting values for RP and CM. These limiting values can be used to assess the suitability of asphalt binder and mixture for different traffic conditions.

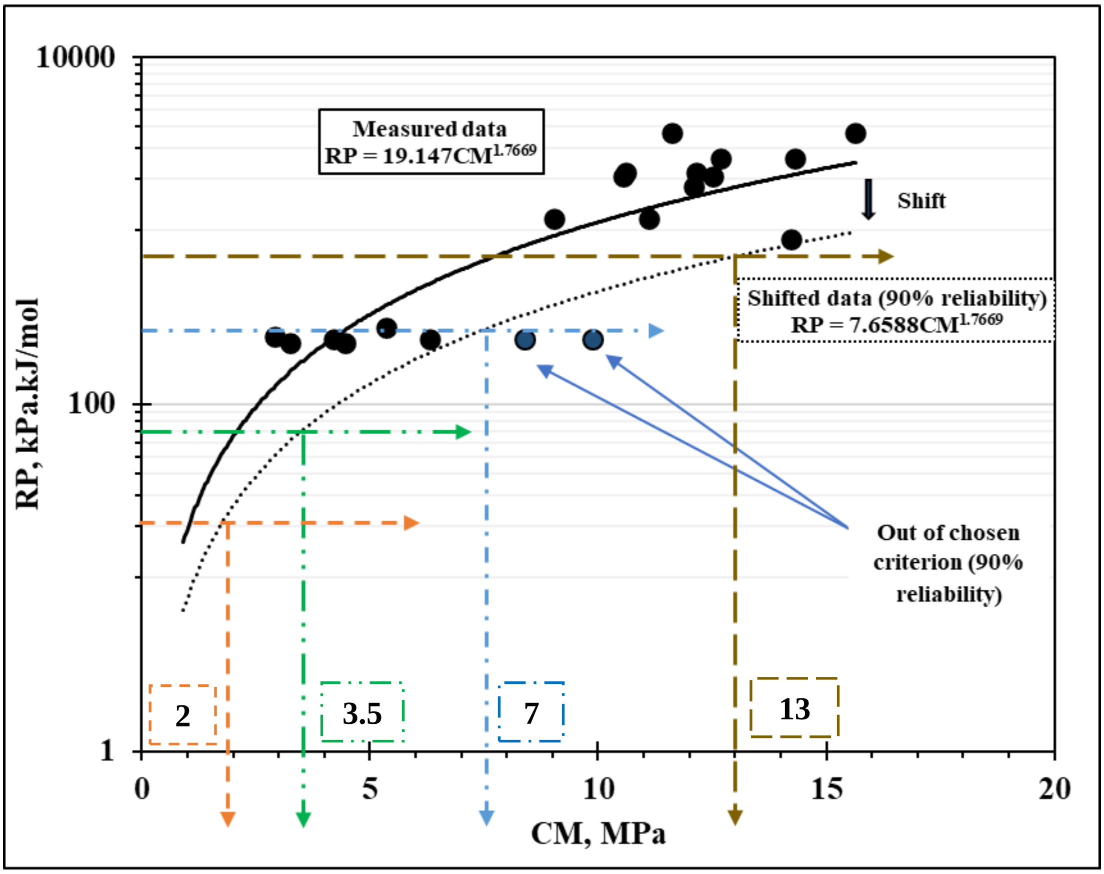


Figure 6.14. Establishment of limiting values of CM

Table 6.3. Limiting values for RP, CM, and corresponding traffic grade

J_{nr}, 1/kPa	RP, kPa.kJ/mol	CM, MPa	Traffic Grade	Specified ESAL and Traffic Condition [611]
≤4.5	≥20	≥2	S (Standard traffic)	< 10 million ESAL and standard traffic speed (> 70 kmph)
≤2	≥70	≥3.5	H (Heavy traffic)	10-30 million ESAL or slow-moving traffic (20-70 kmph)
≤1	≥220	≥7	V (Very heavy traffic)	> 30 million ESAL or standing traffic (< 20 kmph)
≤0.5	≥680	≥13	E (Extreme traffic)	> 30 million ESAL and standing traffic (< 20 kmph) such as toll plazas or port facilities

Note: ESAL refers to Equivalent single axle loads

6.5.2 Correlating Fatigue Performance

In general, the crack first initiates from the binder phase and propagates to the interface of the asphalt binder and aggregate system. Hence, an adequate and reliable fatigue characterization method/parameter for the asphalt binder could aid in selecting a suitable material (asphalt binder and WMA technology here) for attaining acceptable performance at the asphalt mixture level. Thus, the present study aimed to correlate the fatigue test parameters evaluated at the binder level with the FE and FI parameters determined at the mixture level. This would help in assessing the suitability of different test parameters and selecting the best parameter to analyze the fatigue performance.

As asphalt binder is a complex viscoelastic material, its properties are expected to change with temperature and strain values. In general, at a given temperature, asphalt binders with and without WMA additives may or may not display similar performance, in terms of failure strain. Thus, the correlation between failure strain (calculated by analyzing the LAS results at 20°C) and fatigue parameters (FE and FI, calculated by conducting an Ideal CT test on asphalt mixtures at 20°C) were developed. This is presented in Figure 6.15. The correlation of failure strain with FI was relatively better (with a 77% explained variance) than the FE parameter.

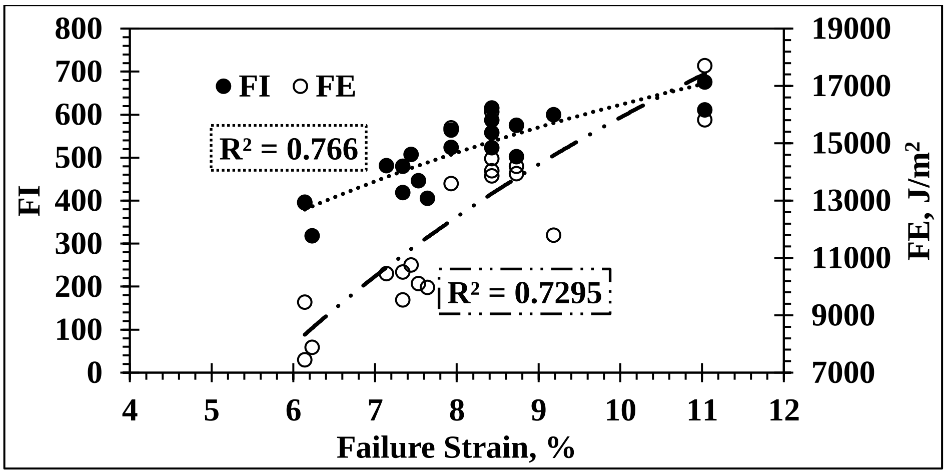


Figure 6.15. Correlation between the fatigue response of asphalt binders and mixtures

Although the relation between the fatigue life of asphalt binders and mixtures was promising, Chen et al. [612,613] indicated that the strength of the correlation is very sensitive to strain values. Several researchers [614–616] recommended 2.5% and 5% strain values for measuring the fatigue life of asphalt binders; however, it merely correlates or simulates the fatigue life of corresponding asphalt mixtures [617,618]. Therefore, it is better to evaluate the fatigue life of the asphalt binder at the strain level, where it simulates the fatigue performance of the asphalt mixture. The particular strain value was determined by correlating the fatigue life of asphalt binders (measured at different strain levels) with FI parameter. This is shown in Figure 6.16. As can be seen, the correlation (indicated by R^2) between FI and N_F tends to increase with an increase in strain level and stabilize at around 0.75 (ranging from 0.74-0.75), particularly after 10% strain value. The obtained results are in agreement with the observations reported in past literatures [612,619]. Hence, an N_F value corresponding to 10% strain value (correlation shown in Figure 6.17) is recommended for the design purpose, and the same has been used for further consideration in the present study.

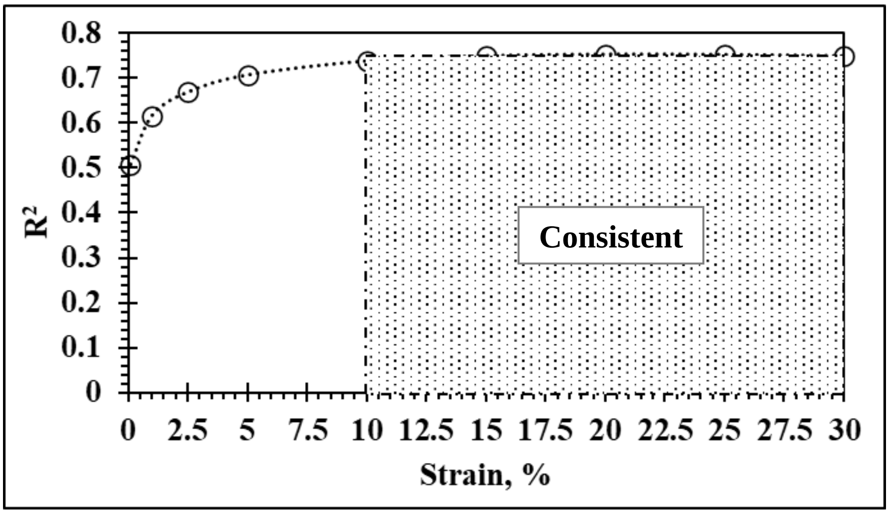


Figure 6.16. R^2 value (N_F-FI) corresponding to different strain levels

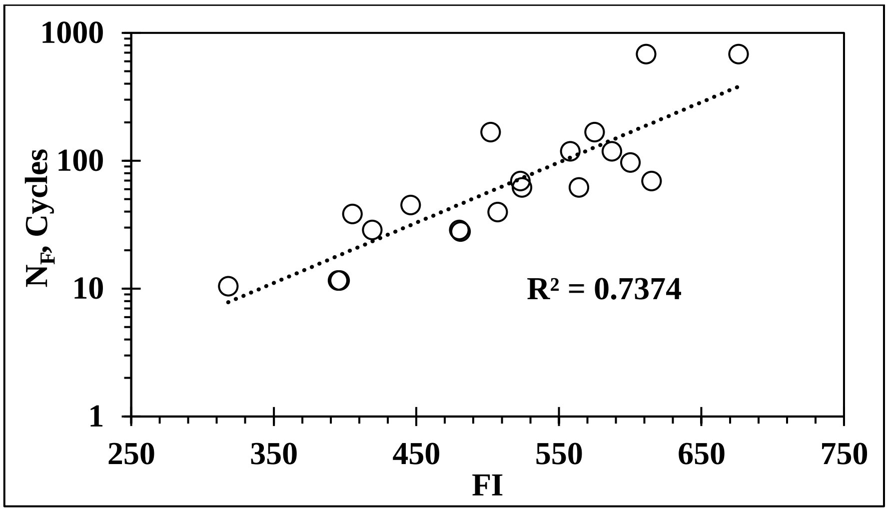


Figure 6.17. Correlation between N_F and FI at 10% strain value

6.5.2.1 Limiting Values of Fatigue Test Parameters

Alike the MSCR traffic grading concept, an attempt has been made to specify a traffic grade for all the considered asphalt mixtures based on the limiting value of FI. An indirect approach has been adopted for analyzing the limiting values of FI. Past researchers [616,620] showed that the asphalt binders could be categorized under different traffic designations same as that of MSCR (i.e., S, H, V, and E) for a strong pavement (expected strain is approximately 2.5%). N_F, in general, represents the traffic volume whose minimum value as well as corresponding traffic grades are shown in Table 6.4. It should be noted that the authors [616,620] specified the N_F values at 2.5% strain values; however, the present study found that N_F values at 10% strain value correlate well with the FI of asphalt mixtures. Thus, the traffic level was initially projected from 2.5% to 10% strain values using a power model, as shown in Figure 6.18. This projection provides the limiting values of N_F at 10% strain value corresponding to different traffic grades (second column in Table 6.5). As per the limiting values of N_F at 10% strain, all the asphalt mixtures, except DVG, GVG, and GS, passed the criteria for extreme traffic conditions.

Table 6.4. Traffic volume and designated traffic grade [620]

N_F at 2.5% strain	Traffic grade
>31000	V, E (same for very heavy and extreme traffic conditions)
>19000	H (Heavy traffic condition)
>15000	S (Standard traffic condition)

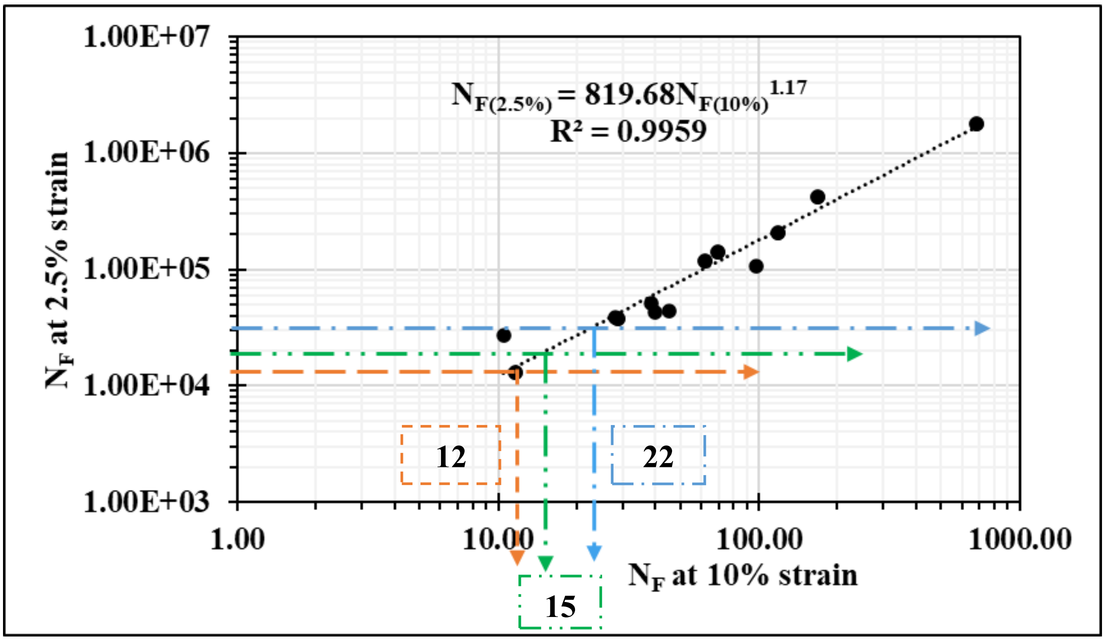


Figure 6.18. Projection of N_F from 2.5% to 10% strain values

After pertaining the traffic grades for N_F value at 10% strain, limiting values of FI were predicted using the correlation between N_F at 10% (shifted curve) and FI, as shown in Figure 6.19. Horizontal lines were plotted intersecting the predicted minimum values of N_F at 10% strain, which is 22, 15, and 12 for (V, E), H, and S traffic conditions (Table 6.5). It was found that the horizontal lines cross the shifted curve at different FI values, which can be considered as the minimum limiting FI value for characterizing a fatigue-resistant asphalt mix. Table 6.5 shows the limiting FI values and their corresponding traffic grades. The asphalt mixtures with FI values greater than 470, 430, and 415 can be categorized under (V, E), H, and S traffic conditions, respectively.

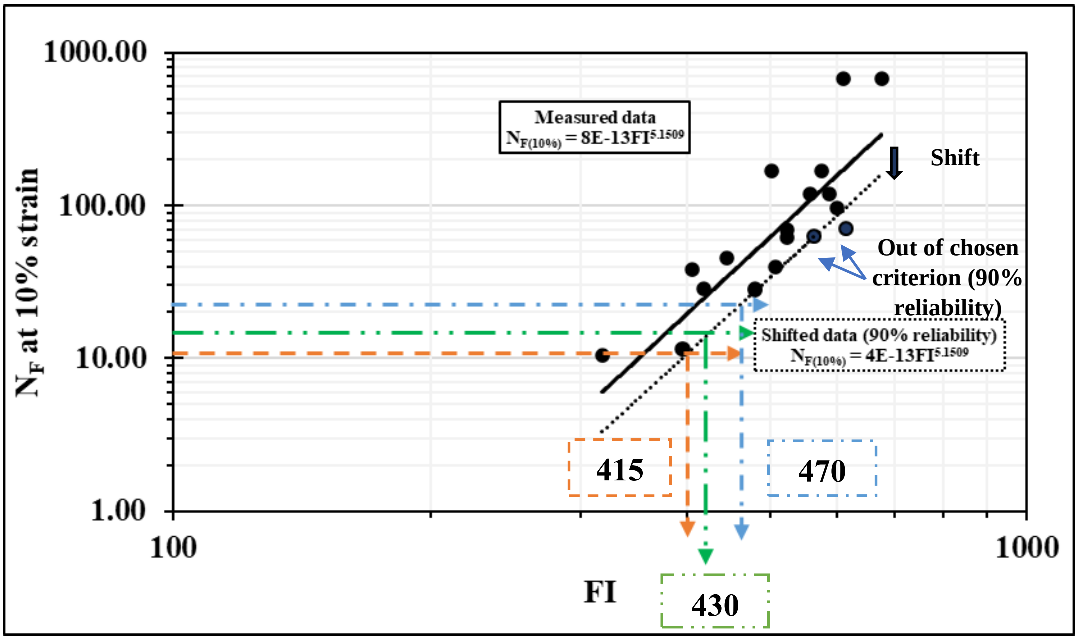


Figure 6.19. Correlation between N_F and FI for establishing the limiting FI value

Table 6.5. Limiting values for N_F, FI, and corresponding traffic grade

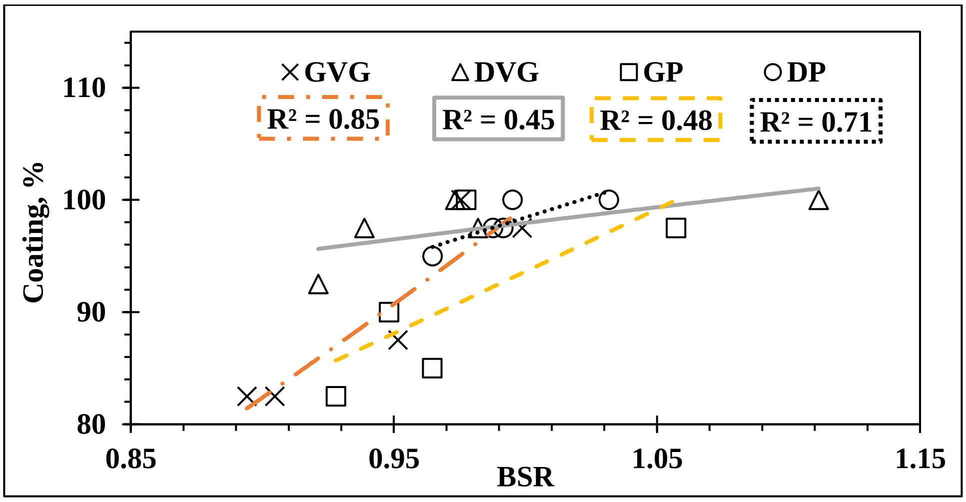
N_F at 2.5% strain, cycles	Projected N_F at 10% strain, cycles	FI	Traffic Grade
>31000	>22	>470	V, E
>19000	>15	>430	H
>15000	>12	>415	S

6.5.3 Correlating Moisture Performance

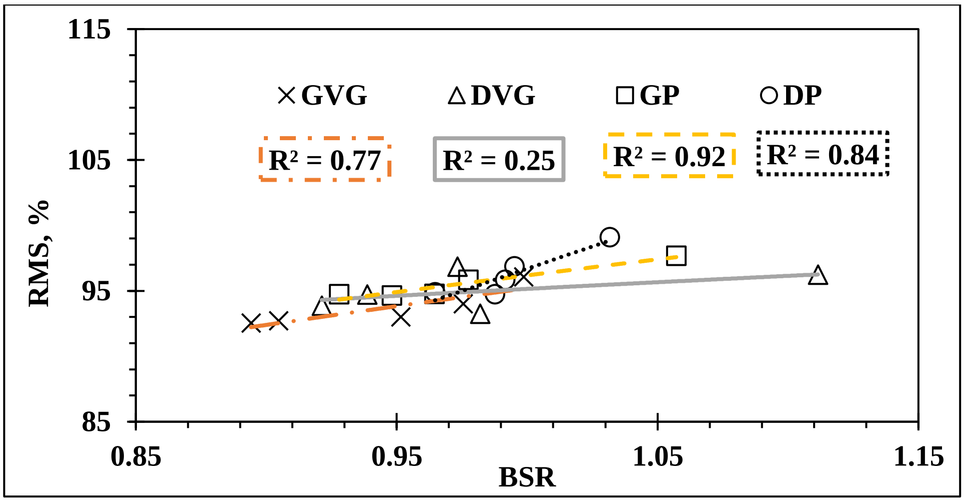
As shown in the previous section (Section 6.4), different test methods may depict variations in moisture resistance of asphalt mixtures. Also, due to the differences in loading condition, conditioning period and temperature, test temperature, and test mechanism, the assessment of cohesive/adhesive damage and the interaction between binder, aggregate, and moisture vary considerably. Nevertheless, all the test methods indicated that the performance of WMA mixtures, prepared at lower mixing and compaction temperatures, is comparable to their respective HMA mixtures in terms of moisture sensitivity. The results of the present study contradict the general belief that lower production temperatures of WMA mixtures lead to reduced resistance against moisture damage. Amongst all the test methods, PAT was able to clearly identify the effect of base asphalt binder and aggregate source on the moisture resistance of control and WMA specimens. Past studies [327,621,622] have also recommended use of BSR as a surrogate measure of moisture resistance in lieu of other methods such as BWT, RMS, and TSR. BSR being a relatively new parameter for quantification of moisture damage with no existing specification on its limiting values, it is imperative to understand its correlation with existing popular methodologies and develop minimum BSR criterion for use in specifications.

The correlations of BSR with other test methods used in this study are shown in Figure 6.20 (a-d). The correlations are shown for different material combinations, viz., (1) granite and VG30 (GVG), (2) granite and PMB40 (GP), (3) dolomite and VG30 (DVG), and (4) dolomite and PMB40 (DP). Though a positive correlation was obtained between the tests methods, for example higher values of coating percentage, RMS, and TSR lead to higher BSR values, significant variation in coefficient of determination (R^2) was obtained corresponding to different test methods. R^2 between BSR and other test parameters, i.e., RMS, TSR, and coating percentage for a particular combination, say GP, was calculated as 0.92 (very strong), 0.63 (fair), and 0.48 (weak), respectively. These discrepancies might be attributed to the variation in adopted test procedures. The correlation also depended on the combination of asphalt binder and aggregate source. For example, as shown in Figure 6.20b, the correlation between BSR-RMS for DVG combination was found to be weak ($R^2 = 0.25$), whereas the combination comprising the same aggregate source with PMB40 (DP) showed a strong correlation ($R^2 = 0.84$).

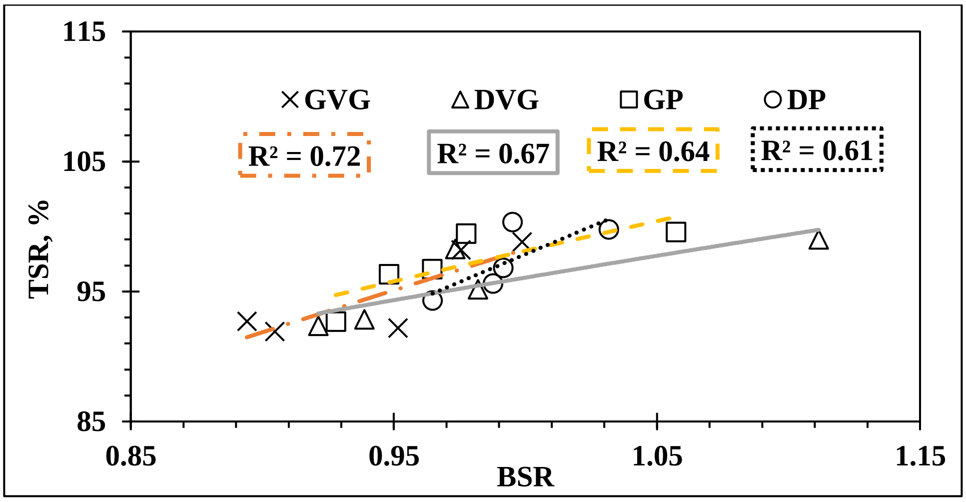
Similar variations in R^2 value were observed between other pairs of test parameters (BSR-Coating and BSR-TSR).



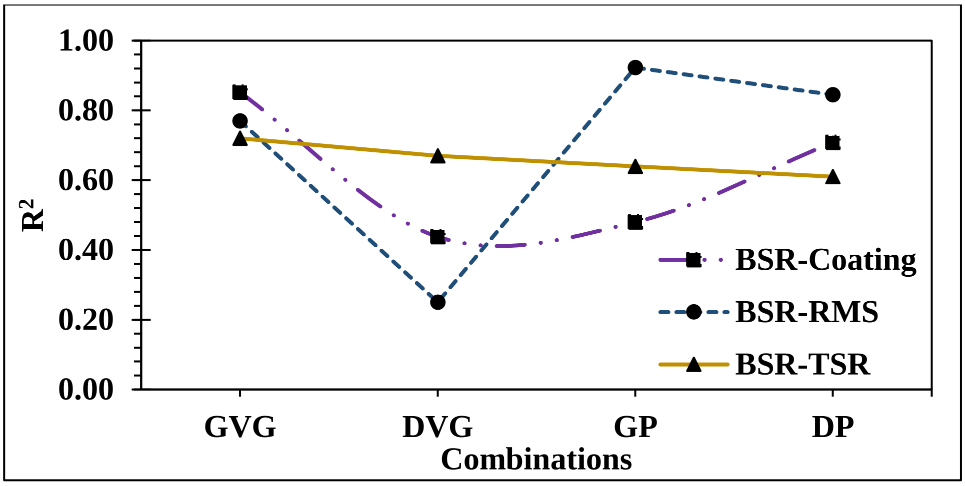
(a)



(b)



(c)



(d)

Figure 6.20. Correlation analysis between moisture test parameters (a) BSR-Coating, (b) BSR-RMS, (c) BSR-TSR, and (d) Variation in R^2 for different test combinations

6.5.3.1 Limiting Value of BSR

Figure 6.20d presents the variation in R^2 between BSR and other selected test parameters (coating, RMS, and TSR) corresponding to different combinations of asphalt binder and aggregate type. As can be seen, a consistent correlation between BSR and TSR was found which varied from 0.61 to 0.72. Other test methods showed higher variation in the values of R^2. Therefore, TSR was considered for developing specification limits of BSR. This is shown in Figure 6.21. Based on the shifted data, a horizontal line was plotted intersecting the minimum threshold value of TSR (≥80%) specified by the MoRTH [1] (and various other specifications). The horizontal line crossed the shifted curve at a BSR value of 0.7. Therefore, the asphalt mixture having BSR ≥ 0.7 is expected to be considered as a moisture-resistant mix and vice-versa.

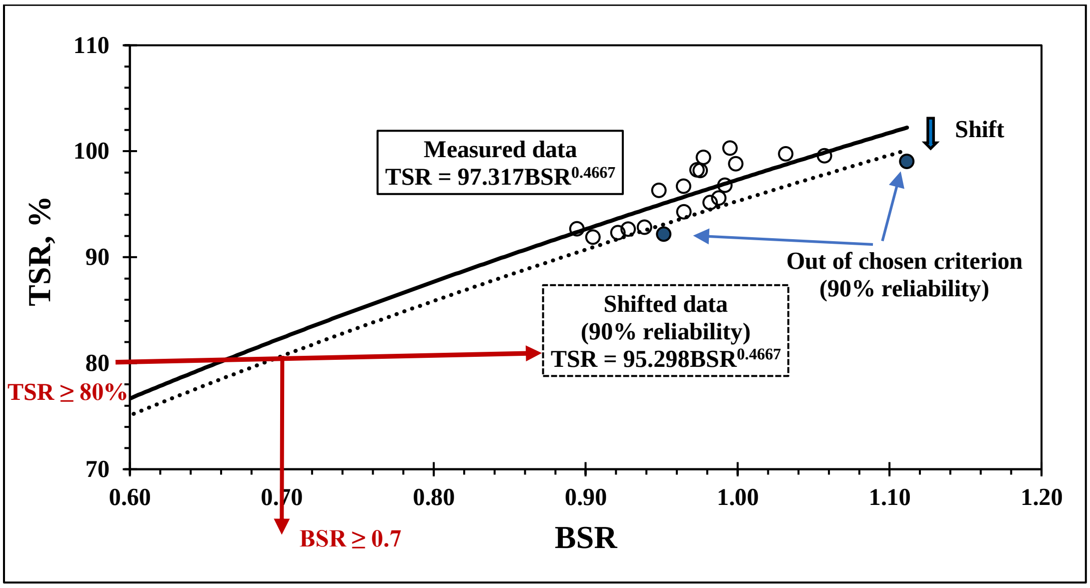


Figure 6.21. Correlation between BSR-TSR for establishing the limiting BSR value

6.6 Summary

As WMA mixtures were produced at lower temperatures, their mechanical performance is often difficult to predict and thus continues to challenge the researchers. Lower production/ageing temperature does not necessarily imply better performance for WMA mixtures. There are several other factors, such as aggregate type, binder grade, and interaction between aggregate, asphalt binder, and WMA technology, which significantly influence the mechanical performance. Previous review works [13,21,72] reported that the influence of WMA technologies on the mechanical performance is not very clear or inconclusive and thus recommends further investigations. This chapter aimed to conduct an extensive investigation on the performance of WMA mixtures against rutting, fatigue, and moisture failure. Also, different test variables such as aggregate source, base asphalt binder, and WMA technology were chosen to comprehensively examine and compare the performance of WMA mixtures with conventional HMA mixtures. Further, the correlations between the results obtained at asphalt binders and mixture level were established. Based on the laboratory investigation and parametric analysis, the following conclusions are drawn:

- Based on CCT results, organic-based WMA additive, such as Sasobit, with any combination of asphalt binder and aggregate source, offered lower accumulated strain values and higher CM, indicating higher rutting resistance. The effect was more prominent for VG30 as compared to PMB40. Chemical and foaming agents showed lower rutting performance in VG30; however, comparable performance (as HMA) was observed in PMB40-based asphalt mixtures. All the asphalt mixtures (HMA and WMA) were passed the termination criteria of 3600 cycles.
- Analysis of Ideal CT test results indicated that FE fails to capture the fatigue response of asphalt mixtures. A new fatigue parameter, FI, was proposed to determine the fatigue resistance in a more conceptual way. It was found that the asphalt mixtures prepared with dolomite aggregates exhibit higher resistance to fatigue cracking. Irrespective of base asphalt binder and aggregate source, WMA mixtures prepared with chemical agents showed better FI, followed by organic and foaming technologies. The highest improvement in fatigue resistance was identified for DR, while GS indicated the worst fatigue performance among different WMA combinations.
- The application of WMA technology was found to facilitate moisture repellent characteristics, irrespective of any test approach (% Coating, RMS, and TSR). Rediset, a chemical WMA agent, displayed superior performance against moisture damage among different WMA additives. Despite lower production temperatures, all the WMA indicated equivalent to better moisture resistance than conventional HMA.

- Overall analysis revealed that organic additives could be preferred in hot climatic regions where the rutting failure is more prominent, whereas chemical agents can be recommended at intermediate temperature conditions and heavy rainfall zones. It has to be noted that WMA technology, aggregate source, binder grade, and interaction between aggregate, asphalt binder, and WMA technology significantly influence mechanical performance.
- The results at the binder stage correlated well with the results obtained at the mixture level, irrespective of the performance (rutting, fatigue, and moisture). It is presumed that the behavior at macro level (asphalt mixture) can be predicted by testing at micro level (binder's level). The R^2 value and the correlation equation varied with the change in performance parameters. The adopted approach for correlation analysis can be used to define the correlations between different parameters and to set the limiting values of any test parameter in a more generic way.