Performance of Foamed Asphalt Stabilized Base Materials Incorporating Reclaimed Asphalt Pavement

PERFORMANCE OF FOAMED ASPHALT STABILIZED BASE MATERIALS INCORPORATING RECLAIMED ASPHALT PAVEMENT

By MATTHEW ZAMMIT, B.ENG

A Thesis Submitted to the School of Graduate Studies in Partial Fulfillment of the Requirement for

the Degree of Master of Applied Science

McMaster University ©Copyright by Matthew Zammit, July 2016

Master of Applied Science (2016) McMaster University

Civil Engineering Hamilton, Ontario

Title: Performance of Foamed Asphalt Stabilized Base Materials Incorporating Reclaimed Asphalt Pavement

Author: Matthew Zammit, B. Eng. (McMaster University)

Supervisor: Dr. P.J. Guo

Number of Pages: XII, 143

Abstract

Foamed asphalt stabilized material as a high quality granular base incorporating high percentages of fine fractionated reclaimed asphalt material is investigated. A foamed asphalt mix is designed using a fabricated asphalt foaming device. The final specimens are tested for indirect tensile strength, indirect tensile resilient modulus, triaxial resilient modulus, triaxial repeated load permanent deformation, and unconfined compression. Results are compared to those with the same aggregate blends without stabilization for triaxial resilient modulus and repeated load permanent deformation. Stabilized materials are tested in soaked and unsoaked states to establish moisture susceptibility. Foam stabilization is found to significantly improve triaxial resilient modulus in all materials as well as permanent deformation resistance in materials with high RAP content. Soaking only marginally reduces triaxial resilient modulus and the effects are lesser in materials incorporating high RAP content.

This thesis presents some new findings in foamed asphalt stabilized base materials involving the performance of fine fractionated RAP blends with Granular B. The most notable finding is that triaxial resilient modulus remains high in blends with high RAP content. Stabilization nearly doubles resilient modulus in all studied cases. Permanent deformation resistance is improved but deformation remains high in foamed asphalt stabilized base material. Five different performance tests were used for comparing soaked versus unsoaked performance. The ITS test remained the most effective method for determining moisture susceptibility. Also, the investigations were conducted using an economical, fabricated bitumen foamer. It was found that the binder must be evenly heated by keeping it constantly moving to improve control of flow rates for laboratory repeatability.

Acknowledgements

I sincerely thank my supervisor Dr. Peijun Guo for his constant help and support throughout every part of this research. There were many times when his guidance allowed me to continue with my research when I was unsure what to do. I would like to thank Dr. Emery for sharing his knowledge and expertise in practical work using foamed bitumen, and his connections with the pavement industry, especially AME who donated the materials used in this research. Also, a big thanks to Dr. Stolle for his revisions of my thesis writing.

I am very thankful for the help of the lab technicians, especially Peter Koudys for his help in the geotechnical lab with building and troubleshooting the foamer, equipment setup, and testing procedures. Kent Wheeler and Paul Heerema provided much help with equipment used when creating specimens at the applied dynamics lab.

I would like to thank MITACS for their help in funding this research. Also, thank you to HCI for the financial support through summer coop positions which they provided me during undergraduate education, and for sparking my interest in the pavement industry.

Last but not least I owe a big thank you to my family, friends, and the Westdale wastrels cycling group for support and motivation. All of whom helped to relieve the frustrations associated with the many struggles encountered during this research.

Contents

List of Figures

List of Tables

Index of Terms

1 Introduction

1.1 Background

There is an increasing interest in recycled pavement materials because of their economic and environmental benefits associated with the reduced use of crude oil [1]. Much research has been put into the use of reclaimed asphalt pavement (RAP) materials to be used as aggregate base, subbase, and even as a fraction of aggregate for new hot mix asphalt (HMA)[2]. The discovery of asphalt foaming by Csanyi[3] in 1957 opened up the possibility for a material that allows the use of less suitable materials. Foamed asphalt stabilization has the potential to be an effective, environmentally and economically friendly way to rehabilitate pavement.

The foaming of bitumen changes its dispersion characteristics when mixing with aggregates. The bubbles burst, sending droplets of bitumen throughout the mix and coating fine particles entirely. This dispersion takes place at room temperature thereby reducing the energy needed during mixing and compaction. The final product is bitumen stabilized material (BSM), which is a partially unbound mixture, since particles are not completely coated with asphalt binder as they are in HMA. This partial coating increases the cohesion between particles, although it does not significantly improve shear properties. The final material resembles a strong granular material with cohesion [4]. A common failure mechanism for BSM is permanent deformation.

1.2 Research Objectives

The objective of this research was to determine the potential of a foamed asphalt stabilized material to be used as a high quality granular base incorporating lower quality aggregates. The focus was on aggregates containing high percentages of reclaimed asphalt material. In addition to this, new asphalt foaming equipment was fabricated to reduce costs associated with lab-scale foaming research.

Previous research at McMaster using non-stabilized aggregates with high RAP content revealed that increasing RAP content up to as much as 75% RAP, tended to increase resilient modulus (M_r), however permanent deformation also simultaneously increased [5]. This research was aimed at further exploring these phenomena, by studying the effect that bitumen stabilization had on the resilient modulus and permanent deformation of aggregate materials containing high RAP content. In addition, successive phases aimed to optimize the final designs by examining the influence of compaction effort and fine material content.

New aggregates in this study, together with previous McMaster studies on resilient modulus and permanent deformation of granular aggregate blends with increasing RAP content, were used as references to investigate the effect of foamed asphalt stabilization. Therefore, testing began at the aggregate stage, with preliminary testing on gradation, plasticity index of fine aggregates, aggregate blending, and optimum moisture content. Furthermore, the resilient modulus and permanent deformation tests were carried out on both aggregates with and without foamed asphalt stabilization to quantify the effectiveness of stabilization.

An entire BSM mix design was completed by considering optimized aggregate structure and optimized asphalt binder content, in terms of indirect tensile strength testing of gyratory compacted specimens. After that, the optimum stabilized mixes were made at varying RAP contents. Specimens compacted using these mixes were tested to determine their resilient modulus and permanent deformation characteristics.

1.3 Thesis Outline

This thesis begins with a literature review in Chapter 2, addressing foamed bitumen stabilized material and the materials which it is composed of, constituent optimization, mix design process, equipment used, specimen preparation, and performance tests.

Chapter 3 includes the experimental studies. It starts with outlining the test program and summary of tests completed. The following subsections describe each test including preliminary aggregate testing and blending to final performance test setups, material and sample preparation, and procedures.

Chapter 4 provides a detailed look into the mix design process as completed. It also includes the mix design results, analysis and discussion.

Chapter 5 summarizes performance test results and includes a comparison and analysis of those results.

Chapter 6 presents a final summary of the research including the conclusions of the findings, and recommendations for further research.

2 Literature Review

2.1 Foamed Asphalt

Foamed asphalt makes it possible to use parent materials which are considered unuseable for HMA. This is well described by Professor Csanyi [3]:

"When an asphalt cement is foamed, it increases tremendously in volume, its viscosity is materially reduced, and it becomes much softer at lower temperatures. Foaming also introduces energy into the asphalt, thereby modifying its surface tension and making it more sticky. It increases its ability to displace moisture from a surface and to coat a surface with a comparatively thin film. When the foam breaks and the energy is dissipated, the asphalt cement recovers its original properties with no change in its chemical composition. Through modified surface tension, cold, wet aggregates or soils can be used, and wet clayey lumps of soil can be permeated with asphalt. Because of the ability of foamed asphalt to coat mineral particles with thin films, the use of ungraded local aggregates in mixes becomes possible and the production of mastics of mineral dusts and asphalt is also feasible. Thus, through the use of asphalt cements as a foam, materials heretofore considered unsuitable can now be used in the preparation of mixes for stabilized bases and surfacing for low-cost road construction"[3].

2.2 Bitumen Foaming

Foamed asphalt is created when bitumen is heated between 160 and 180 degrees Celsius and cool water is applied to it at a controlled rate. Bitumen of various performance grades can be used for foaming, however the Asphalt Academy states that softer bitumen typically has better foaming characteristics [6].

The quality of bitumen foam is characterized by its expansion ratio (ER) and its half life (HL). Expansion ratio is the ratio of the volume of fully expanded bitumen compared to its collapsed state, while the half life is the amount of time that it takes for the expanded volume to reduce by half. The water application rate and bitumen temperature have a significant effect on the foam properties. Increasing bitumen temperature creates better quality foam. Increasing water application rate increases expansion ratio and lowers half life. This application rate must be optimized for the grade of bitumen being used [6].

The minimum required value of ER and HL have been set by the Asphalt Academy and Wirtgen, with values differing slightly. The Asphalt Academy [6] suggests that ER should be greater than 10 times and HL greater than 6 seconds when the temperature of aggregate ($T_{aggregate}$) is between 10 and 25 degrees Celsius. Alternatively they suggest an ER of 8 and HL of 6 when $T_{aggregate}$ is higher than 25 degrees Celsius. On the other hand, Wirtgen suggests an ER of 10 times and HL of 8 seconds when $T_{aggregate}$ is only 10-15 degrees Celsius, and an ER of 10 and HL of 6 when $T_{aggregate}$ is above 15 degrees Celsius [4]. The recommended values of ER and HL are summarized in Table 2.2.1.

Table 2.2.1: Comparison of Foam Characteristics

	Asphalt Academy		Wirtgen	
Aggregate Temperature (deg. C)	10 to 25	>25	10 to 15	>15
Expansion ratio (Times)	10	8	10	8
Half Life (Seconds)	6	6	8	6

2.3 Aggregate Properties

Any aggregate will perform well when used in the production of a bitumen stabilised material as long as it meets certain requirements discussed in this section. The most important properties are the aggregate gradation, plasticity index of the fine fraction of aggregates, and the moisture-density relationship. The Asphalt Academy [6] and Wirtgen [4] have recommended maximum plasticity index and the range of gradation suitable for BSM. Table 2.3.1 and Figure 2.3.1 display the gradations recommended by Asphalt Academy and Wirtgen.

Table 2.3.1: Reccommended Aggregate Gradations [6] & **[4]**

	Percent passing				
	Asphalt Academy		Wirtgen		
Sieve size (mm)	Ideal	Less suitable	Ideal	less suitable (gravel)	typical RAP
50	100		100	100	100
37.5	87 to 100		87 to 100	100	85
26.5	77 to 100	100	76 to 100	100	72
19	66 to 99	99 to 100	65 to 100	100	60
13.2	67 to 87	87 to 100	55 to 90	100	50
9.5	49 to 74	74 to 100	48 to 80	100	42
6.7	40 to 62	62 to 100	41 to 70	100	35
4.75	35 to 56	56 to 95	35 to 62	88	28
2.36	25 to 42	42 to 78	25 to 47	68	18
1.18	18 to 33	33 to 65	18 to 36	53	11
0.6	14 to 28	28 to 54	13 to 28	42	7
0.425	12 to 26	26 to 50	11 to 25	38	5
0.3	10 to 24	24 to 43	9 to 22	34	4
0.15	7 to 17	17 to 30	6 to 17	27	2
0.075	4 to 10	10 to 20	4 to 12	20	1

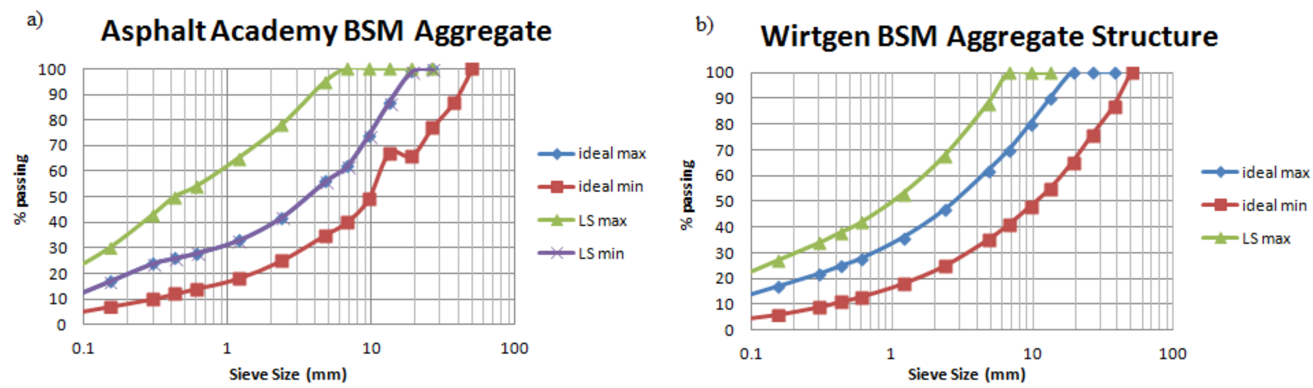


Figure 2.3.1: Comparison of aggregate structures as recommended for BSM-Foam material by a) Asphalt Academy[6] and b) Wirtgen[4] (LS is abbreviated from less suitable)

The percentage of fines passing the 0.075mm (#200) sieve is particularly important in the gradation because fines assist with foamed asphalt dispersion. Foamed bitumen stabilised materials are partially bound because only the fine aggregates are fully coated by asphalt foam. These fines disperse around the mix to “spot weld” the larger aggregates together [6]. For this reason, a minimum of 4% fines is recommended[6]. There is also a problem with having too many fines as they may accumulate in large clumps of mineral filler that is mostly uncoated by bitumen. This prevents bitumen droplets from contacting large aggregates to improve strength, as described by Fu [7]. An example can be seen in Figure 2.3.2 which displays where a crack would propagate based on filler content. Note that in Figure 2.3.2 (a) and (c), having excess and minimal fines content respectively, the foamed bitumen does not provide much strength since the crack can propagate through the mineral filler phase or void spaces by avoiding the bitumen droplets.

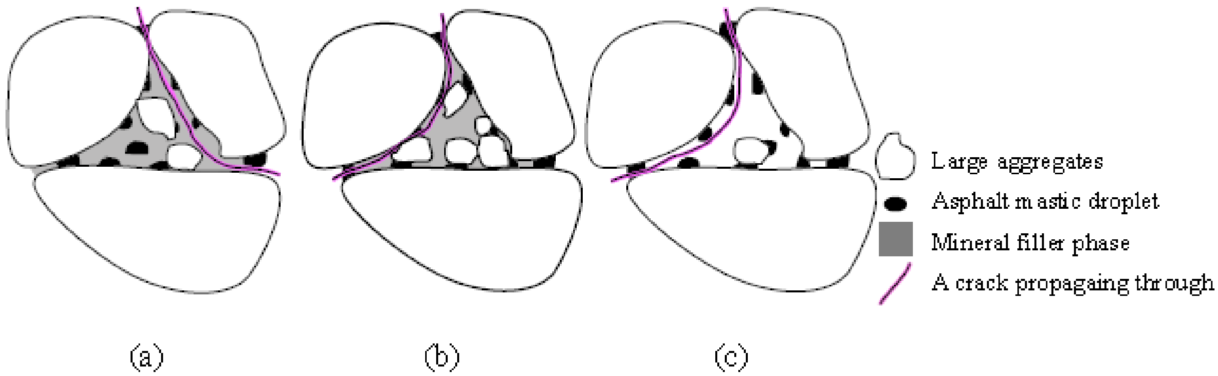


Figure 2.3.2: Crack Propagation with relation to mineral filler content in BSM [7]

Aside from fines, the fraction of aggregate particles with sizes between 0.075mm and 4.75mm is the next important consideration. In order to reduce the volume of voids in mineral aggregate and to obtain a dense material, Emery [1] has suggested a preferred gradation, as shown in Table 2.3.2. For a BSM foam mix, his gradation closely resembles Wirtgen and the Asphalt Academy. However, it is slightly finer throughout, with focus on the fraction passing 0.075mm. Emery believes that the fine fraction passing 0.075mm should be a minimum of 7% rather than 4% as the Asphalt Academy and Wirtgen suggest.

Table 2.3.2: Preferred gradation (Emery)[1]

Preferred Overall Gradation		
	% passing	
Sieve size (mm)	min	max
37.5	100	
19	60	100
4.75	30	60
0.6	15	30
0.075	7	15

The plasticity index of the fine fraction of aggregates also plays an important role for the behaviour of BSM. Both the Asphalt Academy and Wirtgen agree that plasticity index in a foamed BSM should be limited to 10. The plasticity index is an important consideration in BSM because it is a property of fine particles in the material. A high plasticity index indicates that fine particles tend to be cohesive and lump together in the presence of moisture. If the PI is greater than 10, it is likely that the fine particles will lump together and therefore not effectively disperse the bitumen [4].

2.4 Reclaimed Asphalt Pavement (RAP)

RAP has been widely used in pavement construction, either in HMA or in unbound granular base/subbase layers. It is obtained by milling or pulverising the existing pavement to a desired depth. It can either be transported to a facility for further processing, or used directly on site. In general, RAP is a highly variable material that depends on a number of factors pertaining to the in-situ pavement quality and removal processes [8].

Additional plant processing is encouraged after the initial acquisition of the RAP to reduce variability. The screening or fractionation process helps to remove oversize particles or reduce the maximum aggregate size, and can produce fine or coarse fractionated RAP, as shown in Figure 2.4.1. Crushing is another method used in plant processing to reduce maximum aggregate size. This process tends to increase the amount of dust in the mix as well [2].

Figure 2.4.1: Fine (left) and coarse (right) fractionated RAP[2]

RAP itself has two important characteristics that depend on the existing pavement conditions, namely gradation and activity of residual binder. When completing in-situ reclamation, the gradation of the RAP material can be greatly affected by the abilities and concern of those doing the work. The gradation is mainly affected by pavement composition, pavement condition, climate exposure of existing asphalt, depth of milling, speed of milling machine, speed of milling drum rotation, type of milling drum and condition of tools, and direction of milling [4]. Plant processes such as crushing and fractionation reduce variability caused by imperfect in-situ processes.

RAP is ground from the existing asphalt pavement which was designed for a specific use. Different asphalt pavements have different designed gradations, and similarly have different layers of asphalt in a single pavement structure. The wearing course is typically of higher quality due to its need for smooth, wear resistant properties, which makes for high quality RAP as well. The milling process inhibits some degradation of the aggregates yielding an ultimately finer material than crushing. According to Chesner et al.[8] the percent passing the 2.36 mm sieve is expected to increase from 41-

69% to 52-72% and the percent passing 0.075 mm sieve increases from 6-10% to 8-12% after the milling process.

The binder in the RAP material hardens after years of use due to oxidation and weathering. This makes it important to determine whether the binder is active or inactive. If the binder still retains some adhesive properties it is referred to as active material. Typically when RAP is used for untreated or stabilized granular base/subbase courses, the residual binder must be inactive because an active binder makes it difficult to compact the RAP [4].

When the physical properties of RAP are considered, the typical maximum dry density is approximately 1600 to 2000 kg/m^3, slightly lower than that of clean aggregate. Crushed RAP retains more water than clean aggregate when stockpiled, typically around 5% and as much as 7-8% after rainy periods. Depending on the gradation, the residual binder content of RAP generally varies in the range of 3-7% [8].

2.4.1 Considerations for Foaming

When treating RAP with foamed bitumen, the amount of fine particles (passing 0.075mm sieve) can be lessened to 1% rather than 4% as recommended for new aggregate materials. This is because the foamed bitumen splinters are able to adhere to the residual bitumen in the RAP. When residual bitumen in the RAP is inactive, adding foamed bitumen creates a non-continuously bound material. When the residual bitumen is active the material should be blended with 30% by volume of graded crushed stone to achieve a non-continuously bound mix. Otherwise, foam stabilized RAP that is active exhibits continuously bound material properties. Typically, RAP material is fairly coarse, allowing it to be effectively stabilized with low amounts of bitumen around 1.6 to 2.2% by mass[4].

In hot climates, additional care must be taken when stabilizing RAP material with foamed bitumen. Shear properties of the mix should be determined, and axle loads allowed on the road should be controlled at weigh stations. Permanent deformation is a common failure mode for BSM, which is typically amplified in hot climates. In addition, all RAP material in hot climates should be blended with 15% crusher dust or 30% graded crushed stone by volume, to ensure the non-continuous nature of a BSM [4].

2.5 Optimum Bitumen Content

In the mix design process, the optimum bitumen content must be determined by a trial and error method where specimens are created at different bitumen contents. Indirect tensile tests and triaxial compression tests are carried out to determine the indirect tensile strength, cohesion, and internal friction angle. The moisture sensitivity of the mix is also examined. The optimal mix design is chosen according to the mechanical test results [6].

There are three mix design levels, which render an increasingly more optimum mix as levels increase. The mix design level is determined based on design traffic. In mix design level 1, 100 mm Marshall test specimens are created and tested for ITS on dry and soaked specimens, and the optimum is chosen. At mix design level 2, 150mm diameter by 127mm high specimens are created, and a more rigorous curing procedure is followed for a more accurate indicator of strength. Curing methods are discussed in later sections of the literature review. ITS tests are completed on dry and soaked specimens and the optimum is chosen. At level 3 mix design, 150mm diameter by 300mm high specimens are created, cured and used for triaxial testing. Figure 2.5.1 shows a detailed flow chart of the procedure [6].

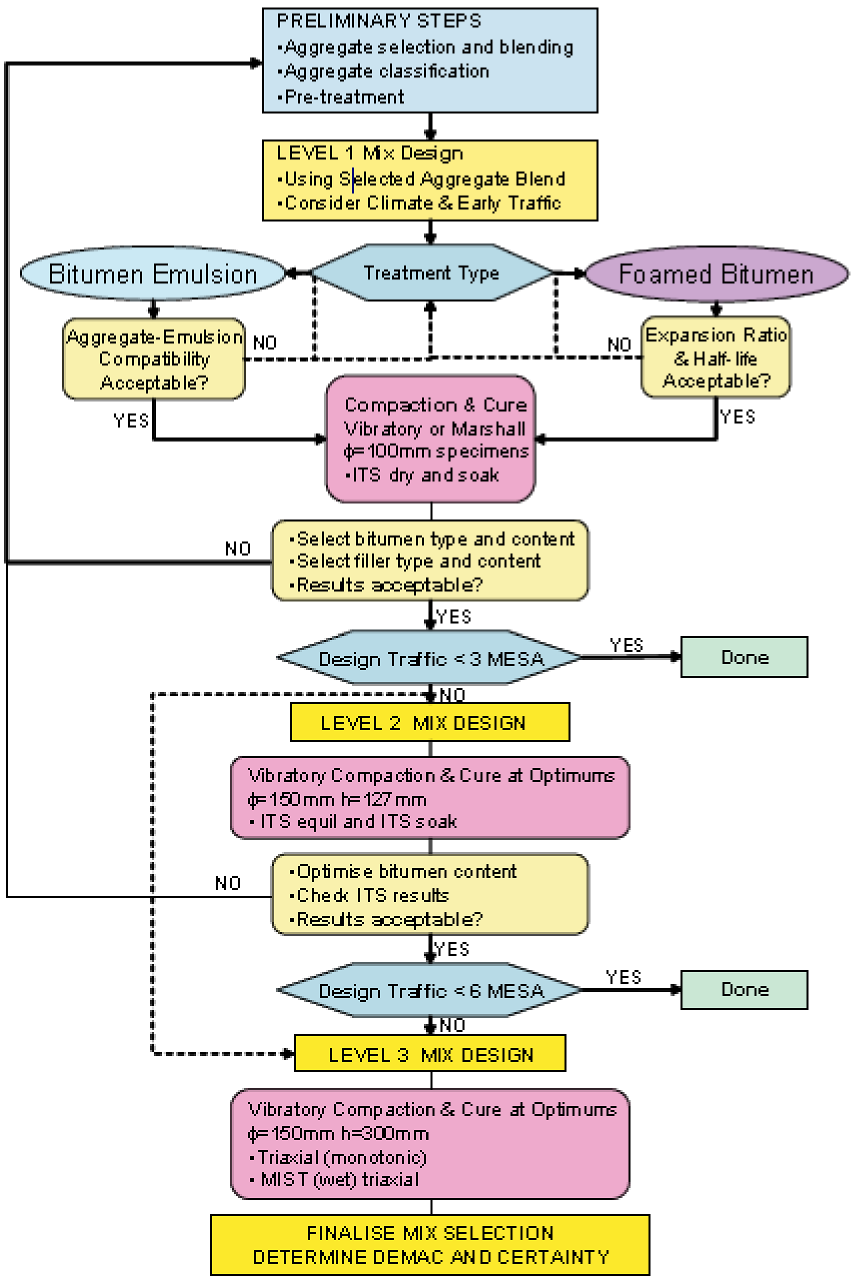


Figure 2.5.1: Mix design procedure[6]

Emery [1] proposes an equation that has been used to predict optimum bitumen contents for over 80 projects since 1997. This equation can be used as a reasonable estimate for the optimum bitumen contents when laboratory test data are not available to identify the optimum binder content. This equation is presented in Figure 2.5.2.

Estimated foamed asphalt content:

Moisture susceptible mix (TSR test)

$$\text{Foamed asphalt (percent)} = \left(\frac{\text{percent aggregate}}{100} \times 4.5\right) + \left(\frac{\text{percent RAP}}{100} \times 1.5\right)$$

(plus hydrated lime (or equivalent slaked quick lime) of 0.5 to 1.0 percent)

non moisture susceptible mix (TSR test)

$$\text{Foamed asphalt (percent)} = \left(\frac{\text{percent aggregate}}{100} \times 4.0\right) + \left(\frac{\text{percent RAP}}{100} \times 1.5\right)$$

Figure 2.5.2: Equation for estimating optimum foamed asphalt content[1]

2.6 Mixing

The mixing stage has a significant effect on the strength of a BSM mix. It is important to consider both the type of mixer used and the way mix constituents are added to the mixing bowl.

In most foamed asphalt applications, when foaming the material, the bitumen expands by a factor of 10 and reduces half of its volume in approximately 8 seconds. From the required ER and HL, bitumen only stays in the foamed state for a short period of time (<1min). This means that in order to mix the materials in a foamed state, the foam must be injected directly into the mixer while the aggregates are being agitated [6].

The type of mixer selected should be able to create a thoroughly mixed specimen, and it should also be able to accurately resemble the mixing process in the field. The optimal mixer as described by the Asphalt Academy and Wirtgen is a pugmill type mixer, which incorporates two rotating shafts equipped with blades, and resembles the mixing near the rotating drum of the recycler. The pugmill mixer makes the aggregates airborne and effectively allows the bitumen droplets to disperse through the mix as the bubbles burst. Optimal mixing is completed within 20-30 seconds [6].

The aggregate moisture content during mixing of an FASB material has a significant effect on the strength of the resulting product. The mixing moisture content (MMC) refers to the water content in the aggregate mix, not to be confused with fluid content which also includes bitumen fluids (used in BSM-Emulsion). Ruckel et al. [9] states that higher moisture contents would free the fine particles in the mix to make them accessible by foamed asphalt particles so that they may be coated and therefore improve the mix strength. Fu et al. [10] study the effect of mixing moisture content on mix properties and finds that there is an optimum range of mixing moisture contents where the foamed bitumen particles are best dispersed to provide maximum strength.

At high mixing moisture contents, greater than the optimum compaction moisture content (CMC) by modified Proctor compaction, finer particles are attracted to each other to form a paste-like substance that agglomerates and attracts to larger particles. This agglomeration effectively removes the mechanism by which foamed bitumen particles disperse, leaving large globules of bitumen in the mix. When the bitumen does not evenly disperse in the mix, the strength of the final product is significantly reduced. Fu et al.[10]

characterize this dispersion of asphalt as fracture face asphalt coverage (FFAC), which is the ratio of asphalt mastic area to the total area of a fracture face.

At low mixing moisture contents, far below the optimum CMC, the mix has good asphalt binder distribution. However the particles are too dry to achieve the appropriate density. This results in a lower strength of the final product. The best estimated mixing moisture content recommended based on observation by Fu et al.[10] is between 75-90% of the optimum CMC by modified Proctor compaction. The results of Fu's study on two pulverised asphalt pavements (PAP–A and –B) are shown in Table 2.6.1 and Table 2.6.2.

Table 2.6.1: Mixing and compaction moisture contents for each mix (Fu et al.) [10]

PAP	Mix	Target MMC (%)	Measured MMC (%)	Target CMC (%)	Measured CMC (%)
PAP-A	I	3.0	2.6	6.0	5.9
	II	4.0	4.0	6.0	6.5
	III	5.0	4.6	6.0	6.0
	IV	6.0	6.3	6.0	6.3
	V	7.0	7.1	7.0	7.1
PAP-B	I	3.0	2.5	5.0	4.9
	II	4.0	3.7	5.0	5.1
	III	5.0	4.8	5.0	4.8
	IV	6.0	6	6.0	6.0

Table 2.6.2: Strenth and stiffness test results (Fu et al.) [10]

Mix[a]		ITS-unsoaked		ITS-soaked			TxRM-soaked			
		ITS (kPa)	FFAC[b] (%)	ITS (kPa)	FFAC (%)	UCS (kPa)	k_1	k_T	k_2	k_3
PAP-A	I	564	9.0	104	9.8	643	4,208	−0.106	0.39	−0.15
	II	596	9.0	98	9.2	639	4,042	−0.104	0.41	−0.17
	III	563	7.7	102	8.2	723	4,407	−0.102	0.40	−0.16
	IV	449	8.8	65	4.4	516	2,914	−0.088	0.50	−0.20
	V	481	5.8	79	5.6	483	3,604	−0.094	0.44	−0.18
PAP-B	I	752	29.8	162	25.3	703	5,827	−0.103	0.28	−0.11
	II	696	34.0	175	33.1	829	6,949	−0.120	0.23	−0.11
	III	780	34.2	146	27.7	790	6,650	−0.116	0.24	−0.10
	IV	773	32.9	153	28.3	924	7,216	−0.113	0.24	−0.10

[a]Mixing and CMCs shown in Table 2. MMC increases with mix number.

[b]FFAC measured for soaked tested specimens (FFAC-ITS-soaked) is more preferable as explained in Fu et al. (2008a).

Subsequent studies by Khosravifar [11] do not replicate the same results regarding MMC. An increase in MMC does not necessarily reduce ITS values, which leads to the notion that the fines content in a mix has an effect on MMC. Khosravifar's results are shown in Table 2.6.3. The aggregates which Fu considered, had larger fines contents (8% and 20%) which would have made the mix more susceptible to mixing moisture contents [11]. This leads to poorer asphalt dispersion and lowered ITS values for a smaller increase in MMC. Regardless of the mix, the mixing moisture content of 75-90% of optimum CMC did not detriment Khosravifar's mix strengths. Wirtgen mix design method recommends 70-90% of optimum CMC (modified proctor) [4], whereas the Asphalt Academy recommends 65-85% of the optimum CMC obtained from modified Proctor compaction [6].

Table 2.6.3: Effect of MMC on ITS and dry density (Khosravifar) [11]

Target MMC (%)	7.0 (90% of OMC)	8.1 (104% of OMC)	7.0 (90% of OMC)	8.1 (104% of OMC)
Target CMC (%)	7.0 (90% of OMC)	8.1 (104% of OMC)	5.7 (73% of OMC)	5.7 (73% of OMC)
Unsoaked ITS (psi)	69.0	49.0	65.0	70.0
Soaked ITS (psi)	41.0	37.0	30.0	32.0
Dry Density (pcf)	131.8	129.8	131.8	133.8

2.7 Compaction

Senior et al.[12] report two case studies, one in Northeastern and one in Northwestern Ontario, of highway projects which failed earlier than expected. Both of these studies were conducted on roads which were reconstructed using full depth reclamation (FDR) as some part of the granular base course. It is expected that the use of pulverised material had some affect on the premature rutting and fatigue cracking of the pavement, possibly from poor compaction and poor drainage characteristics [12].

Lab results were obtained from testing RAP material by MTO test methods. The material was sampled from multiple MTO projects around Ontario. The samples were collected from 20 different projects, at various locations on the job, and at various stages of construction. Some were taken immediately after pulverising and some after grading and compacting but always before paving. A total of 58 samples were taken. These samples were tested for gradation, percent of asphalt coated particles (%ACP), Proctor density, permeability, and California bearing ratio (CBR) [12]. All test values are summarized in Table 2.7.1.

Table 2.7.1: Variation in RAP sample properties from various FDR projects in Ontario (Senior et al.) [12]

Test/Value		# of samples	avg. value	max value	min value
% coarse agg. (retained 4.75mm)	before compaction	50	46.8	76.2	24.5
	after compaction	43	49.8	83	22.7
% Fines (Passing 75µm sieve)	before compaction	56	4.9	10	0.1
	after compaction	42	5.1	10	0.3
Coeff. Of Permeability (cm/s)		57	1.47E-02	2.30E-01	4.11E-05
% Asphalt Coated Particles		52	58.9	89.2	0.5
Proctor density @ 5% mc* (gm/cm^3)	Max Wet Density	14	2.086	2.349	1.19
	Max Dry Density	14	1.986	2.235	1.82
California Bearing Ratio (%)	A	36	21.9	61.1	3.4
	B	12	21	47.8	8.6

*Note: mc used as short form for moisture content.

Gradation requirements were generally satisfied. Only 5 samples had problems, 4 with fines content too large and 1 with too few particles greater than 4.75mm. The majority of the samples were found to have greater than the maximum allowable asphalt coated particles. Only 29% of the tested samples had less than 50 %ACP. This is unexpected as FDR requires a depth of granular base milling equivalent to the depth of old asphalt milled [12].

Compaction testing on 14 samples resulted in an optimum moisture content of 5%. Compacted samples at this moisture content were found to have a density of 2 t/m^3 compared to typical values of 2.2 t/m^3 on natural aggregates [12].

Fifty-Seven of the samples were tested for permeability. Most of these samples met the requirements for a free draining granular base material at greater than 10^{-4} cm/s with an average permeability coefficient of 1.47×10^{-2} cm/s. However, lower values are more likely to cause premature pavement failure, and more than half of the samples were close to the minimum requirement with permeability less than 5.94×10^{-3} cm/s [12].

CBR test results were the farthest from expected with values ranging from 3.4% to 61.1%. A good granular base material is expected to have a CBR value between 90-125%, which means these samples were not nearly as strong as they should have been. Compacted samples were soaked for 48 hours and allowed to drain for 2 hours (Test A) or 5 days (Test B) prior to testing. There was not a significant difference in results for the two draining periods [12].

Neither CBR nor permeability tests are a requirement on MTO projects, however these are known tests whose results indicate a well performing granular base. Most of the permeability results from these samples were approaching undesirable permeability values, and the CBR values all fell very short of desirable [12].

The report by Senior et al. [12] shows that compaction effort is not being used effectively in Ontario. Both the Asphalt Academy [6] and Wirtgen [4] agree that target density should be, on average, 100% of maximum dry density based on modified Proctor compaction. However, MTO's OPSS 501[13] requires a target density of 100% of the maximum dry density as per Test Method LS-706, which uses standard Proctor compaction. Standard Proctor compaction has a lower compaction effort than modified AASHTO resulting in lower densities that could be responsible for the performance.

Fu[7] compacted 100mm ITS specimens using the Marshall compaction method at 3 different compaction efforts (35, 50, and 75 blows per face), and records bulk specific gravity (BSG), soaked indirect tensile strength, and FFAC. As shown in the test results summarized in Table 2.7.2, when the compaction effort is increased, the average density (BSG) of the compacted sample slightly increases as well. As the average density increases by very small amounts, the soaked indirect tensile strength increases significantly. Fu goes on to argue that increasing density means that the aggregates are being packed more tightly together, meaning that bitumen droplets are being squeezed between them. If a bitumen droplet has a finite volume, then when it is squeezed tighter, the surface area of that droplet creates a greater area of contact with the adjacent aggregates, providing an increase in strength, as illustrated in Figure 2.7.1 [7].

Table 2.7.2: Effects of compaction effort on density, soaked strength, and FFAC[7]

PAP	75 blows/face			50 blows/face			35 blows/face		
	BSG[1]	ITS (kPa)	FFAC	BSG[1]	ITS (kPa)	FFAC	BSG[1]	ITS (kPa)	FFAC
33-A	2.12	170	12.7%	2.07	131	11.5%	2.03	89	9.2%
33-B	2.11	209	21.2%	2.06	131	17.0%	1.98	92	14.4%
33-C	2.15	95	4.2%	2.14	89	4.0%	2.10	75	4.0%
88-A	2.06	187	5.5%	1.99	143	5.0%	1.95	103	5.0%
88-B	2.11	236	12.2%	2.09	176	9.9%	2.03	121	8.6%
88-C	2.11	128	4.4%	2.07	110	3.8%	2.02	81	3.1%

[1]BSG = bulk specific gravity

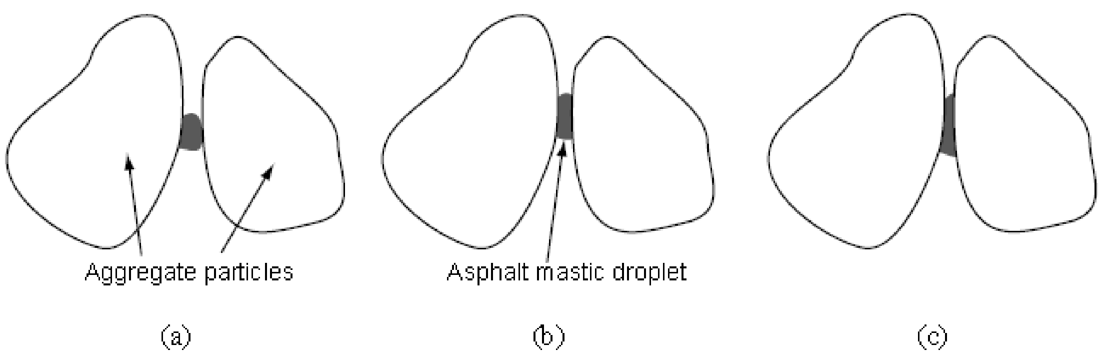


Figure 2.7.1: Illustration of increasing density with compaction effort. Compaction effort increases from a) to c)[7]

2.7.1 Gyratory Compaction

Khan et al.[14] completed a study on different laboratory compaction methods for HMA to determine which method best resembles the techniques used in the field. The experimental study involved taking non-compacted HMA samples from construction sites and compacting them by laboratory techniques, and then comparing their properties to those of field cores taken from the same projects after compaction. The laboratory compaction methods that were compared include automatic Marshall hammer, manual Marshall hammer, California kneading compactor, gyratory compactor at 1.25 degree angle, and gyratory compactor at 6 degree angle. The compacted specimens were compared for bulk density, air voids, resilient modulus, static creep, and stability.

Khan et al. [14] found that the gyratory compaction method with a 1.25 degree angle of gyration best resemble the field compaction methods based on engineering properties. They also found that the automatic Marshall impact hammer creates specimens which least resemble the field compaction, most likely due to the absence of a kneading effect. The results of this study are shown in Figure 2.7.2 and Table 2.7.3.

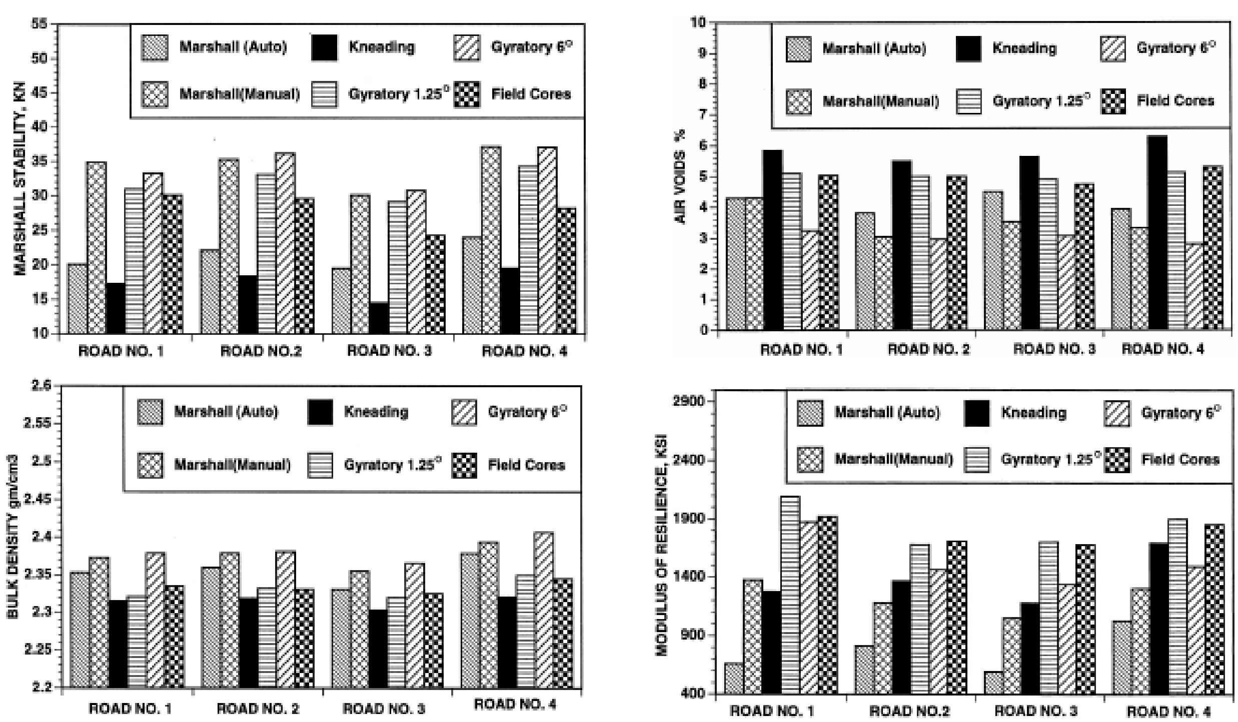


Figure 2.7.2: Comparison of Marshall stability, air voids, bulk density, and resilient modulus for specimens compacted using different compaction methods[14]

Table 2.7.3: Summary of creep test results for various compaction methods[14]

Road No.	Compaction strain	Elastic strain	Viscoelastic strain	Permanent strain	Total
1	Marshall (auto)	6.72E-03	2.98E-03	8.51E-03	1.82E-02
	Marshall (manual)	8.68E-03	3.66E-03	8.17E-03	2.05E-02
	Kneading	9.36E-03	4.26E-03	6.72E-03	2.03E-02
	Gyratory 1.25°	1.03E-02	3.64E-03	4.91E-03	1.89E-02
	Gyratory 6°	4.67E-03	4.08E-03	3.03E-03	1.18E-02
	Field cores	9.33E-03	2.42E-03	7.02E-02	1.87E-02
2	Marshall (auto)	1.13E-02	4.54E-03	1.01E-02	2.59E-02
	Marshall (manual)	1.03E-02	3.15E-03	1.14E-02	2.48E-02
	Kneading	8.59E-03	3.15E-03	6.97E-03	1.87E-02
	Gyratory 1.25°	8.85E-03	3.37E-03	4.35E-03	1.65E-02
	Gyratory 6°	1.03E-02	3.35E-03	7.17E-03	2.08E-02
	Field cores	8.41E-03	3.23E-03	5.43E-03	1.71E-02
3	Marshall (auto)	7.51E-03	4.36E-03	1.31E-02	2.50E-02
	Marshall (manual)	9.45E-03	4.12E-03	9.93E-03	2.35E-02
	Kneading	5.97E-03	2.9E-03	7.87E-03	1.67E-02
	Gyratory 1.25°	8.06E-03	4.03E-03	4.29E-03	1.64E-02
	Gyratory 6°	8.60E-03	3.27E-03	8.57E-03	1.84E-02
	Field cores	7.97E-03	5.91E-04	5.97E-03	1.78E-02
4	Marshall (auto)	9.60E-03	5.40E-03	6.13E-03	2.11E-02
	Marshall (manual)	9.57E-03	6.27E-03	5.72E-03	2.16E-02
	Kneading	8.34E-03	3.66E-03	6.90E-03	1.89E-02
	Gyratory 1.25°	6.51E-03	2.58E-03	4.00E-03	1.31E-02
	Gyratory 6°	1.30E-02	2.73E-03	3.94E-03	1.97E-02
	Field cores	1.32E-02	3.00E-03	4.76E-03	2.09E-02

Kuna et al.[15] completed an experimental study on foamed bitumen mix design with the use of the gyratory compactor. They identified the number of gyrations required to compact a foamed bitumen mix to the modified Proctor compacted density using an angle of gyration of 1.25 degrees and compaction pressure of 600 kPa. Their study involves mixing and compacting foamed bitumen specimens using modified Proctor compaction to determine a target density. The same mixes are then compacted using gyratory compaction with a high number of gyrations, and the running height calculation of the compactor is used to determine when the target density is achieved. The specimens are mixed at the optimum mixing moisture content, and at varying foamed bitumen contents. The study was also completed for different aggregate/RAP blends, including 100% virgin aggregate (VA), 50% RAP and 75% RAP.

Kuna et al.[15] found that the number of gyrations for compaction is independent of the foamed bitumen content since for a given RAP/aggregate blend mixed at the same mixing moisture content, the target densities are achieved within a small range of gyrations. For 100% VA this range is between 120-160 gyrations, so a design number of gyrations (N_{design}) of 140 is chosen. They also found that this range changes based on the amount of RAP in the blend. The 50% RAP blends with varying foamed bitumen content fall close to 110 gyrations, and the 75% RAP blends fall between 80-120 gyrations or an average N_{design} of 100 gyrations.

2.8 Curing

A foam BSM material requires the moisture to evaporate from the compacted mix in order for it to gain strength. The process of moisture leaving the specimen is called curing. In some cases where active fillers are used, it may be desirable to slow down the rate of water evaporation. When using Portland cement for example, water is necessary for its hydration process and strength gain. In a foamed asphalt specimen, curing improves the contact surface between bitumen droplets and aggregates as seen in Figure 2.8.1.

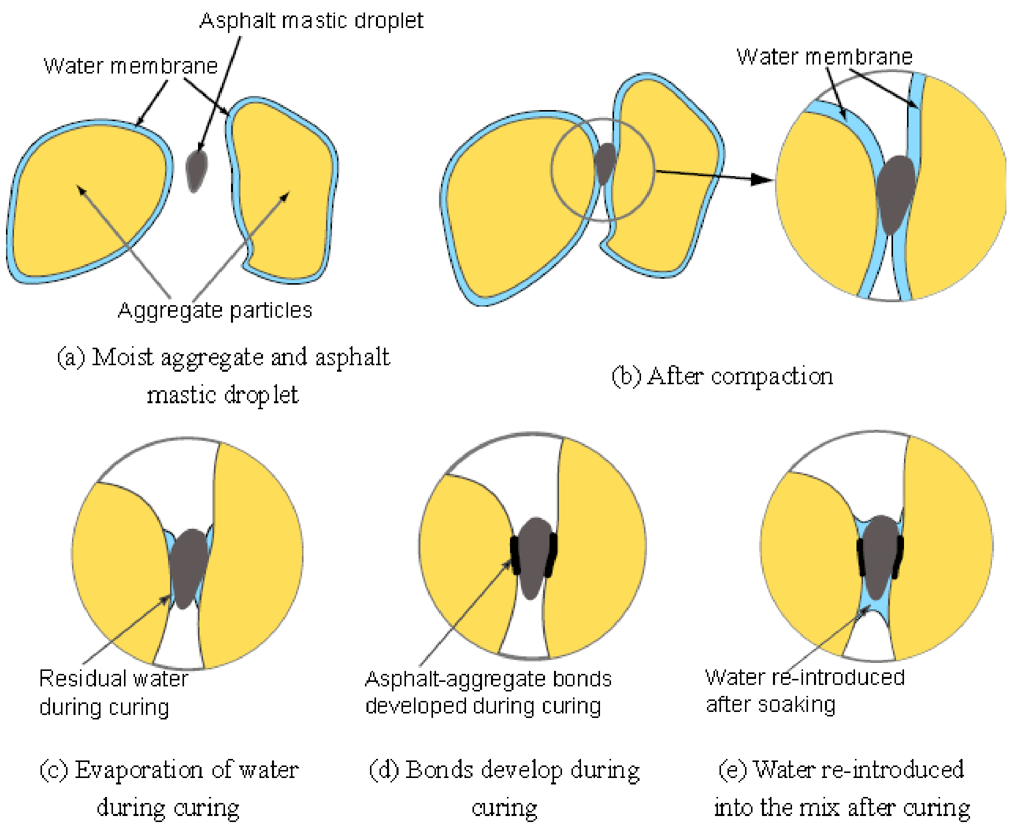


Figure 2.8.1: Curing mechanism of FASB [7]

Many different laboratory curing processes have been used over the years to simulate field conditions. The curing temperature, humidity, and time are especially important for curing. A detailed review of curing techniques is given by Fu [7].

A curing method at 60 degrees Celsius for 72 hours is used by Emery [1]. This procedure is also used by many other researchers. Fu[7] shows that specimens cured at 50 degrees Celsius result in bitumen flow which does not occur in specimens cured at 40 degrees Celsius. He also finds that a temperature of 50 degrees Celsius is rarely surpassed in stabilized base layers in California. Fu [7] also notes that a number of researchers use ambient temperatures to cure their specimens. However, ambient temperatures have fluctuating properties. For example, large fluctuations in humidity can be observed at 20 degrees Celsius compared to 40 degrees Celsius [7].

Sealing specimens in plastic bags is another idea which Fu explored. This method is used to simulate equilibrium moisture content in the field. Research determines that this method leaves samples more wet than is typically experienced in California, however it may be used as a more conservative estimate since the presence of moisture reduces strength during testing [7].

Both the Asphalt Academy and Wirtgen's suggested curing methods are in agreement with Fu's findings. They both state that dry 100mm specimens should be cured at 40 degrees Celsius for 72 hours. They also both recommend that 150mm specimens should be cured at 30 degrees Celsius, unsealed for 20 hours and then sealed in a bag of twice its volume for 48 hours at 40 degrees Celsius [4]. Asphalt Academy also states that the plastic bag should be replaced with a dry one every 24 hours of sealed curing [6].

2.9 Indirect Tensile Strength and Tensile Strength Ratio

A BSM is a non-continuously bound material and therefore water is capable of permeating through it at a higher rate than in HMA (bound) but a lower rate than in granular aggregate (unbound). A BSM is also typically used as a granular base course meaning that it is susceptible to fluctuations in ground water level. For these reasons, moisture susceptibility is a very important factor to consider in BSM design [7].

The indirect tensile strength (ITS) test is commonly used for optimizing bitumen content in HMA. It has been adopted for the same purpose in the foamed asphalt mix design procedure. The Asphalt Academy and Wirtgen recommend using it in both level 1 and level 2 mix design on cylindrical specimens of 100mm and 150mm diameter, respectively. Tensile strength ratio (TSR) is the ratio of soaked ITS to unsoaked ITS, and it is used to determine whether a mix has adequate moisture resistance [11]. Wirtgen[4] and Asphalt Academy [6] have recommended minimum ITS values for soaked and unsoaked states depending on design traffic levels. They also note a minimum TSR at 50%, and up to 75% for moisture resistance depending on climate. MTO's requirements on a foamed asphalt stabilized material are stated in OPSS 331, which requires a minimum dry ITS of 225kPa, minimum soaked ITS of 100kPa, and minimum TSR of 50% [16].

The tensile strength of a specimen is generated by the mineral filler and the asphalt mastic. In a BSM, the mineral filler phase is typically composed of recycled or pulverised asphalt pavement. This material is expected to attribute tensile strength by means of weak chemical bonding due to impurities at particle contact points, osmotic suction from residual water in the pore spaces, and adhesion from residual binder. All of these tensile strengths are greatly reduced by voids filled with water. Fu [7] finds that the mineral filler phase loses approximately 81% of its tensile strength when the specimen is soaked, whereas the asphalt mastic phase only loses 45% of its tensile strength. This means that the asphalt mastic is more water resistant than mineral filler, and that it provides most of the tensile strength when soaked.

Chen et al.[17] investigated a foamed asphalt highway project in Texas which was built on a subgrade known to have high moisture content. The highway failed in a number of sections, which was believed to be due to a lack of moisture resistance in the foamed asphalt stabilised base layer. A series of soaked ITS tests and a 10 day capillary ITS test were performed on samples taken from severely distressed area, intermediate, and good sections. The samples from the severely distressed area did not remain intact. ITS tests resulted in a 20% and 17% retained strength after soaking in the good and intermediate sections respectively, and fell well short of the 75% requirement by Wirtgen. Using the 10 day capillary conditioning instead of soaking, the good and intermediate sections resulted in 78% and 68% retained strength respectively. The capillary rise was observed to travel only 25mm up the good core but it travels all the way to the top of the intermediate core. This was found to be a more representative test of the field conditions and the sample from the good section proved to retain adequate strength by this method [17].

Further investigations into the aggregate properties of the failed and intact sections were performed in order to determine the cause of moisture susceptibility in failed sections. The plasticity index of fine fractions of aggregate in the distressed section was actually lower than the intact section, however still higher than the maximum of 10 suggested by the Asphalt Academy and Wirtgen. The dry density of the FASB in the failed section is slightly lower than that of intact sections and ultimately the excessive fines content further reduced moisture resistance [17].

Chiu and Lewis[18] conducted performance tests on several foamed asphalt mixes including a comparison with 100% virgin aggregates and 80/20 RAP/GA blend. They found that the mixes including 80% RAP had a higher retained strength in the soaked state than those of 100% new aggregates, all with the addition of a 1.5% lesser foamed bitumen content. They also observed that the addition of fly ash as a mineral filler results in lower tensile strengths than mixes with blast furnace slag [18]. Results are summarized in Table 2.9.1 and Table 2.9.2.

Table 2.9.1: Components of four BSM mixes investigated Chiu et al. [18]

	Components			
Mix Type	Coarse Aggr., %	Sand, %	RAP, %	Additives
Mix 1-1	50	47	0	1.5% cement+ 1.5% fly ash
Mix 1-2	50	47	0	1.5% cement+ 1.5% BFS[a]
Mix 2-1	0	17	80	1.5% cement+ 1.5% fly ash
Mix 2-2	0	17	80	1.5% cement+ 1.5% BFS[a]

[a]Blast furnace slag.

Table 2.9.2: ITS results, and mix properties from four FA mixes [18]

	100 % New Aggregate Mix		80 % RAP Mix	
Property	Mix 1-1	Mix 1-2	Mix 2-1	Mix 2-2
Compaction moisture/fluid content (%)	6.7	6.7	6.2	6.2
Maximum density	2.185	2.185	2.11	2.11
Selected foamed asphalt content (SFAC) (%)	3.5	3.5	2.0	2.0
Bulk Relative Density @ peak strength	1.973	1.976	1.98	1.99
Indirect Tensile Strength (dry) kPa @ SFAC	626	906	512	800
Indirect Tensile Strength (soaked) kPa @ SFAC	391	728	438	703
Retained ITS (%)	62.5	80.4	85.6	87.9

Many researchers have investigated the improved moisture resistance of a BSM-Foam mix when adding hydraulic fillers such as cement or lime. Hodgkinson and Visser[19] investigated the effects of active filler on a bitumen stabilized material using

RAP as aggregate. Their study was conducted on both bitumen emulsion and foamed bitumen specimens, where bitumen content varies between 1% and 4%. Different types of fillers including both active and inert fillers were considered in the BSM mixes at 1.5% concentration by mass [19]. ITS tests were performed at dry and wet states. The dry and wet tensile strengths and the percent retained ITSs were calculated. The results for foamed bitumen tests were summarized in Table 2.9.3. The results portray that the addition of active filler coincides with a significant increase in the wet tensile strength for both bitumen emulsion and foam however only a significant increase in dry tensile strength with foamed bitumen when compared with no filler [19]. Inert fillers are found to have similar results for both types of binder with dry tensile strengths around 300 kPa, which is a similar strength to those with no filler. Foamed bitumen mixes tend to retain tensile strength poorly between the dry and wet ITS tests. With inactive fillers, the strength retention is typically around 40% and falling as low as 20%. The cementitious binder played a far greater role in strength gain in the wet state, however TSRs still almost always remain below 80% [19]. There is not a significant difference between types of cementitious binder.

Table 2.9.3: ITS test results of BSM-foam mixes incorporating active and inert fillers (Hodgkinson et al.) [19]

Bitumen	1.5% of OPC	1.5% OPC & FA 82/18	1.5% OPC & FA 75/25	1.5% OPC and limestone 90/10	1.5% slagment	1.5% rock flour	neat material
Foam 1%	620	533	396	507	260	370	313
	473	342	271	338	105	78	80
	76	62	68	61	40	21.1	25.6
Foam 2%	647	725	572	646	333	240	303
	478	388	353	429	170	114	100
	74	54	62	66	51	47.5	32.9
Foam 3%	829	760	670	794	494	328	270
	543	511	305	501	250	177	104
	64	61	46	63	51	54	38.5
Foam 4%	755	794	704	749	433	286	137
	509	668	523	623	254	170	77
	67	84	74	83	59	59.4	56.2

Dry ITS, Wet ITS, and TSR in top, middle, and bottom lines respectively

2.10 Resilient Modulus

Resilient modulus is an indicator of the stiffness of a material under cyclic loading. Luo [5] performed a number of resilient modulus tests on unstabilized aggregate/RAP blends with different RAP contents and compared the resilient moduli with natural aggregate. He found that increasing the RAP content tends to reduce resilient modulus initially (25% RAP), but with further increasing RAP content, the resilient modulus increases and peaks at 75% RAP. This is especially apparent at high bulk stress levels. He noted that increasing density typically increases the resilient modulus of a sample but increasing the percentage of RAP tends to reduce the density of a sample for the same compaction effort. Therefore the improved resilient modulus must be due to other factors such as the RAP's higher resistance to degradation under repeated loading, compared to natural aggregates. This idea is quantified by the significantly better performance of RAP at high bulk stresses, where the degradation of natural aggregate samples typically increases [5].

Kim et al.[20] evaluated the behaviour of RAP/aggregate blends in terms of resilient modulus and permanent deformation. They used gyratory compaction to compare specimens since it most closely represents compaction in the field. The specimens were prepared at different RAP contents and then tested at two moisture contents, 65% and 100% of OMC. Similar to Luo[5], Kim et al.[20] found that increasing RAP content leads to an increase in stiffness. They also noticed that the specimens tested at lower moisture content have a 10-116% increase in stiffness over those at 100% OMC. The specimens compacted at 65% OMC have higher cohesion, likely due to increased soil suction [20].

Fu[7] studied the stiffness of foamed asphalt mix by the means of resilient modulus tests. He observed that most researchers use an indirect tension (IDT) setup for resilient modulus determination in a foamed asphalt treated specimen. Typically the results of IDT M_r are significantly higher than field M_r values. This is concluded from back-calculation of falling weight deflectometer (FWD) tests resulting in an M_r of 500-3000 MPa compared to the over 5000 MPa M_r from IDT. Triaxial M_r and flexural beam tests provide more realistic values [7].

Fu[7] performed triaxial M_r tests on mixes with various pulverised asphalt pavement (PAP) parent materials in soaked and unsoaked, stabilized and unstabilized states. He observed that adding foamed asphalt stabilization had little effect on M_r in the unsoaked state since only one of six PAP materials resulted in an improved unsoaked-M_r after stabilizing. Stiffness improvements were more dominant in the soaked state with improvements in four of six PAP materials. The PAPs with coarser surface texture tend to have less improved stiffness over smoother PAPs when stabilized. It is concluded that the stiffness of the mix changes from being dominated by aggregate properties in the untreated mix to binder properties in the treated mix.

Nataatmadja[21] performed a study on the resilient modulus of foamed bitumen mixes at various stages after compaction with various compaction methods. Resilient modulus is determined immediately after compaction (before curing), after curing by different curing methods, and after curing and then soaking. The different curing methods which are compared include 28 days air curing, 3 days curing at 40 degrees Celsius, and 3 days curing at 60 degrees Celsius. The results show that the 60 degree curing results in significantly higher resilient modulus, likely due to increased bitumen aging. The 40℃ curing method after three days produces M_r similar to the air cured specimen after 7 days [21]. These results can be seen in Figure 2.10.1.

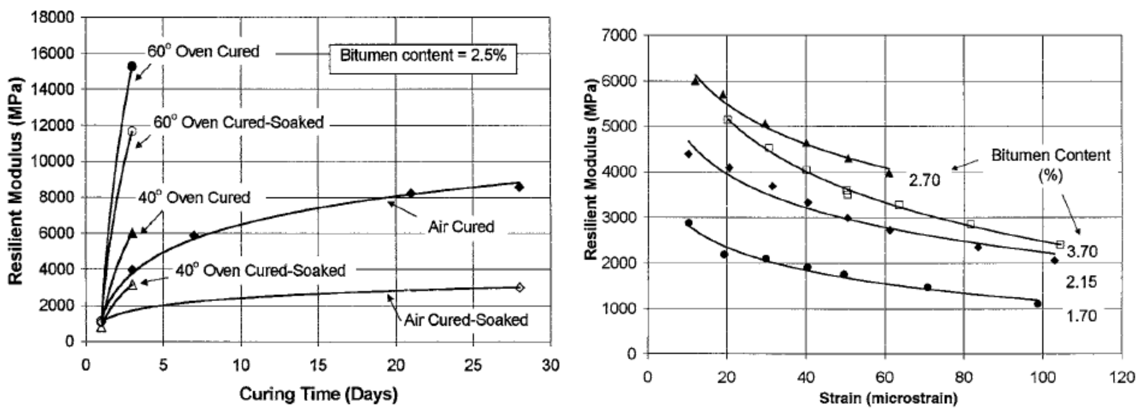


Figure 2.10.1: Effect of curing on M_r (left) and applied strain on M_r (right)[21]

Another finding by Nataatmadja[21] is the effect of strain level on resilient modulus. Figure 2.10.1 shows the resilient modulus at various strain levels and bitumen contents for specimens compacted by 50 blows (Marshall compaction method), and cured at 60℃. Regardless of the bitumen content, the resilient modulus decreases as strain level is increased. As the bitumen content increases, M_r increases and reaches the maximum at 2.7% bitumen content. Further increase of bitumen content reduces M_r.

Khweir[22] studied the affect of fines content on the stiffness of a foam stabilized material. He compared the stiffness of BSMs at varying fines content and active filler content. As shown in Figure 2.10.2, increased fines content results in higher stiffness of foam stabilized materials, especially in the presence of hydraulically activated fillers. He also found that a reduction in compaction level by as little as 7% can result in a 50% loss of stiffness, as shown in Figure 2.10.3.

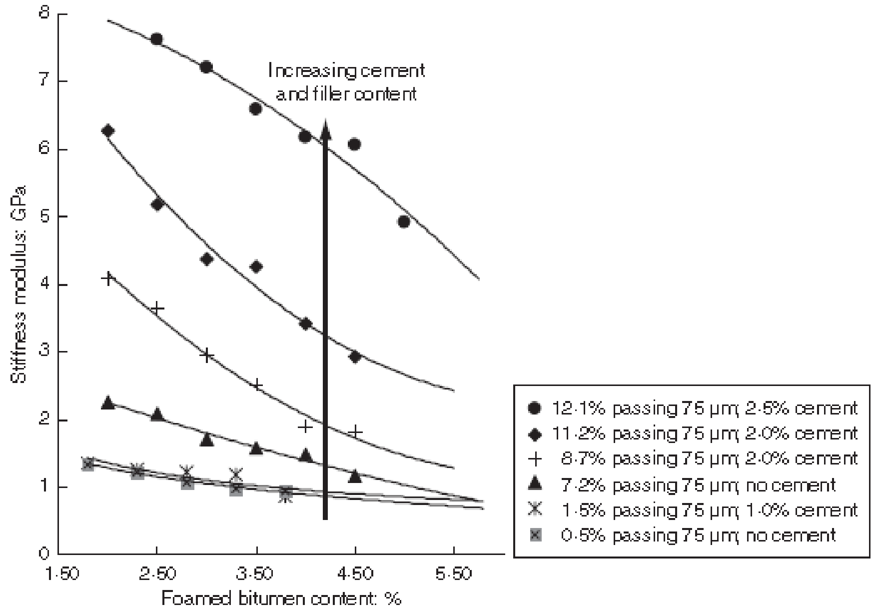


Figure 2.10.2: Influence of fines content on BSM stiffness [22]

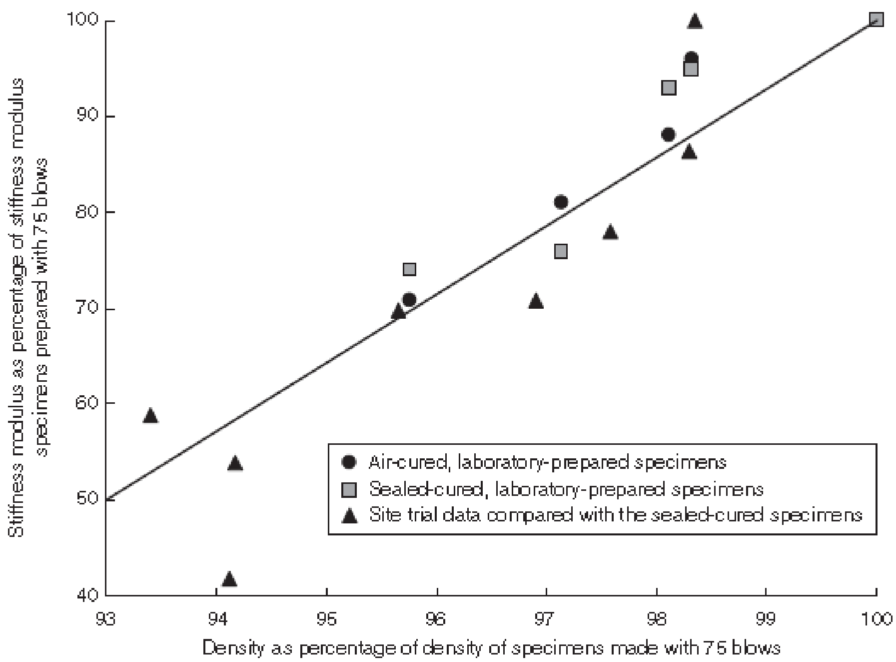


Figure 2.10.3: Stiffness with respect to target density of a BSM [22]

2.11 Permanent Deformation

Kim et al.[20], Luo[5], and Stolle et al.[23] investigated the permanent deformation of RAP/aggregate blends at different RAP contents by performing cyclic triaxial testing. In Kim's experiments, the materials are compacted at the optimum moisture content using gyratory compaction. Two gyratory specimens are molded together by scratching the surfaces between them and compacting with the vibratory hammer. As the RAP content increases, the permanent deformation increases as well, exhibiting at least two times the deformation of 100% aggregate [20].

Stolle et al.[23] and Luo [5] also reveal that increasing RAP content leads to increasing permanent deformation of RAP/aggregate blends. However, their results show minimal increase in permanent deformation up to 50% RAP. Figure 2.11.1 clearly demonstrates this trend from Luo's findings.

As shown in Figure 2.11.2, different stress levels are utilized in Luo's testing, which illustrates that stress level significantly influences permanent deformation. As a specimen is loaded under a given stress level, the rate of permanent deformation starts off large and reduces with the increase of loading cycles. If the stress level is subsequently increased, the rate of permanent deformation would jump up again and continue to gradually reduce as the specimen is loaded[5].

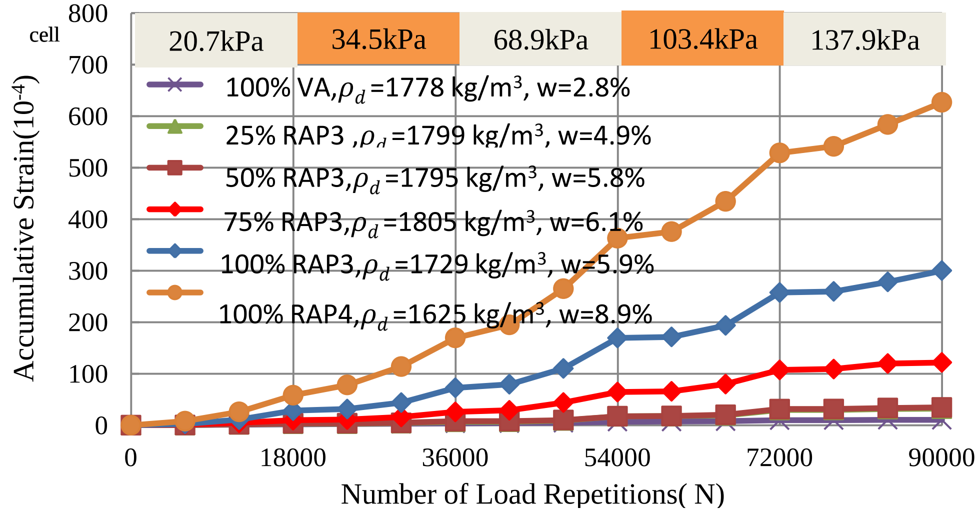


Figure 2.11.1: Effect of RAP content on permanent deformation[5]

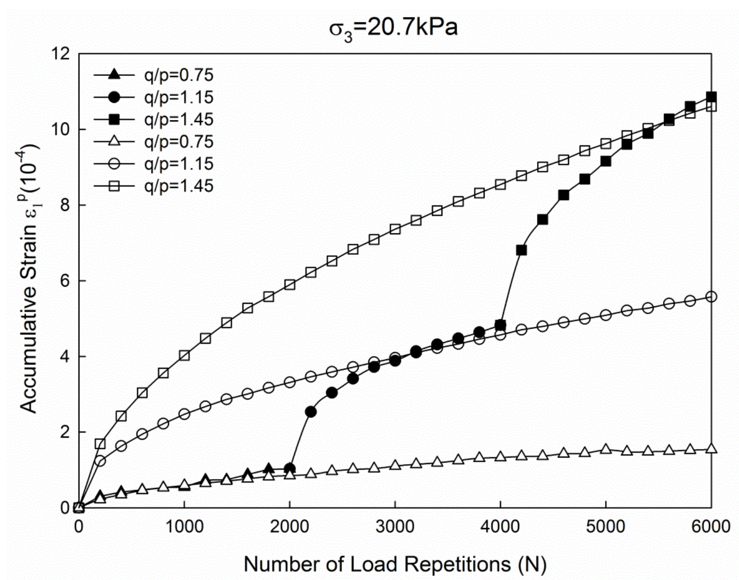


Figure 2.11.2: Effect of stress level on permanent deformation[5]

Stolle et al.[23] found that blends containing RAP have reduced shear strength compared to natural aggregates. This is due to the deformability of the RAP particles. They concluded that an increase in compaction effort on some blends will provide strength properties similar to those of natural aggregates [23].

Jitareekul and Thom[24] performed an experimental study on the permanent deformation characteristics of foamed asphalt stabilized base materials incorporating varying RAP contents. Their studies also tested the effects of different binders and active filler. Initial testing was carried out in a triaxial setup using loads representative of those expected in the center of a typical pavement structure. Similar to unbound RAP materials, increasing the load amplitude results in greater permanent deformation. Increased confining pressure reduces permanent deformation, even at low cyclic loads. Lastly, using stiffer binders results in a mix with less permanent deformation.

Jitareekul and Thom[24] also performed repeated load experiments at the Nottingham Pavement Test Facility which uses a moving wheel to apply the load to a constructed pavement. Four different sections of 80mm thick FASB are constructed on top of a 450mm subbase of crushed limestone. Of the 4 different sections, the FASB layers are constructed of 75% RAP with PG 50/70 foamed bitumen, 75% RAP with PG70/100, 50% RAP with PG 70/100, and 50% RAP with PG 70/100 and 1.5% cement. The results confirm that increasing RAP content results in increased permanent deformation, increased penetration grade of bitumen slightly increases permanent deformation, and addition of cement as active filler greatly improves resistance to permanent deformation. The results are illustrated in Figure 2.11.3.

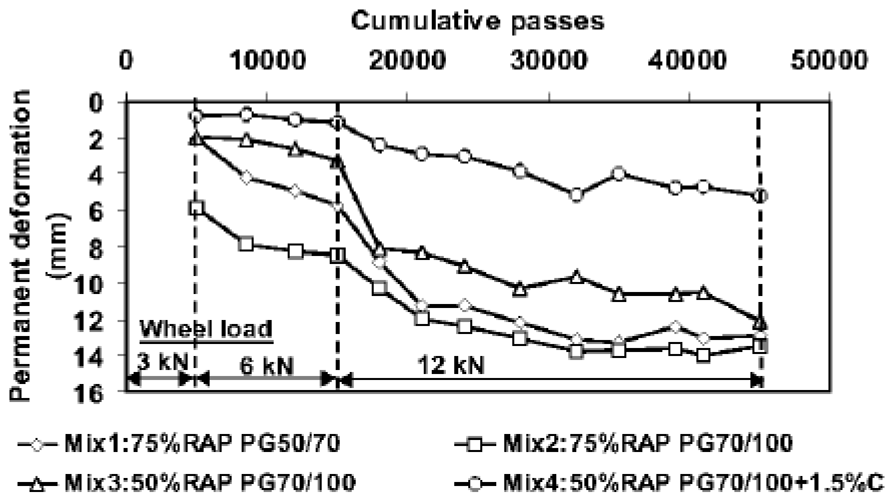


Figure 2.11.3: Permanent deformation results on various BSM-Foam mixes constructed at Nottingham [24]

3 Experimental Studies

3.1 Test Program

This section is a brief overview of the laboratory testing program of this research including the materials acquired for the research, the BSM-foam mix design, an overview of laboratory testing including preliminary material testing and final mix performance testing, as well as the number and order of tests. A summary of all completed tests is provided.

3.1.1 Materials

(a) Natural Aggregates

Two types of clean natural aggregates were acquired for this research. The first material type labelled “A1” was crushed quarried bedrock from Milton, Ontario. It was classified as granular B type II with 19mm maximum aggregate size. The second material type labelled “F1” was limestone screening, which was used for adjustment of fines content in the aggregates.

(b) RAP

One type of RAP was used for this project, labelled “B4”. It was fine fractionated RAP with 100% of material passing the 13.2mm sieve. This research is part of a larger project which includes RAPs B1-B3, which are not considered in the scope of this thesis.

(c) Blends

Of the natural aggregates and RAP materials acquired, blends were created to analyze the effect of mix constituents on the performance of final test specimens. The

RAP/aggregate blends were created to analyze the effects of different RAP contents. For comparison reasons, 100% Milton Granular B type II, and 100% fine fractionated RAP were also tested. All material blends other than 100% RAP were adjusted to densest gradation with 7-8% fines content using the fine material F1. The different blends are summarized in Table 3.1.1 along with shortened names.

Table 3.1.1: Material blends

Material Blends	
Shortform	description
C1	19mm granular B type II
C2	50/50 GB/RAP*
C3	25/75 GB/RAP
C4	75/25 GB/RAP
C5	Fine fractionated RAP
C6	50% GB (3% fines)/50% RAP

*Note: 50/50 GB/RAP stands for blend with 50% aggregate and 50% RAP

<u>(d) Asphalt binder</u>

The asphalt binder used for foaming was PGAC 58-28, which is a typical grade used for HMA in southern Ontario.

3.1.2 Mix Design

Preliminary mix design was carried out according to the Asphalt Academy[6], Wirtgen[4] and Emery[1]. The ITS tests, asphalt binder (AC) content, and quantity of specimens for each material blend used in the mix design, are shown in Table 3.1.2. The complete description of all mix design preparation, performance tests, and analysis of test results are found in Chapter 4.

Table 3.1.2 : Estimated optimum AC% and mix design specimen targets

			ITS test	
Material	Estimated opt. AC%	target AC %	dry	soaked
C1	4%	3.5	xx	xx
C1		4	xx	xx
C1		4.5	xx	xx
C1		5	xx	xx
C2	2.80%	2.3	xx	xx
C2		2.5	xx	xx
C2		2.8	xx	xx
C2		3.3	xx	xx
C3	2.10%	1.6	xx	xx
C3		2.1	xx	xx
C3		2.6	xx	xx
C3		3.1	xx	xx
C4	3.40%	2.9	xx	xx
C4		3.4	xx	xx
C4		3.9	xx	xx
C4		4.4	xx	xx
C5	1.50%	1	xx	xx
C5		1.5	xx	xx
C5		2	xx	xx
C5		2.5	xx	xx

Notes:

1. Four specimens for each AC content and 16 specimens per material;
2. AC content estimated using Emery equation (Not moisture susceptible):

$$AC\ content\ (\%) = 1.5 * \left(\frac{\%RAP}{100}\right) + 4.0 * \left(\frac{\%Granular}{100}\right)$$

3.1.3 Testing Program

(a) Preliminary aggregate testing and mix design testing

During preliminary aggregate testing, sieve analyses, Atterberg limits tests for plasticity index of fine fraction aggregates, and Proctor compaction (standard and modified) were performed on the acquired materials. The corresponding results were used to optimize aggregate gradation, mixing water content, compaction water content, and maximum dry density for each material combination. It should be noted that plasticity index results may indicate the need for active fillers, which are not considered in this program therefore plasticity index was checked for acceptability only.

An asphalt foamer was designed and fabricated to make asphalt foam (see Section 3.3.1). The PGAC 58-28 binder was tested for foaming properties in the binder expansion test to determine the optimum foaming water content by testing expansion ratio and half life of the foam at different water contents.

The optimum asphalt binder content was determined by completing ITS tests for the dry and soaked states at different foamed asphalt contents whose targets are specified in Table 3.1.2. ASTM test standard D6931-12 was followed. IDT resilient modulus tests were also conducted during this stage due to the non-destructive manner of the testing.

Each batch of foam stabilized material at a given binder content had enough material to create three compacted specimens. Of those three specimens, one was used to determine the soaked indirect tensile strength (ITS). Using the ITS of the soaked specimen, values could be estimated for dry and soaked ITS failure load for the remaining two specimens from the same mix. This was because the applied load in the indirect tension resilient modulus (IDT M_r) test is less than 20% of the specimen's ITS failure load, and the same specimens could be used for both IDT M_r and ITS testing. The finished specimens and tests completed in the mix design are displayed in Table 3.1.3. Notice that mix designs were only completed for materials C1, C2, and C5.

Table 3.1.3: Actual mix design specimens and tests

	AC content, %		ITS test		IDT M_r test	
Material	target	actual	dry	soaked	dry	soaked
C1	4%	1.96	x	xx	x	x
C1		2.2	x	xx	x	x
C1		2.58	x	xx	x	x
C1		3.2	x	xx	x	x
C1		4.95	x	xx	x	x
C1		5.5	x	xx	x	x
C1		5.7	x	xx	x	x
C1		6.13	x	xx	x	x
C2	2.80%	1.69	x	xx	x	x
C2		2.03	x	xx	x	x
C2		2.45	x	xx	x	x
C2		2.75	x	xx	x	x
C2		3.26	x	xx	x	x
C2		4.11	x	xx	x	x
C5	1.50%	1.56	x	xx	x	x
C5		2.1	x	xx	x	x
C5		2.36	x	xx	x	x
C5		3.1	x	xx	x	x
C5		3.38	x	xx	x	x
C5		4.8	x	xx	x	x
C5		4.9	x	xx	x	x

Note: Subject to AA and Wirtgen

(b) Performance tests of non-stabilized materials

In this research, non-stabilized material specimens were tested for comparison with foam stabilized materials. All tests were completed in the triaxial setup. The specimen sizes were approximately 300mm tall and 150mm in diameter. All tests were carried out using the vibratory hammer with a density target of 100% modified proctor compaction.

The completed non-stabilized material tests were the triaxial resilient modulus test (TriM_r) and repeated load permanent deformation test (RLPD). The triaxial resilient modulus test was completed following AASHTO T307-99 to determine resilient modulus of aggregate blends prior to foaming. RLPD tests were completed to determine the creep resistance of non-stabilized aggregates under repeated loads. The test setup follows that of AASHTO T307-99, however there is no standard for loading. The number of tests performed is displayed in Table 3.1.4.

Table 3.1.4: Non-stabilized material performance tests

Material	AC %	RLPD	TriM_r
C1	0	xx	xx
C2	0	xx	xx
C5	0	xx	xx

(c) Performance tests of foam stabilized materials

Foam stabilized materials are non-continuously bound. Therefore they do not perform exactly like either unbound granular material or continuously bound HMA. Performance testing may therefore be completed using either IDT tests designed for HMA, or triaxial tests designed for aggregate material. Triaxial tests were primarily used for performance testing in this study, however IDT M_r tests were also completed during the mix design stage. Approximately 300mm tall by 150mm diameter specimens were tested in the triaxial setup. All specimens were fabricated at optimum AC content and 100% modified Proctor compaction using the gyratory compactor.

IDT resilient modulus tests were conducted on specimens during the mix design stage. These tests were carried out at multiple foamed asphalt contents on 100mm tall by

150mm diameter specimens, compacted at 100% modified Proctor compaction using the gyratory compactor. The data was used for performance evaluation rather than optimum bitumen content determination.

The completed stabilized material tests were IDT M_r tests, triaxial resilient modulus tests, RLPD tests, and unconfined compression tests. The IDT resilient modulus tests were completed following ASTM D7369 − 11 designed for bituminous mixtures. The triaxial resilient modulus test used the same apparatus and loading sequence as non-stabilized material tests following AASHTO T307-99 for comparison with non-stabilized materials. However specimens were fabricated by the BSM-foam design and compacted using gyratory compaction to the maximum density of 100% modified Proctor compaction. Specimens for RLPD and unconfined compression tests were also fabricated following BSM-foam design and tested following the procedure for aggregates.

Five mixes were created for each material blend (C1, C2, and C5), aimed at the optimum AC (bitumen) content shown in Table 3.1.5. The mixes tended to vary from the optimum AC content by +/- 0.5% by mass of the total specimen. Specimens are reused for different tests when physical deterioration is not present. The testing matrix for FASB mixes is displayed in Table 3.1.5.

Table 3.1.5: Performance tests on optimum FASB mixes

Material	Opt AC %	RLPD dry	RLPD wet	TriM_r dry	TriM_r wet	UC dry	UC wet
C1	2	xxxx	x	xxxx	x	xxxx	x
C2	3	xxxx	x	xxxx	x	xxxx	x
C5	3.2	xxxx	x	xxxx	x	xxxx	x

Additional tests were proposed however time constraints did not allow for their completion. These additional comparison tests are described in Appendix 8.1

3.2 Physical Properties of Aggregates and RAP/Aggregate Blends

The aggregates used in this study were Milton granular B type II with 19mm nominal maximum particle size and fine fractionated RAP with 100% passing 13.2mm. Limestone screenings were also used as a blending material in order to optimize the fine gradations. The RAP and granular B are pictured in Figure 3.2.1.

Figure 3.2.1: Dry aggregates, a) 19mm Milton granular B type II b) 13.2mm fine fractionated RAP

Milton Granular B Type II was light grey in colour. By visual inspection the aggregates had high angularity and less than 5% of particles appeared to be flat or elongated. The RAP particles appeared to be slightly more rounded than the Granular B and with even less flat and elongated particles. The residual binder had a dull black colour and did not feel sticky to touch, therefore the binder was considered to be inactive.

Preliminary aggregate tests were completed on all materials in order to optimize foamed bitumen performance. The tests included washed and unwashed sieve analysis to determine gradations, plastic and liquid limit tests to determine the plasticity index of the fine fraction of aggregates, and Proctor compaction for target densities and optimum moisture content.

3.2.1 Individual Aggregates

3.2.1.1 Unwashed Sieve Analysis

The purpose of the unwashed sieve analysis was to determine the gradation of particles in the parent aggregates. The unwashed sieve analysis was most applicable to foamed asphalt mix design because fines content plays an important role in foamed asphalt dispersion in the mix. Only the loose fines aid in dispersion, therefore it is more accurate for the fine content to be determined without deliberately loosening fine particles, which naturally adhere on large particles. The unwashed sieve analysis was performed following MTO LS 602[25].

Results and Discussions

The unwashed sieve analysis was completed twice for all three parent materials which were to be blended and used in the creation of a foamed asphalt mix. The results of the two tests were averaged and compared to the recommended maximum and minimum requirements for parent materials as described by Wirtgen. The Asphalt Academy and Wirtgen have very similar recommendations as described in the literature review, however Wirtgen recommends slightly higher fines content and therefore it was chosen.

The 19 mm crushed rock gradation curve had entirely fit between the recommendations described by Wirtgen as seen in Figure 3.2.2. The fine RAP shown in Figure 3.2.3 had too little material passing the #40 sieve (below 0.425mm). This is a critical zone in ensuring density of the parent material and ultimately the final product. The gradation of the limestone screenings shown in Figure 3.2.4 closely bordered the maximum recommended amount of finer particles therefore this material can be blended with the RAP in order to create an optimum gradation.

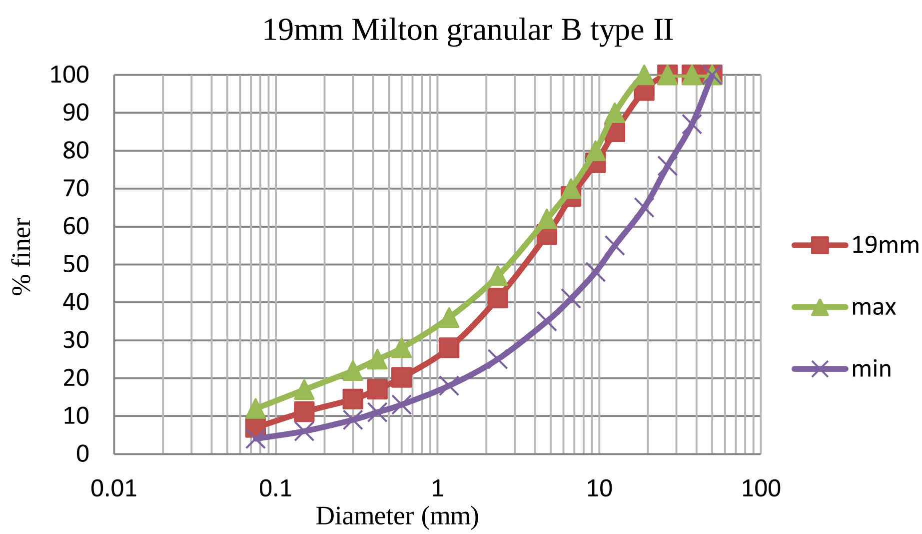


Figure 3.2.2: Unwashed gradation of 19mm Milton granular B type II

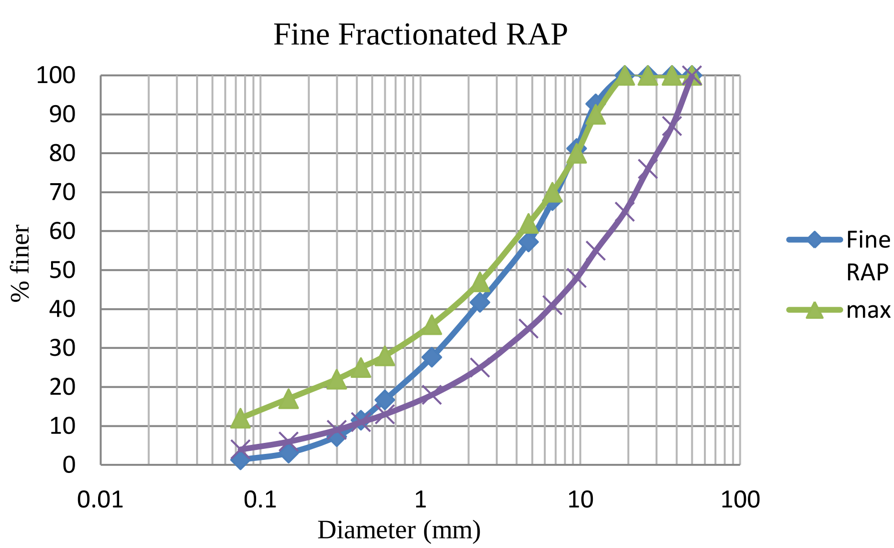


Figure 3.2.3: Unwashed gradation of fine fractionated RAP

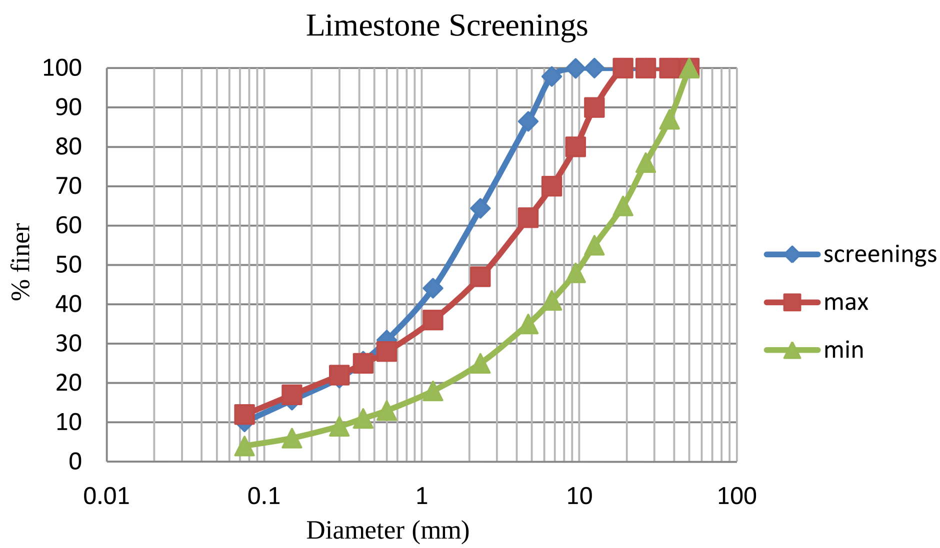


Figure 3.2.4: Unwashed gradation of limestone screenings

3.2.1.2 Washed Sieve Analysis

The purpose of the washed sieve analysis was to determine the actual amount of fine particles in the mix by freeing those that may cohere on large particles in the unwashed sieve analysis. The washed sieve analysis was performed following ASTM C117-13[26] by washing with plain water (Procedure A).

<u>Results and Discussions</u>

Washed sieve analyses were completed for the Milton Granular B Type II and the limestone screenings. ASTM C117 states not to save the wash water and to count all the lost mass from the original sample to be fine content. The wash water was saved and dried in this study, however there was a small spill of fines when transferring the wash water from the bucket into dishes for drying, while performing test #2 on Milton Granular B.

When the fine material was saved, the total dried sample after washing, including fines, had 0.4% lower mass on average, than the total dried sample before washing. Test #2 of Milton Granular B had a slightly higher overall loss percentage of 0.51%. The calculations were repeated assuming that all lost material during the washing process was fine material. Therefore, the material sample after washing had a mass equal to the material before washing, and the fine material portion was equal to the total sample mass less the saved coarse aggregate fraction mass. This removed the error of lost material due to the spill.

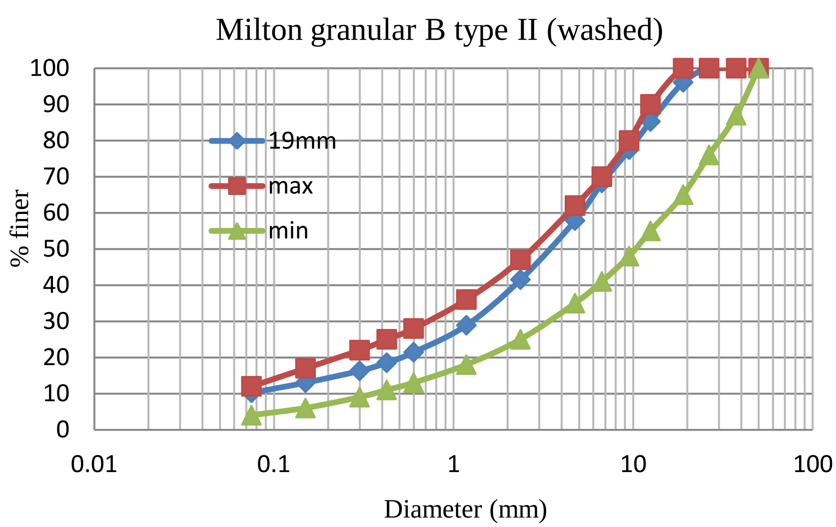


Figure 3.2.5: Milton granular B type II washed sieve analysis

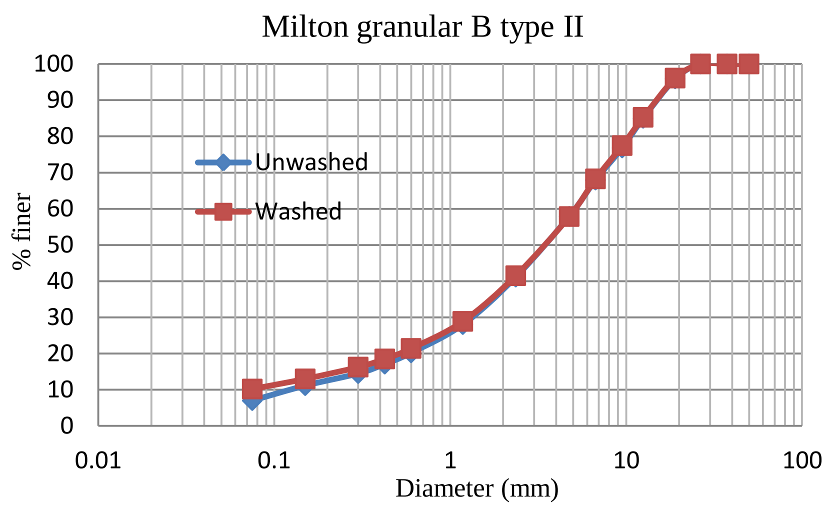


Figure 3.2.6: Milton granular B type II washed vs. unwashed sieve analyses

From Figure 3.2.5 it can be seen that the Milton Granular B type II sample in the washed analysis still fell within the boundaries for an ideal mix set out by Wirtgen. Figure 3.2.6 shows that the washed sample was mostly the same as the unwashed sample. The only differences were the fractions passing the 0.15mm and 0.075mm sieves, which had greater values in the washed sieve analysis. The percentage passing the 0.15mm sieve increased from 11.14% to 13.01% on average. The percentage passing the 0.075mm sieve increased from 7.01% to 10.21% on average.

The same observations were made when examining the screening samples for washed and unwashed sieve analysis. The grading had fit the Wirtgen recommendations and the fine fraction passing 0.15mm and 0.075mm sieves were significantly increased in the washed analysis, as seen in Figure 3.2.7 and Figure 3.2.8, respectively. The percentage passing the 0.15mm sieve increased from 15.65% to 17.98% on average. The percentage passing the 0.075mm sieve increased from 10.13% to 13.94% on average.

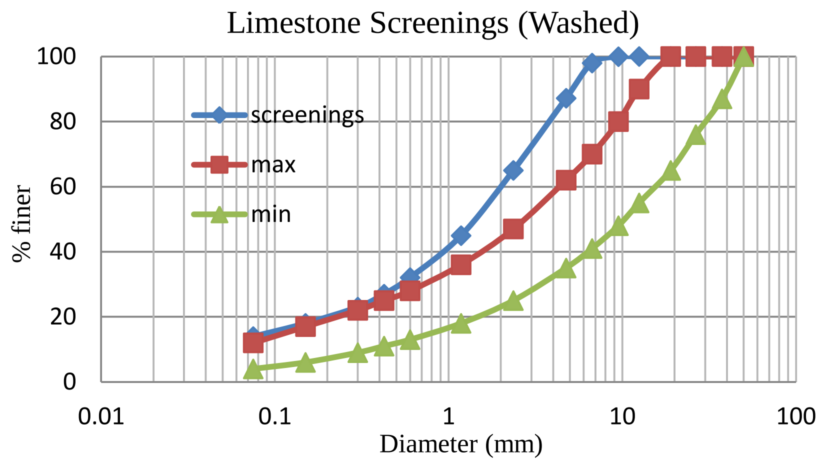


Figure 3.2.7: Limestone screenings washed sieve analysis

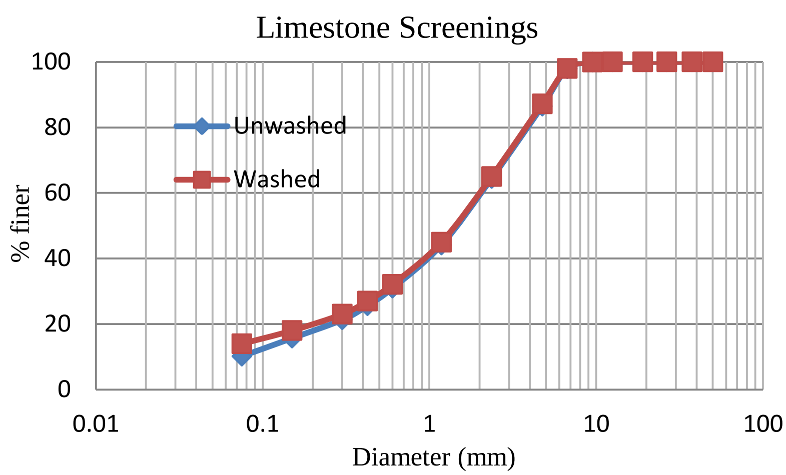


Figure 3.2.8: Limestone screenings washed vs. unwashed sieve analysis

3.2.1.3 Plasticity Index of fines

The plasticity index was determined by performing the Atterberg limits testing. The purpose of determining the plasticity index was to estimate the plasticity of the fines in the parent material. The Atterberg limit tests were performed following ASTM D4318-10[27].

Results and Discussions

The plastic limit was not obtainable for any of the materials. All materials began to crack at a thickness greater than 3mm when trying to form a cylinder. When increasing moisture content the materials were unable to be rolled and maintain shape, therefore the fines were considered to be non-plastic. This was likely due to the fines being those of crushed limestone rock flour rather than shale or clay.

3.2.2 Aggregate Blends

In this study, aggregates were blended to the gradations as recommended by Asphalt Academy[6] and Wirtgen[4], which are compared in Table 3.2.1. The target fine contents (passing #200) were aimed for 7% as recommended by Emery [1]. The purpose of this gradation was to create the densest possible aggregate structure with sufficient fines for adequate dispersion of foamed asphalt.

Table 3.2.1: Recommended gradation by AA[6] and Wirtgen[4]

Sieve size (mm)	% Passing							
	Asphalt Academy				Wirtgen			
	Ideal		Less suitable		Ideal			
	min	max	min	max	min	max	Less suitable	Typical RAP
50	100				100		100	100
37.5	87	100			87	100	100	85
26.5	77	100	100	100	76	100	100	72
19	66	99	99	100	65	100	100	60
13.2	67	87	87	100	55	90	100	50
9.5	49	74	74	100	48	80	100	42
6.7	40	62	62	100	41	70	100	35
4.75	35	56	56	95	35	62	88	28
2.36	25	42	42	78	25	47	68	18
1.18	18	33	33	65	18	36	53	11
0.6	14	28	28	54	13	28	42	7
0.425	12	26	26	50	11	25	38	5
0.3	10	24	24	43	9	22	34	4
0.15	7	17	17	30	6	17	27	2
0.075	4	10	10	20	4	12	20	1

Aggregate blends were created in RAP/GB blends as outlined in the experimental design (Ch 3.1). They included 100% GB, 25% RAP/75% GB, 50% RAP/50% GB, 75% RAP/25% GB, and 100% RAP. Another blend was created as 50% RAP/50% GB with reduced fines content. These blends had adjusted fines contents using fine material extracted from the aggregate screenings. The 50/50 blend was adjusted using fine fraction aggregates passing #40 sieve. The 75/25, and 25/75 blends were adjusted with fines passing #200 sieve. No adjustments were carried out on the 100% RAP and 100% GB materials. The gradations of each blend can be seen in Figure 3.2.9 through Figure 3.2.11. Gradations of 100% granular B and 100% RAP can be seen in the unwashed sieve analysis of section 3.2.1.1. A summary of all of the gradations are shown in Table 3.2.2.

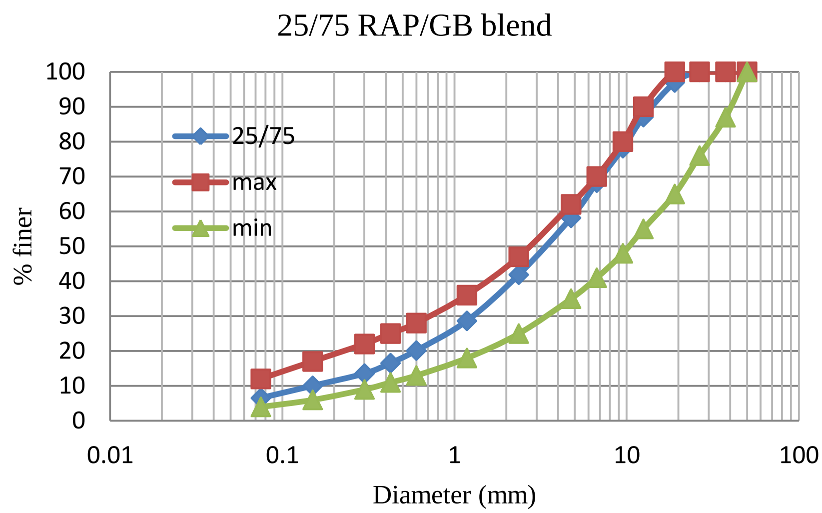


Figure 3.2.9: 25% RAP / 75% GB blend

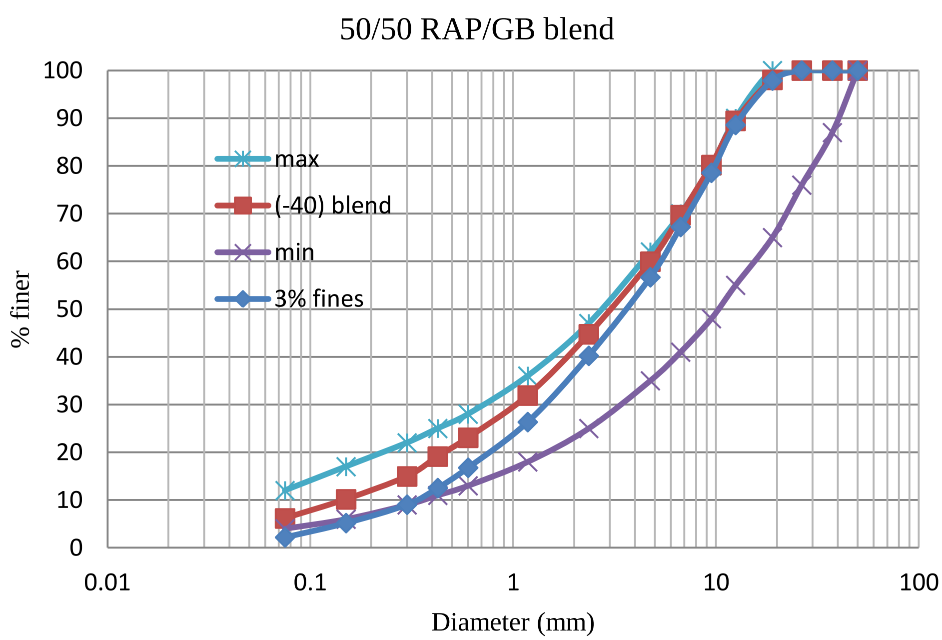


Figure 3.2.10: 50% RAP / 50% GB blends with adjusted fines content (7% entitled (-40) blend, and 3% entitled 3% fines)

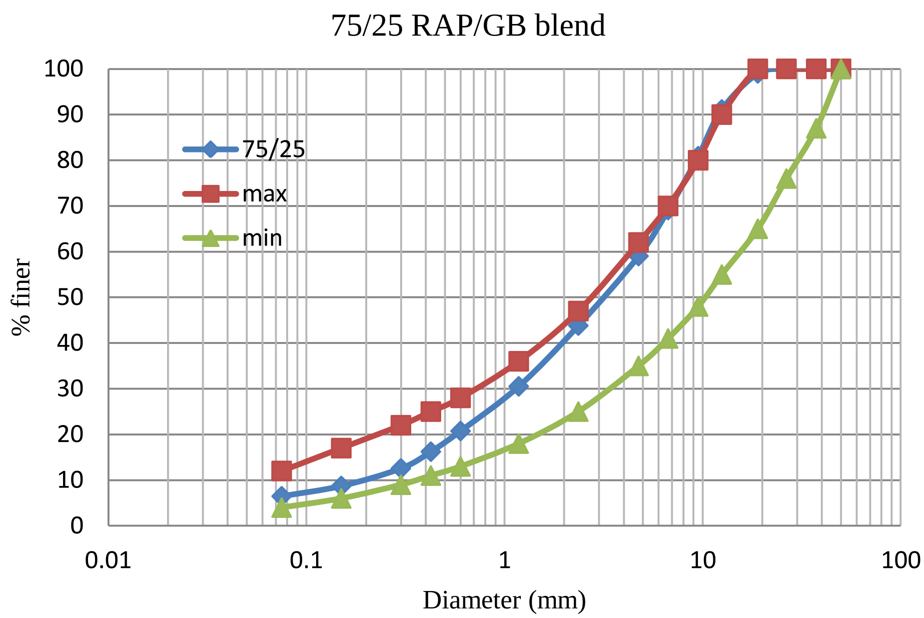


Figure 3.2.11: 75% RAP / 25% GB blend

Table 3.2.2: Summary of all gradations; individual aggregates and blends

sieve	% finer						Wirtgen % finer		
(mm)	avg RAP	avg 19mm	25RAP/75GB	50/50 (7%)	50/50 (3%)	75/25	optimum	max	min
50	100	100	100	100	100	100	100	100	100
37.5	100	100	100	100	100	100	100	100	87
26.5	100	100	100	100	100	100	100	100	76
19	100	95.88	96.94	98.05	97.90	99.01	100	100	65
12.5	92.68	84.98	87.04	89.46	88.60	91.11	82.83	90	55
9.5	81.23	76.79	78.12	80.20	78.58	80.88	73.20	80	48
6.7	67.85	67.93	68.23	69.71	67.23	69.10	62.56	70	41
4.75	57.23	57.89	58.14	59.96	56.69	59.03	53.59	62	35
2.36	41.73	41.13	41.86	44.74	40.23	43.83	39.12	47	25
1.18	27.65	28.01	28.63	31.91	26.35	30.52	28.64	36	18
0.6	16.70	20.19	20.12	23.06	16.78	20.74	21.12	28	13
0.425	11.47	17.09	16.52	19.13	12.53	16.23	18.09	25	11
0.3	7.26	14.47	13.53	14.96	9.05	12.56	15.46	22	9
0.15	3.04	11.14	10.01	10.15	5.19	8.71	11.32	17	6
0.075	1.30	7.01	6.52	6.16	2.20	6.47	8.29	12	4

Note: In 50/50 (7%) and 50/50 (3%) the number in parentheses indicates fines content

3.2.3 Proctor Compaction

Modified Proctor compaction tests were carried out on all individual aggregates and blends. Standard Proctor compaction was also carried out as a comparison on the 50/50 blend with 7% fine content. These tests were performed in order to determine the optimum moisture content (OMC) for the BSM design process, and also for the purpose of finding the desired target density for gyratory compaction. The Proctor compaction tests were performed following ASTM D1557[28].

Results and Discussions

A comparison of Proctor compaction tests were carried out on RAP/GB blends with increasing RAP content, shown in Figure 3.2.12. The results are generally consistent with findings in the literature review. As the RAP content increased in the aggregate blends, the maximum dry density decreased [5]. In terms of the optimum water content, all samples peaked at approximately 6% by mass with slight variations. Full tabulated results are presented in Table 3.2.3.

Table 3.2.3: Results from Proctor compaction tests

Blend (RAP/GB)	Max. dry density (kg/m3)	OMC (%)
100% GB	2310	5.2
25/75	2292	6.1
50/50 (7% mod.)	2254	5.8
50/50 (7% std.)	2155	6.9
50/50 (3% mod.)	2174	6.2
75/25	2141	6.1
100% RAP	2047	6.7

Three Proctor tests were completed on the 50/50 blends shown in Figure 3.2.13. The control mix was the 50/50 blend with fines content adjusted to 7%, and compacted by modified proctor. The figure shows that standard Proctor compaction results in a maximum dry density that is 100kg/m^3 lower than the same mix with modified Proctor compaction. This result was caused by a reduction in compaction energy from modified to standard effort.

A 50/50 blend with reduced fines content (3% fines) was also tested at modified Proctor compaction. Its density was reduced by approximately 75kg/m^3. This reduction was attributed to its deviation from optimum gradation, since the appropriate particles were not available to fill voids.

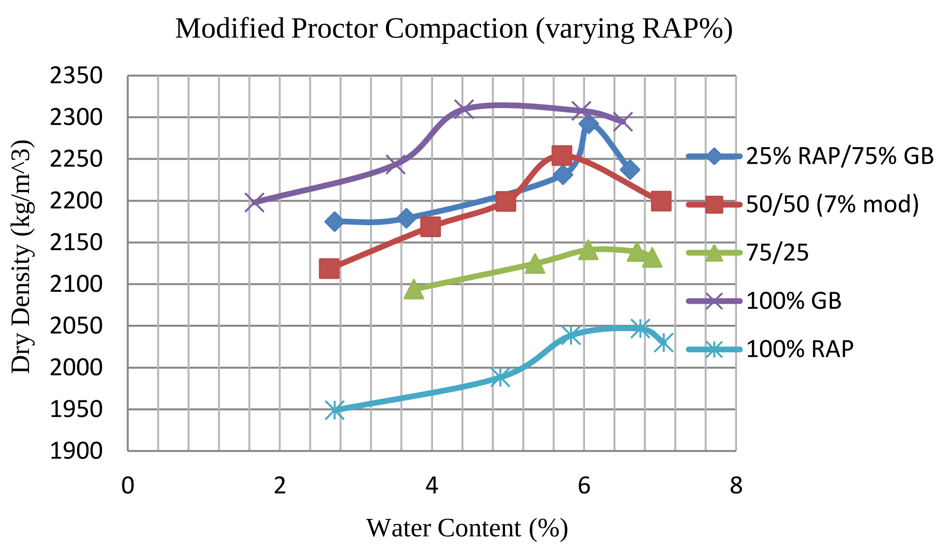


Figure 3.2.12: Proctor compaction test results: influence of RAP content

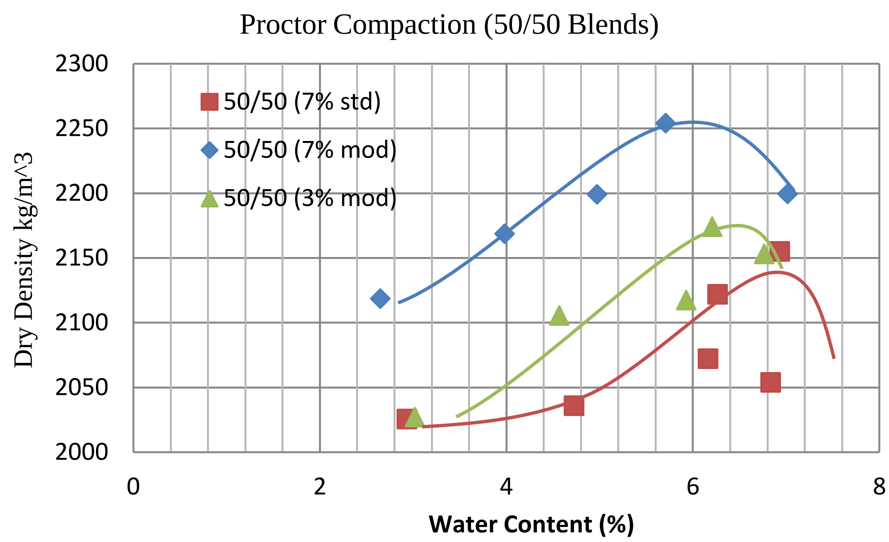


Figure 3.2.13: Proctor compaction test results: influence of fines content and compaction effort on 50/50 blends

3.3 Bitumen Foaming

3.3.1 The Foamer

The foaming devise, shown in Figure 3.3.1, was constructed to complete this research. It applied air pressure to force an asphalt cement quantity into an expansion chamber. A metered valve was used to control the flow rate and hence the quantity of asphalt cement. Air pressure was also added to the metered water line, after the water metering, to create a fine mist. The mist readily reacts with the hot bitumen in the expansion chamber to create a foamed bitumen product of adequate quality, as shown in the foaming results (Section 3.3.3).

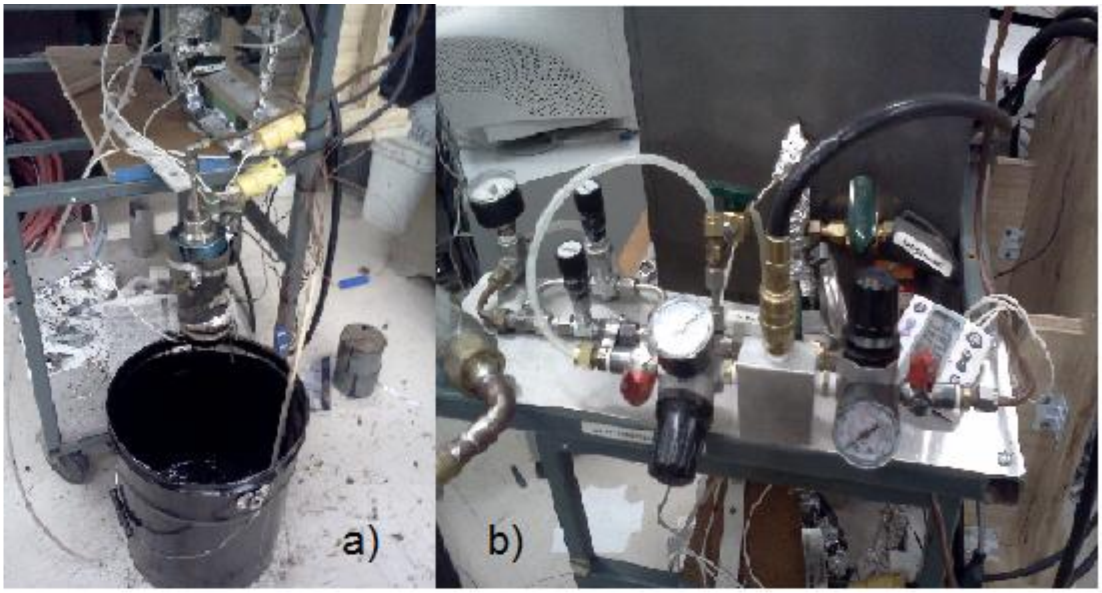


Figure 3.3.1: The foamer; a) expansion chamber; b) water, compressed air input, and controls.

Bitumen must be 160 degrees Celsius prior to adding it to the foamer's bitumen tank. The tank was heated to maintain the bitumen temperature or only slightly increase it. The lines between the tank and the expansion chamber were wrapped in a heater coil to maintain 160-170 degrees Celsius and prevent heat loss during flow. A schematic design of this foamer used in the research is displayed in Figure 3.3.2.

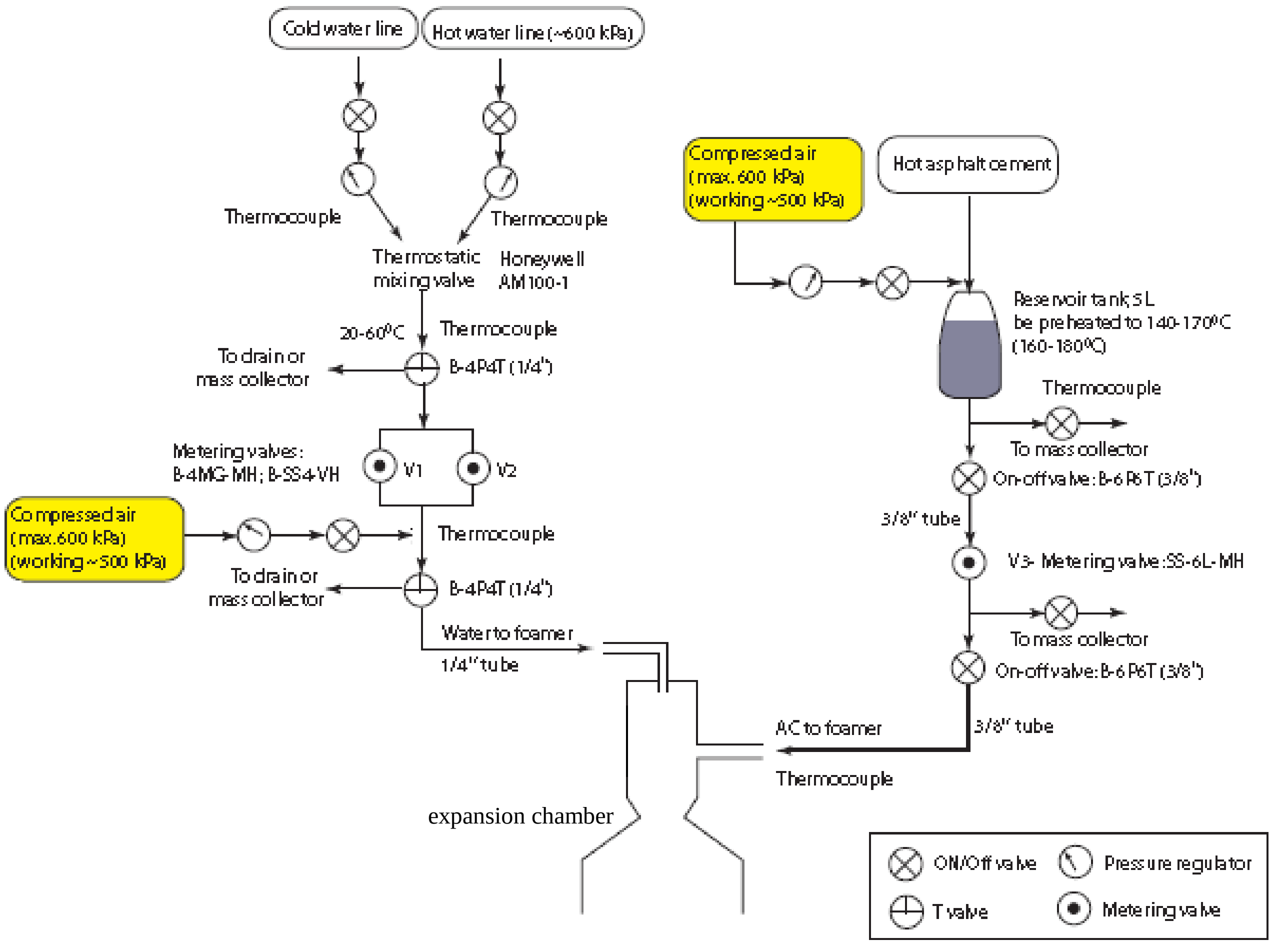


Figure 3.3.2: Schematic design of the foamer for this research

Some flaws of this foamer had been identified, especially with the control of bitumen flow rates. When using the foamer to determine the foaming water content, bitumen flow rates proved to be fairly steady. This is likely because the test was conducted at a significantly higher flow rate when compared to the actual mixing process with small batches. During expansion testing, a flow rate of 80g/sec was used in order to accurately measure the half life and expansion ratio of the material in a 20 Litre bucket. During mixing, the mixer was capable of successfully stirring a 14kg sample without jamming the agitator. The Asphalt Academy recommends mixing to take place over a 20-30 second period[6]. This equates to a foaming rate of approximately 30g/sec for a bitumen content of approximately 4%.

The necessity for a lower flow rate made the metering valve more susceptible to clogging from oxidized asphalt, especially when the molten bitumen was held stagnant in the oven-heated tank. Based on the literature[29], bitumen deposits can become stuck to the sides of the tank from locally overheating, since the material was not constantly stirred. Overheated bitumen can subsequently break off from the sides of the tank and get clogged in the lines.

In Wirtgen's design [30], the bitumen is constantly flowing in a closed loop to better control flow rates and to keep the material mixed and evenly heated throughout. Figure 3.3.3 is a schematic illustration of the Wirtgen WLB 10 laboratory foamer. As shown, there is a closed loop through the heated bitumen tank via a bitumen pump. Upon demand, the on-board compressed air system will actuate the tappet and force the bitumen out of the closed loop and into the foam nozzle for mixing with water, to create foam.

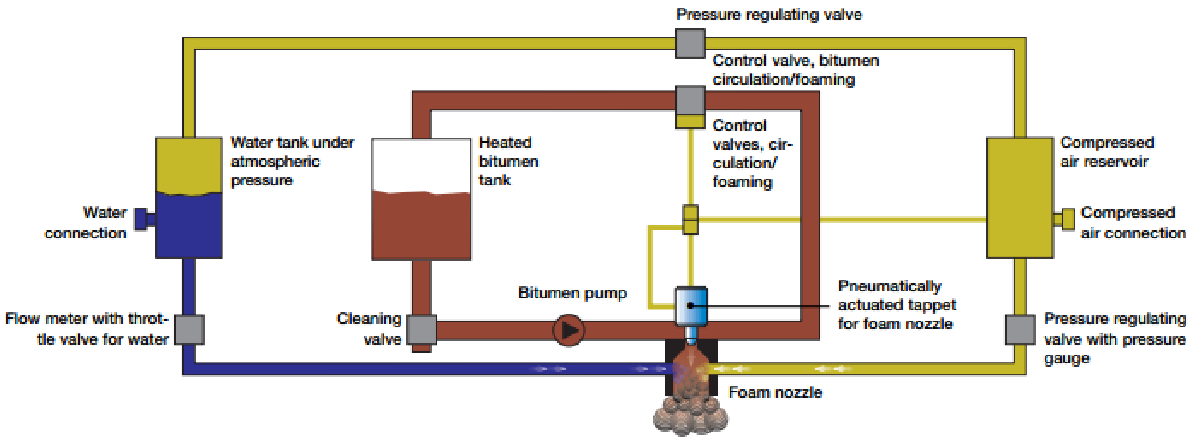


Figure 3.3.3: Schematic illustration of the Wirtgen WLB 10 laboratory scale foamed bitumen plant[30]

3.3.2 Foaming Procedure

The following procedure describes how to determine the foaming water content of a non-polymer modified bitumen, using the above foaming device.

1) Heat the bitumen tank to around 150℃ internal air temperature. Also, set the bitumen lines to 170℃.

2) Bring the PG58-28 binder to a temperature of 165-170℃ in the asphalt kettle.

3) While heating the bitumen in a safe place nearby, attach the water lines and bleed the air out through the bypass line.

4) Unbolt the tank on the foamer and carefully pour the hot bitumen into the preheated tank.

5) Tighten all the bolts on the tank, attach the air line, and ensure that everything is sealed.

6) Attach the air line to the source by the downstream end first, to prevent the line from whipping. Apply the air pressure slowly and listen for leaks.

7) Check the temperature of the bitumen in the tank. It should be between 160-180℃. Once it reaches 160, check the bitumen flow rate and adjust it to the desired flow.

8) Adjust the water flow rate to the desired foaming water content. To check optimum foaming water content, create foam at 1, 2, 3, and 4% water content by mass and compare the half-life (HL) and expansion ratios (ER). Do not reuse the foamed bitumen.

9) Place a preheated 20L bucket with uniform cross section beneath the foaming nozzle. Insert a clean stick used to measure foam height.

10) Open the water, air, and bitumen nozzles in that order, and start a stopwatch to measure foaming time. (5 seconds and 80 g/sec was used for determining foam properties)

11) After foaming, record the foaming time, mass of the foam (bucket, bucket + foam), height of expanded and collapsed foam, and the time required for half of the foam to collapse (HL).

Note: Optimum values recommended by Asphalt Academy [6] are a minimum of 6 seconds and 8 times for HL and ER respectively.

12) When foaming is finished for the day, ensure that all the lines are clean by blowing them out with air into a waste bucket. Also, detach the air and water lines and cool the bitumen lines when not in use.

13) The oven may be left on overnight to maintain heat however it should not be left on without use for days at a time. This will cause oxidation of the bitumen in the tank.

3.3.3 Results and Discussion

In order to determine the optimum foaming water content, two important factors must be considered; the expansion ratio (ER) and half life (HL) of the foam. The optimum foaming water content was determined as the half-way point between water contents for minimum acceptable ER and HL. The minimum acceptable ER is 8 and the minimum acceptable HL is 6 when the aggregate temperature is above 25℃. Figure 3.3.4 presents the results from a series of foaming trials at different water contents. An ER of 8 corresponded to a water content of approximately 2.5% and an HL of 6 seconds corresponded to a water content of 4% therefore the optimum water content was at 3.25%. Tests were performed on binder between 160 and 170℃.

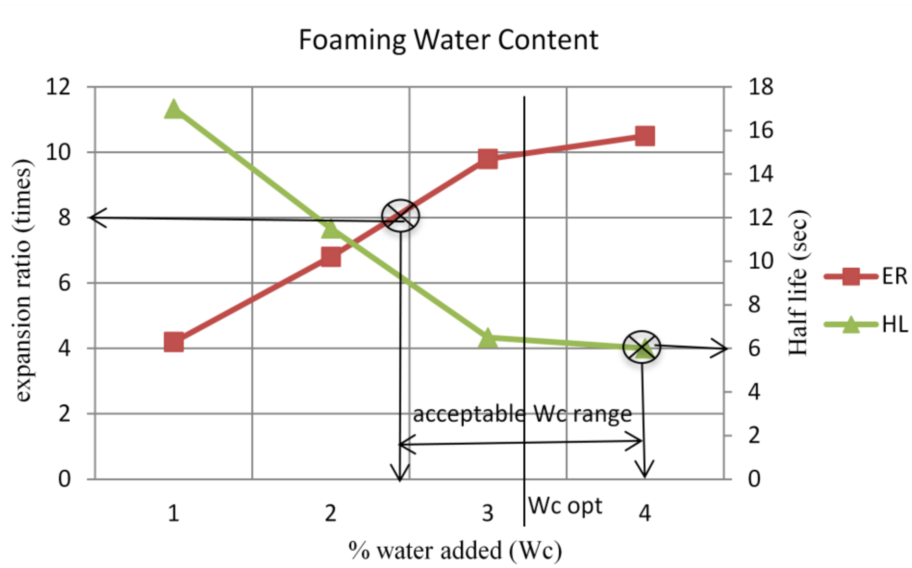


Figure 3.3.4: Determination of optimum foaming water content

The results presented in Figure 3.3.4 show a range of acceptable foaming water contents based on the minimum ER and HL recommendations by Asphalt Academy [6] and Wirtgen[4]. However, Wirtgen[30] claims that bitumen foaming should result in an expansion of nearly 20 times the original volume of bitumen depending on its grade. AME[31] performs asphalt half life and expansion analysis on the same PG58-28 binder grade as used in this research, although from different manufacturers. These tests show upper ends of ER in the 15 to 20 times range whereas this research does not. AME's foamant water content graphs are provided in Figure 8.1.1 and Figure 8.1.2 of the appendix Section 8. AME's test on Yellowline Asphalt Products' PG58-28 binder at 160 ℃ results in an optimum foaming water content of 2.8%, with an HL of 7.8 seconds and ER of 12.6 times. AME also tested McAsphalt Industries' PG58-28 binder at 165 ℃ resulting in an optimum foamant water content of 2.5%, with an HL of 7.2 seconds, and ER of 10.7 times. These two tests provide similar optimum foaming water content with a small difference of 0.3% between the two. The test on the same grade of bitumen using the foamer constructed for this research resulted in an optimum foamant water content of 3.25%, an average difference of 0.6% water by mass compared to tests prepared by AME.

3.4 Indirect Tension Tests

3.4.1 Test Apparatus

The apparatus used for indirect tension tests was a hydraulic testing machine capable of applying controlled force and deformation while simultaneously measuring force and vertical displacement. It was also capable of applying a haversine-shaped load pulse over a range of durations, amplitudes, and rest periods between pulses, and collecting real-time data at 200 scans per second [32].

Following ASTM testing method D6931-12 [33], the gyratory compacted cylindrical specimens were diametrically loaded on loading strips machined to conform to the circumference of 150 mm diameter specimens. The lower loading strip was mounted to a movable box-setup on a rigid member. The upper loading strip was positioned on the specimen during setup prior to contact with the top loading mechanism. It was positioned to load diametrically through the centre of the specimen as shown in Figure 3.4.1.

Figure 3.4.1: Indirect tension test setup with LVDT's for measuring horizontal displacements used in IDT Mr testing

The box-setup, in which the cylindrical specimen was placed, had a detachable upper portion with two sensors attached to it for measuring horizontal displacement of the specimen while allowing it to compress vertically (for resilient modulus). This system was not capable of controlling temperature, therefore all tests were conducted at room temperature.

3.4.2 Specimen Preparation

The 150 mm diameter gyratory compacted specimens ranging in height from 110 mm to 125 mm were subjected to soaked or dry conditions prior to testing. Soaked specimens were prepared by submerging the specimen in a water bath at 25° C for 24 hours prior to testing. For dry conditions the specimens were left in air at room temperature for 24 hours prior to testing. Soaked and dry conditioning was completed after the curing stage. Specimen preparation up to and including the curing stage is detailed in the mix design Section 4.1.

3.4.3 Indirect Tensile Strength

The indirect tensile strength (ITS) test is commonly used for the determination of optimum bitumen content in the early design stages of bitumen stabilised materials. It is used in the BSM design guidelines for the Asphalt Academy[6] and Wirtgen[4]. Mixes are optimised based on their optimum dry strength, wet strength, and tensile strength ratio (TSR). TSR is the ratio of soaked to dry ITSs and it is an indicator of the moisture sensitivity of the mix.

The test was performed by loading a cylindrical specimen by two opposing line loads, applied on the diametric plane. The loads were applied on the curved surfaces through two loading strips which were machined to conform to the radius of curvature of the specimen. The load was increased at a constant rate until failure occured and the maximum load was recorded.

In this research, ITS tests were used as recommended by Wirtgen and the Asphalt Academy in the mix design stage. The procedure was generally carried out according to ASTM D6931-12 and the following steps were taken:

1) Measured the thickness and diameter of the specimen. Thickness was measured as the average of 4 equally spaced points around the perimeter of the specimen. Diameter was measured as the average of two perpendicular diameters.

2) Specimens were bathed in air or water depending on whether a dry or soaked test was being performed. If performing a dry test, the specimen was placed in an air bath for 24 hours. If performing a wet test, the specimen was placed in a water bath for 24 hours.

3) The specimen was removed from the bath and immediately brought into contact with the loading strips in the loading apparatus. The specimen was taken to maximum load within 2 minutes of removal from the bath. In some cases, an M_r test was also carried out on the specimen and therefore specimen failure occurred 20-30 minutes after removal from the bath.

4) The specimen was vertically loaded at a constant rate of vertical deformation. It is recommended to be loaded at a rate of 50 +/- 5 mm/min. A rate of 2.5 +/- 0.5 mm/min was used for safety, and to better represent the gradual accumulation of cracking experienced by a pavement. (An initial sample test indicated that slower loading resulted in lower ITS values).

5) The maximum load was recorded and the indirect tensile strength was calculated using the following equation.

$$ITS = \frac{2000 * P}{\pi * t * D}$$

where,

ITS = Indirect Tensile Strength (kPa)
P = Maximum load (N)
t = Initial thickness of the specimen (mm)
D = Diameter of the specimen (mm)

3.4.4 Indirect Tension Resilient Modulus

Resilient modulus can be used in the evaluation of asphalt mixture quality. For example, Austroads uses resilient modulus to determine the optimum bitumen content in the mix design stage [34]. In general, the resilient modulus of an asphalt mixture can be determined using different methods, such as a cyclic triaxial test, indirect tension test (ASTM D7369-11), or Nottingham asphalt test.

In this study, indirect tension tests were one of the methods used to determine the resilient modulus of BSM mixes. The same cylindrical specimens as those in the ITS test were used. There is no specific loading sequence for resilient modulus testing outlined in ASTM D7369 [32]. It specifies that 100 cycles of preconditioning should be carried out before testing. Sufficient cyclic loading cycles should be applied to obtain five stable cycles (<1% change in M_r over 5 cycles), and the peak load that the specimen should experience during preconditioning and testing, is between 10 to 20 percent of its peak failure load. In each cycle, a 0.1 second loading period is followed by a rest period of 0.9 seconds, to simulate traffic loading on a given point of a pavement surface.

In this research, M_r tests were conducted in both the mix design stage and on final performance testing. The M_r testing in the mix design stage was not used to select optimum bitumen content, but rather for comparison between the performances of specimens with different bitumen contents. The procedure used in this research was ASTM D7369-11 with some minor changes, such as:

1) To complete the M_r test on a bituminous mix sample, an ITS (Indirect Tensile Strength) test must first be completed on the mix. Since the M_r test is non-destructive, an

ITS test must be completed to find the appropriate loading. This ITS test was completed as specified in ASTM D6931.

2) The thickness and diameter of the specimen were measured. Thickness was measured as the average of 4 equally spaced points around the perimeter of the specimen. Diameter was measured as the average of two perpendicular diameters.

3) The specimen was preconditioned.

-100 cyclic load-cycles were performed with an amplitude of 10% ITS as determined by ASTM D6931.

-Cycles were composed of a 0.1 second halversine impulse followed by 0.9 seconds of rest.

-The specimen was rotated 90 degrees and an additional 100 cycles of the same loading were completed. (No rest is necessary between rotations)

-If the total vertical deformations were greater than 0.025 mm during preconditioning, the load was reduced to the minimum by which measureable deformations still occur.

4) Testing the specimen.

-Measured deformations were recorded from each sensor as soon as preconditioning was over.

-No rest is necessary between preconditioning and testing.

-Testing was completed at loads between 10 and 20% of the mix's ITS

-Load cycles were composed of 0.1 seconds of loading followed by 0.9 seconds of rest.

-The sample was loaded with 10 cycles at 10% of ITS, 10 cycles at 13% of ITS, 10 cycles at 16% of ITS, and 10 cycles at 19% of ITS.

-After testing the specimen on the first diametral plane, it was rotated by 90 degrees and the testing was completed again.

-Once the M_r testing was finished, an ITS test was performed to determine the actual strength of that specific specimen as per ASTM D6931.

5) The resilient modulus was calculated according to ASTM D4123 based on previous experience at McMaster.

$$Mr = P * \frac{0.27 + v}{dh * t}$$

where,

M_r = resilient modulus (MPa)
P = average peak load applied over the last five cycles (N)
v = Poisson’s ratio, 0.35 assumed for bituminous mixture
dh = instantaneous displacement due to applied cyclic load (mm)
t = specimen thickness (mm)

3.5 Triaxial Tests

In this study, triaxial tests were carried out to determine the permanent deformation and resilient modulus of stabilized and non-stabilized compacted specimens under repeated loading.

3.5.1 Test Apparatus

The triaxial apparatus used for this testing is shown in Figure 3.5.1. It consisted of a triaxial pressure chamber, loading device, as well as a load and deformation measuring system as specified in AASHTO T307-99 [35]. The pressure chamber used air to apply confining pressure over a sealed specimen. Two LVDTs were attached to the chamber piston rod outside of the chamber in order to measure vertical displacements. The loading device was a top loading electro-hydraulic testing machine that was capable of applying repeated haversine shape load pulses. A load cell was mounted between the actuator and chamber piston rod to measure load magnitudes. The setup was able to measure testing time, displacement of the actuator, two LVDT displacements, and the load magnitude at a rate of 200 measurements per second.

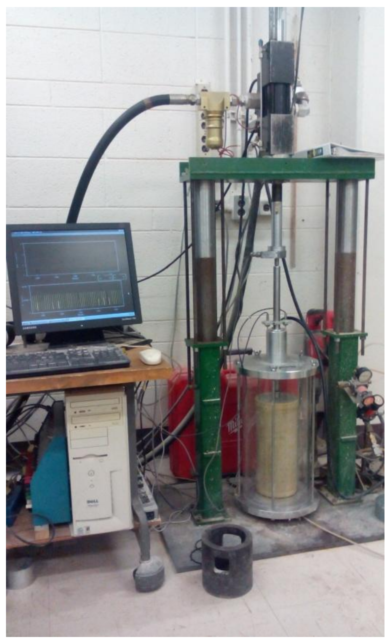


Figure 3.5.1: Triaxial Testing Apparatus

3.5.2 Specimen Preparation

In this test setup, gyratory compacted foamed bitumen stabilized specimens with diameter of 150 mm were used. Since the height of a gyratory compacted specimen was only 100 mm, three of them were attached together to create a 300 mm tall specimen for cyclic triaxial testing. This was achieved by removing the filter paper from the ends of the gyratory specimen, brushing off any loose particles, applying a thin coating of heated bitumen to the interface with a paint brush, and then applying a small vertical pressure on the 300 mm tall specimen. A completed specimen is shown in Figure 3.5.2. FASB specimens were tested either in the dry state or in the wet state after being soaked in a lukewarm water bath for 24 hours.

Figure 3.5.2: FASB Specimen bonding for cyclic triaxial testing

Additionally, triaxial testing was completed on specimens of natural aggregates without being stabilised with foamed bitumen. These specimens were compacted in the 150 mm diameter split mould to a height of approximately 315 mm. They were compacted in 5 lifts using the vibratory compaction hammer as described in AASHTO T307-99. Specimens were compacted to 100% of maximum dry density at optimum water content determined from modified Proctor compaction. The aggregate blends were prepared in the same way as those of the stabilized material specimens.

3.5.3 Repeated Load Permanent Deformation (RLPD)

The RLPD test was carried out under triaxial stress conditions. With a constant cell (or confining) pressure, cyclic axial stress was applied on the specimen, which was 150 mm diameter and 300 mm tall. A stress cycle consisted of a 0.1 second loading period, followed by a 0.9 second rest period. For a select confining pressure and cyclic axial stress amplitude, 30000 stress cycles were applied. The axial deformation of the specimen was measured during cyclic loading by two LVDTs at opposite points of the specimen's diameter. Before the testing sequence, the specimen was first conditioned at the confining pressure of 103 kPa (following AASHTO T307-99) to eliminate the effects of the time between compaction and loading/reloading. It also helps to improve contact between the sample cap and specimen before testing [35].

This test was completed on both foamed bitumen stabilized and non-stabilized materials. The loading sequences for conditioning and testing were the same for all materials. They are shown in Table 3.5.1.

Table 3.5.1: RLPD Test Sequences

Sequence #	σ_3 (kPa)	Deviator load (N)	Deviator stress (kPa)	# of pulses	Pmax (kPa)	q/p
conditioning	103.4	1830	103.6	750	137.9	0.75
1	137.9	5100	288.6	30000	234.1	1.23

3.5.4 Triaxial Resilient Modulus

Triaxial resilient modulus tests were carried out following AASHTO T307-99. For both non-stabilized materials and foamed bitumen stabilized materials, a 300 mm tall by 150 mm diameter specimen was loaded 0.1 seconds with 0.9 seconds of rest every 1 second, for 15 sequences of 100 load cycles. The load was applied axially to a specimen, which was exposed to a confining pressure to simulate the ground around it. Displacement readings were taken in the direction of loading at opposite points of the specimen's diameter. Before this loading sequence, the specimen was first conditioned at the confining pressure of 103kPa in order to eliminate the effects of the time between compaction and loading/reloading, and also to improve contact between the sample cap and specimen before testing [35]. The loading sequences for conditioning and testing are summarized in Table 3.5.2, which corresponds to those outlined in AASHTO T307-99.

Table 3.5.2: Loading sequences in cyclic triaxial M_r tests

AASHTO T307-99 for Base/Subbase							
sequence #	σ_3 (kPa)	Deviator stress (kPa)	Cyclic stress (kPa)	Constant stress	# of pulses	Pmax (kPa)	q/p
0	103.4	103.4	93.1	10.3	500-1000	137.9	0.75
1	20.7	20.7	18.6	2.1	100	27.6	0.75
2	20.7	41.4	37.3	4.1	100	34.5	1.20
3	20.7	62.1	55.9	6.2	100	41.4	1.50
4	34.5	34.5	31	3.5	100	46.0	0.75
5	34.5	68.9	62	6.9	100	57.5	1.20
6	34.5	103.4	93.1	10.3	100	69.0	1.50
7	68.9	68.9	62	6.9	100	91.9	0.75
8	68.9	137.9	124.1	13.8	100	114.9	1.20
9	68.9	206.8	186.1	20.7	100	137.8	1.50
10	103.4	68.9	62	6.9	100	126.4	0.55
11	103.4	103.4	93.1	10.3	100	137.9	0.75
12	103.4	206.8	186.1	20.7	100	172.3	1.20
13	137.9	103.4	93.1	10.3	100	172.4	0.60
14	137.9	137.9	124.1	13.8	100	183.9	0.75
15	137.9	275.8	248.2	27.6	100	229.8	1.20

3.6 Unconfined Compression

The unconfined compression test is a simple test to determine the ultimate compressive strength of a cohesive material. The specimen is axially loaded without confinement therefore the specimen must have cohesion to develop resistance to loading.

3.6.1 Test Apparatus

The unconfined compression (UC) test was performed using an MTS loading machine. The frame and load cell were rated to a maximum load of 50000 lbs and the actuator was rated to 35000 lbs. The deformation of the specimen and the applied load were measured as well as the testing time. The system recorded 1 reading per second of the load, displacement, and time. The actuator loaded the specimen at a controlled rate of displacement between 0.5% and 2% of the specimen height per minute as specified in ASTM D2166M (Standard Test Method for Unconfined Compresive Strength of Cohesive Soil)[36]. Approximately 4 mm per minute was used, which equates to 1.3% per minute of the 300 mm specimens. The loading device is pictured in Figure 3.6.1.

Figure 3.6.1: Unconfined Compression Test Apparatus

3.6.2 Specimen Preparation

The specimens used for this test were approximately 300 mm tall with 150 mm diameter. These specimens were tested in triaxial M_r and RLPD before being brought to ultimate failure in UC. Most tests were completed in the dry state, however some specimens are tested in the soaked state after 24 hours in a lukewarm water bath immediately prior to loading.

4 Mix Design

This chapter discusses the mix design for foamed asphalt stabilization of a select natural aggregate, select RAP, and an aggregate blend with 50% RAP. The preparation and testing of 150mm specimens at varying bitumen content was completed using the Asphalt Academy mix design method to determine the optimum bitumen content. Varying bitumen content targets for each parent material are provided in Table 3.1.2.

4.1 Specimen Preparation

4.1.1 Aggregate Preparation

The aggregates were first air dried because the residual binder could have become reactivated if oven dried. Aggregates were blended to their densest gradation as indicated in Table 3.2.2. When blending material, the appropriate dry mass of each material (100% GB, 100% RAP, 100% passing #200, etc) was weighed as a percentage of the total desired mass, based on the percentages used to calculate the densest gradation chart.

Prior to foaming, the aggregates were first brought to optimum mixing water content (OMWC), which was 80% of the material's optimum moisture content, based on modified Proctor compaction tests. Water was added slowly with a spray bottle while stirring regularly to prevent materials from segregating. The moist material was stored in a sealed container and left for no less than four hours to allow the moisture content to distribute uniformly.

4.1.2 Foaming

The bitumen foamer and the foaming method presented in Section 3.3 were used to carry out this mix design. A bitumen flow rate of 35 g/sec was adopted to make specimens with approximately 350-700g of bitumen in 10-20 seconds. The bitumen flow rate was controlled via the metering valve and proper air pressure applied to the bitumen tank. Prior to adding foamed bitumen to aggregates in the mixer, at least two foaming trials were completed to calculate the actual foam flow rate, and check the quality of foam in a 20L metal bucket. The final flow rate was then used to determine the amount of foam added to the aggregate during mixing, by measuring the time over which the foam was applied.

4.1.3 Mixing

Mixing was completed using a Hobart model A-200 table-top planetary mixer shown in Figure 4.1.1. It was large enough to mix 14 kg of aggregates along with water and bitumen foam, without jamming. This mix was enough for the preparation of three 150 mm gyratory specimens.

Figure 4.1.1: Hobart A-200

Mixing was started after the aggregates had been prepared and the foaming trials were completed. First, the mixing bowl and agitator head were weighed prior to adding the aggregates. The aggregate at the desired mixing water content was then added to the mixing bowl, which was placed under the foaming nozzle. The mixer ran for 10 or 15 seconds prior to foaming, to ensure that jamming did not occur. Next, the foamer was engaged and the timer started simultaneously. Foaming took place for a predetermined time in order to achieve the desired bitumen content of the mix. After the foaming was completed for the desired time, the foamer was shut off and the mixer continued to run for an additional 40-50 seconds to ensure adequate mixing. Mixed Granular B material is shown in Figure 4.1.2. Asphalt Academy[6] recommends that the entire mixing takes place within 20-30 seconds when using a pugmill style mixer. This time was not adequate for the Hobart mixer. Sunarjono[37] completed his studies using a Hobart planetary mixer and found that 60 seconds of mixing time produced the best results. After mixing, the bowl, agitator head, and complete mix were weighed together to determine the actual bitumen content remaining in the mix.

Figure 4.1.2: Mixed BSM specimen, Granular B aggregate

4.1.4 Compaction

Compaction was completed using the gyratory compactor. 150 mm diameter specimens were created with 4500 grams of mixed material, corrected to dry mass. Each specimen was compacted to the maximum dry density determined by the modified Proctor compaction test on the original aggregate blends before stabilization. The density of the specimen was controlled during compaction by specifying the termination height. Final densities were subsequently determined with measurements of volume and mass for the completed, dried specimen after the curing stage.

4.1.5 Curing

Immediately after compaction, all specimens were labelled and placed in a forced draft oven set at 40℃ for 72 hours. Specimens were then left at room temperature for 24 hours to cool down before their mass and dimensions were measured for a final density calculation. Lastly, the specimens were covered in plastic bags before future testing.

4.2 Determination of Optimum Bitumen Content

There are many different methods for determining the optimum bitumen content in a foam BSM material. The methods used by Asphalt Academy, Austroads, and Caltrans are briefly discussed before indicating the method used in this research.

The Asphalt Academy[6] mix design procedure begins with preliminary tests on the aggregates to ensure that the stabilization will work, and if not, to determine how to pre-treat the aggregates to make it work. Next, the level 1 mix design includes making initial (Marshall) specimens and testing them for indirect tensile strength and moisture susceptibility using tensile strength ratio to determine an initial bitumen content, active filler content, and stabilization agent. Level 2 and 3 mix designs involve making larger specimens and further optimising the bitumen content for higher traffic roads. Proctor specimens are used for level 2, and even larger 150 mm diameter by 300 mm high triaxial specimens for level 3. Optimum bitumen contents are primarily based on the maximum ITS_{dry} however there are different minimum requirements for ITS_{dry} and ITS_{wet}, depending on traffic level. These requirements are shown in Table 4.2.1, for BSM3, BSM2, and BSM1, which increase in traffic level respectively.

Table 4.2.1: Asphalt Academy BSM-Foam mix design requirements[6]

Test	Specimen Diameter	BSM1 (kPa)	BSM2 (kPa)	BSM3 (kPa)	Purpose
ITS_{dry}	100mm	>225	175 to 225	125 to 175	Indicates optimum bitumen content
ITS_{wet}	100mm	>100	75 to 100	50 to 75	Indicates need for active filler
TSR	100mm	N/A			Problem when TSR<50 and ITSdry>400
ITS_{equil}	150mm	>175	135 to 175	95 to 135	Optimize bitumen content
ITS_{soaked}	150mm	>150	100 to 150	60 to 100	Check value on ITSwet

Austroads' mix design procedure [34] uses the IDT resilient modulus test to determine optimum bitumen content. They recommend starting with three bitumen contents between two and four percent by mass. Aggregates with higher fines content typically require higher bitumen content. Three wet and three dry samples are fabricated at each bitumen content and then tested for resilient modulus. The results are plotted as resilient modulus versus bitumen content to determine the optimum bitumen content. The optimum bitumen content is selected based on maximum modulus, wet to dry modulus ratio, and local experience.

Chandra et al.[38] evaluated mix design methods for foamed bitumen stabilized mixes. They summarized that Caltrans' procedure has two levels of testing. Both levels involve ITS testing performed on Marshall compacted specimens, which are 100mm in diameter and 63.5mm tall. Only soaked ITS testing is required in Caltrans' procedure however unsoaked (dry) specimens may be tested as an option. Specimens are prepared at varying bitumen content up to 4% and an additional control mix is left untreated. As bitumen content increases, the soaked ITS tests are compared to the ITS of the untreated mix. The first mix which sustains a soaked ITS of 100kPa greater than the untreated mix is determined to be the optimum bitumen content.

In this research, the optimum bitumen content was determined based on the ITS test results from both the dry and soaked states. Six different bitumen contents were used for each material blend, and three specimens were created at each bitumen content. Of these three specimens, two were used for wet ITS tests, and one for dry ITS tests. The optimum binder content was chosen based on clear maximums indicated by ITS_{dry} or ITS_{wet} while ensuring that the TSR remained above 60%.

IDT resilient modulus tests were also conducted on one wet specimen and one dry specimen per bitumen content. The resilient modulus results were not used for the determination of the optimum bitumen content however they were still completed during this stage.

4.2.1 Mix Design Results

The summary of ITS_{wet}, ITS_{dry}, and TSR results at various bitumen contents for each aggregate blend are presented in Figure 4.2.1, Figure 4.2.2, and Figure 4.2.3.

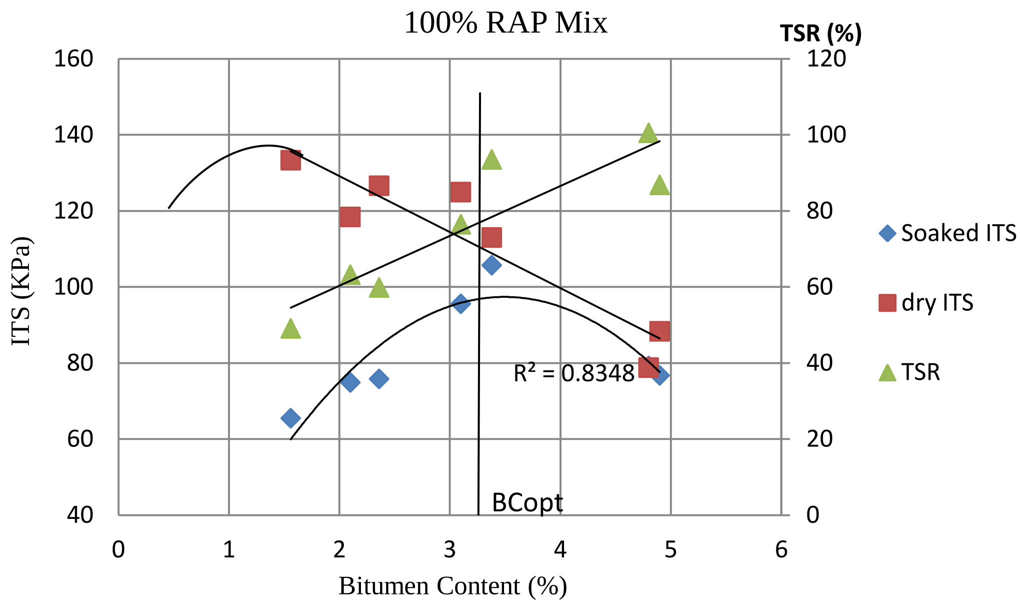


Figure 4.2.1: Mix design results for 100% RAP mix

Figure 4.2.1 displays all ITS test results from the mix design stage completed on the 100% RAP with BSM foam mix. The ITS_{wet} curve clearly indicates an optimum bitumen content at around 3.3-3.6% bitumen. ITSdry appears to continuously drop off along the x-axis based on all acquired data. However, it can be assumed that since the residual binder in the mix was inactive, the cohesive properties would drop off at extremely low binder content, but may not fully disappear. The line of best fit was therefore extended into an expected curve of best fit in the lower range of binder contents.

The optimum binder content was chosen at 3.3% for its maximum soaked strength however a binder content of 2% also exhibited desired values in the dry ITS curve. 2% binder correlates to a TSR of 60% but any lower binder content posed risks based on recovered data. 60% TSR is still acceptable under Asphalt Academy [6] recommendations. The 100% RAP mix at 2% binder content classified as a BSM3 in the Asphalt Academy classification shown in Table 4.2.1.

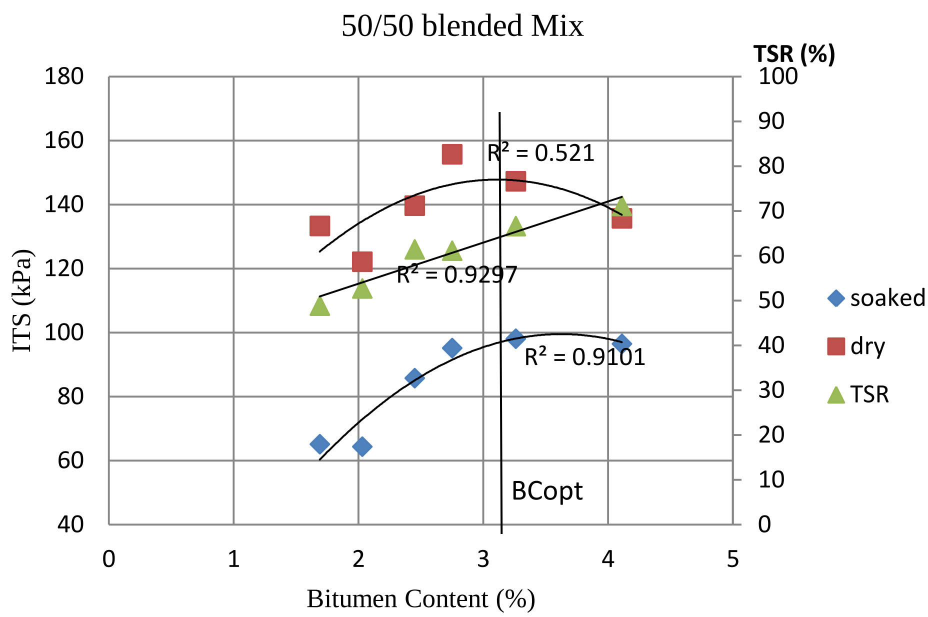


Figure 4.2.2: Mix design results for 50/50 mix

Figure 4.2.2 displays all ITS test results from the mix design stage that was completed on the 50/50 RAP/granular B with BSM foam mix. These curves emulate an ideal mix design with clear maximum values in both ITS_{dry} and ITS_{wet} at nearly the same bitumen content. The optimum bitumen content was chosen at 3.1% corresponding to the peak in ITS_{dry} with a value of approximately 150 kPa. ITS_{wet} peaked a little bit higher at a bitumen content range of 3.5-3.7%. Acceptable TSR values were obtained throughout the entire range of specimens according to Asphalt Academy's minimum of 50%. Although peak values were obtained at higher bitumen contents according to the lines of best fit, the actual specimen results show consistent values around 3% bitumen content. The single specimen with over 4% bitumen could be an anomaly. Lower ITS values would shift the peak bitumen content lower as well. The results at 3.1% binder for the 50/50 mix are classified as BSM3 according Asphalt Academy [6]. However, ITS_{wet} values were nearing the minimum for BSM1.

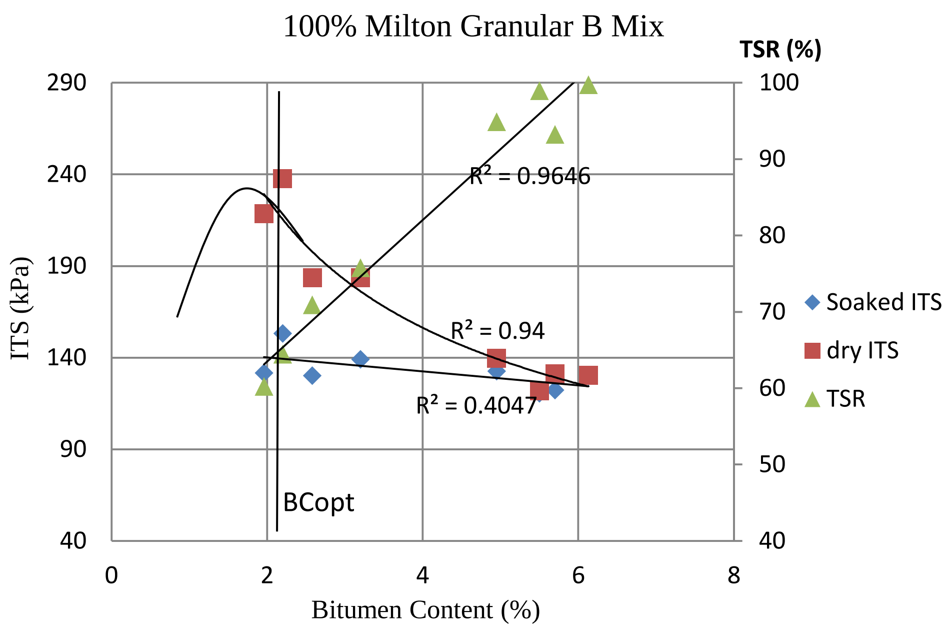


Figure 4.2.3: Mix design results for 100% Milton granular B mix

Figure 4.2.3 displays all ITS test results from the mix design stage completed on the 100% Granular B with BSM foam mix. Based on these results, the ITS_{wet} in this mix was not sensitive to bitumen content. ITS_{dry} was greatly affected and it showed strongest values around 2% binder. Similar to the 100% RAP mixture, the acquired values of ITS_{dry} did not drop off at low bitumen contents. It is known that the unstabilised granular materials do not produce much tensile strength. The expected curve of best fit is drawn at low binder content in order to show that the ITS_{dry} should begin to approach zero. The optimum binder content was chosen at 2% where both ITS_{dry} and ITS_{wet} were maximized, and TSR was close to 60%.

According to Figure 4.2.1, Figure 4.2.2, and Figure 4.2.3, all parent materials resulted in a similar trend for the variation of TSR with bitumen content. In general, TSR increases with increasing binder content. The variation of ITS_{wet} and ITS_{dry} with bitumen content was different for all three materials.

Based on the figures, optimum bitumen contents were chosen for each material. The 100% RAP blend indicated optimum bitumen content at 3.3% based on the maximum ITS_{wet} and a TSR value of 80%. The 50/50 blend showed a peak in both ITS_{dry} and ITS_{wet} at bitumen contents above 3%, and 3.1% was chosen as optimum. Lastly the 100% Milton Granular B mix showed highest strength in wet and dry states at low bitumen contents; however TSR simultaneously reduced to an acceptable value of TSR~65%. Therefore the optimum bitumen content was chosen as 2%. Quantitative results of the mix design are summarized in Table 4.2.2.

Table 4.2.2: Summary of optimum bitumen contents from mix design

Parent Material	Optimum bitumen content (%)	ITS_{dry} (kPa)	ITS_{wet} (kPa)	TSR (%)
100% RAP	3.3%	112	96	75
50/50 blend	3.1%	147	97	65
100% Granular B	2.0%	228	140	64

4.2.2 Mix Design Discussion

TSR results appeared exactly as expected with increasing TSR as bitumen content increased for all parent materials. TSR is a ratio of wet ITS to dry ITS. A value of 100% indicates that the wet tensile strength is the same as the dry tensile strength. As explained by Fu [7], the tensile strength of aggregate bonds such as matric suction and weak chemical cementation are easily reduced by the presence of water whereas tensile strength of bitumen particles is not. Therefore increasing bitumen content increases moisture resistance.

Ideally, the variation of ITS_{dry}, ITS_{wet}, and TSR with bitumen content for all parent materials should be similar to the 50/50 RAP/aggregate blend results presented in Figure 4.2.2, in which the ITS_{dry} curve had a clear peak with ITS_{dry} maximized at the bitumen content of 3.2%. Clean aggregates themselves do not have significant tensile strength, therefore an increasing binder content is expected to increase tensile strength of the mix. A maximum tensile strength should occur before bitumen particles get too large and interconnected at high bitumen contents.

Based on Emery's[1] equation for predicting optimum bitumen content of a foamed asphalt stabilized material, the increase of RAP content in a RAP/aggregate blend leads to decreased foamed asphalt content to obtain the optimum performance. This is attributed to two reasons: 1) the residual bitumen content in RAP is still active and has some remaining adhesive property, which can aid the added bitumen; and 2) RAP typically contains lower fines content, and foamed bitumen stabilization relies on fine material for even dispersion. It should be noted that for a RAP/aggregate blend, the residual bitumen in the RAP tends to promote better dispersion, particularly for fine RAP.

The optimum bitumen content from the test results are inconsistent with Emery's equation. A close examination reveals that the inconsistency could be attributed to the mixing process in this study. The 100% RAP mix did show some conformance to the equation if the dry tensile strength peak was used for optimum binder content determination where 2% was achieved and 1.5% was expected. However, the wet ITS results were higher than expected. 100% Granular B results were far from predicted. They indicated optimum binder content at 2% while 4% was expected. Lastly, the 50/50 blend results indicated optimum binder content at 3.10% while the optimum was expected at 2.80%, which is relatively close.

In the 100% RAP mix design, the higher optimum binder content in wet material was likely due to the reduction of frictional properties created by inactive binder during soaking. More strength was provided by the binder itself and therefore, a higher binder content resulted in a greater strength up to a certain extent.

In the 100% granular B mix, the low optimum bitumen content was likely a result of the mixing stage. Sunarjono [37] used a Hobart mixer and generated acceptable results, however Asphalt Academy [6] recommends that a pugmill mixer be used. During mixing of 100% Granular B, the mixer would often slow down, or jam up completely. Jamming was more common in mixes with higher bitumen content. Figure 4.2.4 clearly shows shiny bitumen globs in a 100% Granular B mix with high bitumen content. Globs of bitumen in the mix indicate an uneven mix, or excessive amounts of binder which create weak zones by acting as a lubricant. Jamming did not occur in mixes including RAP, possibly due to the lower bulk density of material, or reduction of maximum particle size. For a proper mixing process, Emery's equation is expected to be valid.

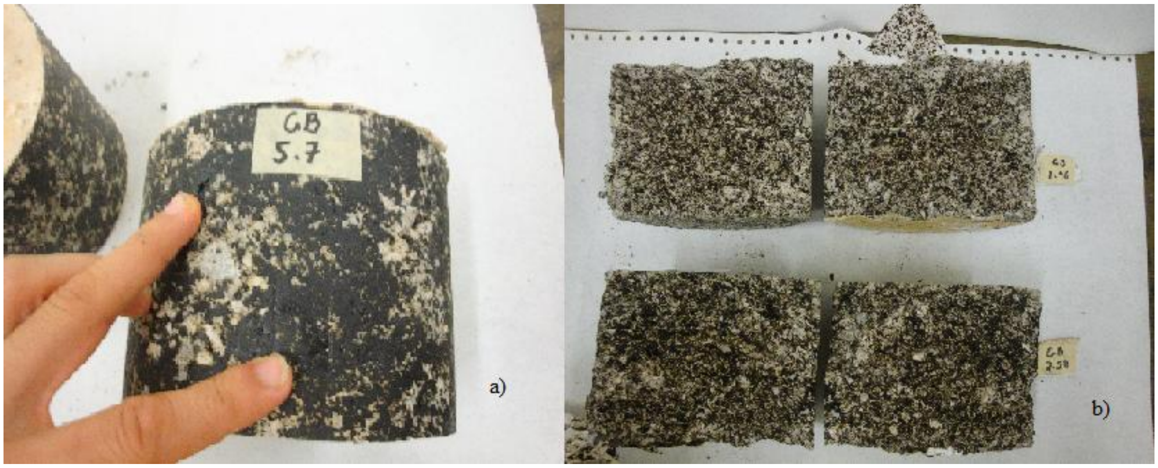


Figure 4.2.4: Visual inspection of binder content: a) 100% aggregate mix with 5.7% foamed bitumen showing visible bitumen globs, b) 100% aggregate mixes with 1.96% binder (top) and 2.58% binder (bottom) showing more even dispersion

Ultimately, the values in Table 4.2.2 clearly represent the optimum mixes that we were able to create given the equipment provided. Therefore these values were used to create the optimum mixes for final performance testing.

5 Test Results and Analysis

This chapter summarizes the results of performance testing on FASB and unstabilized base specimens under the various conditions. The performance tests included indirect tensile strength and resilient modulus, triaxial repeated load permanent deformation and resilient modulus, and unconfined compression. The influences of various factors on the performance of FASB specimens are now discussed, by taking into account the intrinsic mechanisms.

5.1 Indirect Tension Tests

Indirect tension tests were performed to determine the influence of asphalt binder content on the performance of mixes composed of three different aggregate blends. Within each aggregate blend, all parameters were held equal aside from bitumen content. Actual densities were calculated as being close to the targets for all specimens (see Appendix). Target densities were 2310 kg/m^3 for 100% Granular B aggregate, 2254 kg/m^3 for the 50/50 blend, and 2047 kg/m^3 for 100% RAP, as presented in Section 3.2.3.

It is also notable that fines content differs between the mixes. 100% Granular B naturally contains 7.01% fines passing the #200 sieve. The 50/50 blend was adjusted to be similar in fines to granular B. It was adjusted to 6.16% fines while maintaining optimum density gradation. Lastly, the RAP was left as a representative RAP with 1.30% fines in the state that it arrived.

5.1.1 Indirect Tensile Strength

Figure 5.1.1 and Figure 5.1.2 summarize the variation of ITS with asphalt binder content for different mixes under dry and wet conditions, respectively. The corresponding TSR are presented in Figure 5.1.3. The figures display results of mix design testing for each blend on the same graph in order to compare between aggregate blends. These tests were only completed during the mix design stage. It should be noted that the optimum asphalt binder contents were found to be 2%, 3%, and 3.3% for 100% GB, 50/50 blend, and 100% RAP respectively.

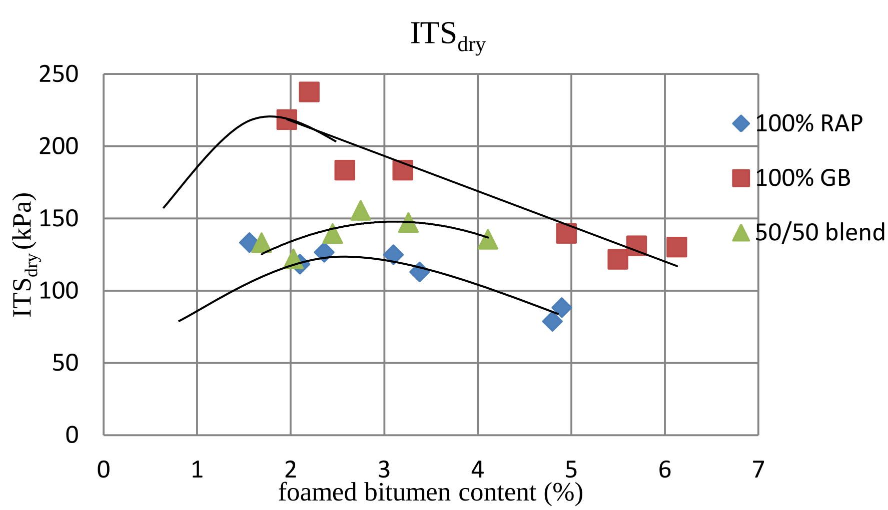


Figure 5.1.1: Variation of ITS$_{dry}$ with bitumen content

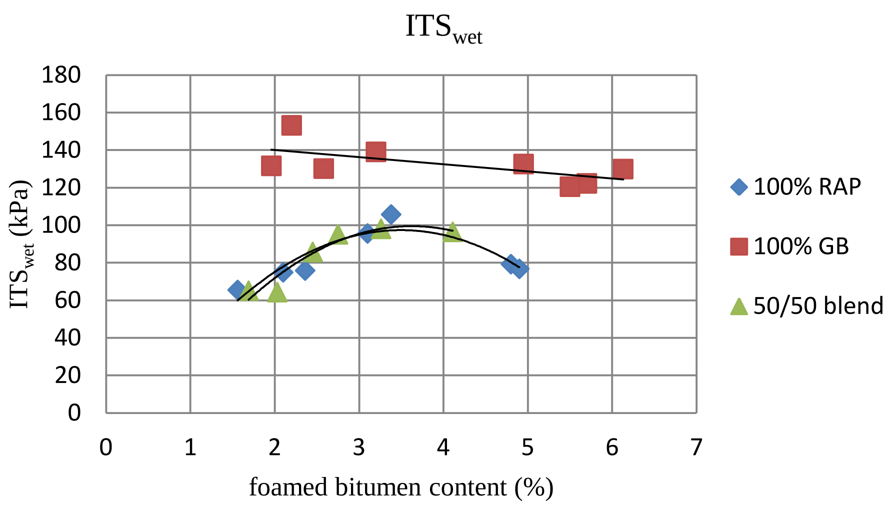


Figure 5.1.2: Variation of ITS$_{wet}$ with bitumen content

When all mixes were compacted to their maximum dry densities obtained from modified Proctor compaction, Granular B aggregates had greater indirect tensile strength compared to specimens which include RAP, at all respective asphalt binder contents, in both dry and wet testing. This is likely due to the high density of the compacted natural aggregates, as well as the high angularity of particles. The angularity creates a strong interlocking structure that increases friction between aggregate particles. One would expect that friction would not have a strong effect on the tensile strength. However, it has been shown that interlocking of angular aggregate particles increases the tensile strength of hot mix asphalt concrete by many researchers, including Chen et al. [39], Anirudh et al.[40], and Ahmed et al. [41].

The 50/50 blend appears to behave more similarly to the 100% RAP blend than the 100% Granular B when looking at the optimum bitumen content. The ITS$_{wet}$ values were almost identical between the two mixes containing RAP (Figure 5.1.2). The ITS$_{dry}$ plot displays the 50/50 blend with an optimum mix around 25 kPa stronger than 100%

RAP, whereas the optimum 100% Granular B mix was around 90 kPa stronger than 100% RAP (Figure 5.1.1). This is consistent with the finding by Fu [7] that bitumen content provides a more significant amount of the specimen's strength in the wet state. This would explain why 100% RAP at the optimum bitumen content shows a smaller difference in strength to the mixes containing Granular B, in the wet state compared to the dry state. In dried specimens, aggregate interlock and friction plays a bigger role, which is more dominant in specimens containing Granular B. Density did not appear to have a large influence when compacted to 100% modified Proctor compaction density, since the density of the 50/50 blend (2254 kg/m^3) was 207kg/m^3 more than 100% RAP (2047 kg/m^3), and only 56kg/m^3 less than 100% GB (2310 kg/m^3), yet it still behaved more similarly to RAP.

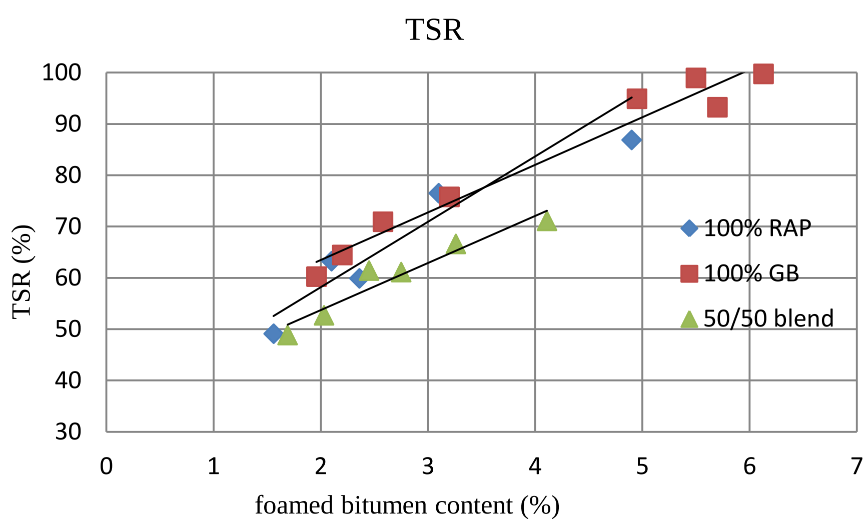


Figure 5.1.3: TSR of different materials at various foamed bitumen contents

Figure 5.1.3 summarizes the variation of the TSR value with foamed asphalt content for all parent materials. TSR increases as bitumen content increases. However, the rate at which TSR increased appears highest in the 100% RAP mix and lowest in the

50/50 blend mix. The 100% RAP mix also had a residual bitumen content, which is known to improve moisture resistance similar to the newly added bitumen. However, this does not explain the rate of TSR increase in the 50/50 blend compared to that of 100% Granular B. By this theory, the 50/50 blend should also have a higher TSR at lower bitumen content compared to Granular B, since it contained residual bitumen as well. Moisture induced damage is not a unique function of binder content. The Granular B mix had higher density and particle angularity compared to the 50/50 blend which also improves moisture resistance. The mineral makeup of the aggregates can have an effect as well, for instance limestone is typically less moisture susceptible.

It is noteworthy that both 100% Granular B and the 50/50 blend had a similar fines content of 6-7%, while the 100% RAP only had 1.3% fines. Fines are suspected to increase moisture susceptibility in Granular materials, and quite possibly in a foam stabilized material as well. In FASB, fine content also improves dispersion of bitumen, which would increase moisture resistance. The moisture susceptibility would only increase due to fines content once large agglomerations of fine materials start to form in the compacted mix. This is expected to occur in excess of 15% fines, which is not represented in this study[10].

5.1.2 Indirect Tension Resilient Modulus

Similar to the ITS test, IDT M_r tests were conducted on the specimens used during the mix design stage. Figure 5.1.4 to Figure 5.1.6 summarize the results of resilient modulus obtained from indirect tension testing under various conditions. Figure 5.1.5 is simply a more focused view of the specimens containing RAP from dry testing.

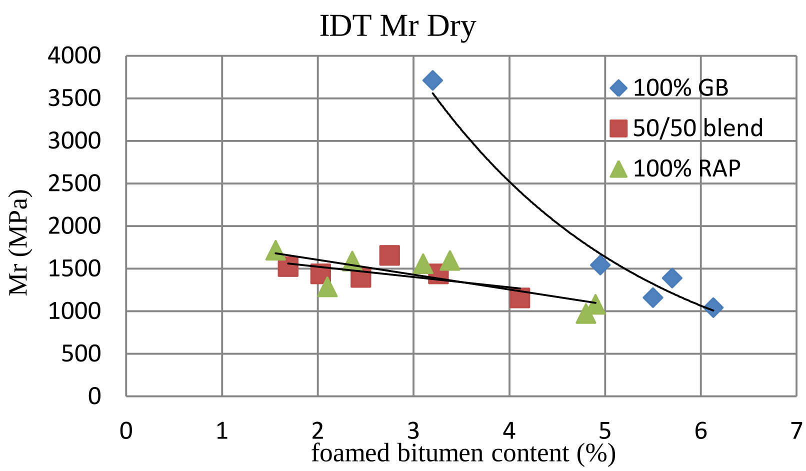


Figure 5.1.4: Indirect tension resilient modulus results (dry)

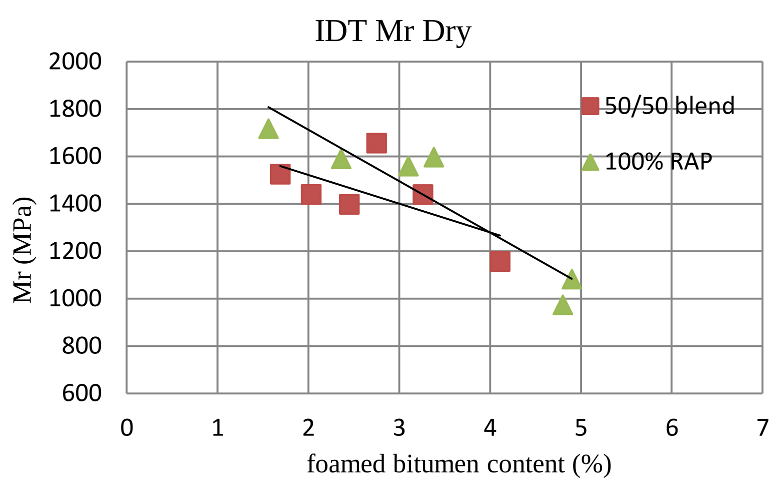


Figure 5.1.5: IDT M_r (dry) specimens containing RAP

The results for foam stabilized natural Granular B material shown in Figure 5.1.4 displays that these specimens have higher resilient modulus compared to those containing RAP. Figure 5.1.5 presents IDT resilient modulus of specimens containing RAP under dry condition. The difference in M_r between the 100% RAP and 50/50 blend is observed around 10% or less, which is not significant. For the bitumen content in the range of 1.5 to 3.5%, the resilient modulus of 100% RAP specimens were marginally higher (~100MPa) than that of 50/50 RAP/aggregate blends. The results have been known to be inflated in IDT M_r due to the nature of the material [7].

The soaked IDT M_r results of all specimens are presented in Figure 5.1.6. There are clear differences between aggregate blends in these results. The optimum bitumen content can be identified from the maximum resilient modulus, namely 3%, 3.3%, and 2.5% for 50/50 blends, 100% RAP, and 100% natural aggregates, respectively. The results in Figure 5.1.6 also reveal that specimens of foam stabilized natural aggregates had the highest resilient modulus, while the 50/50 blends had the lowest modulus. The specimens of 100% RAP stabilized by foam had higher IDT M_r values than that of the 50/50 blends. Similar observations on the influence of RAP contents on triaxial M_r values is reported by Luo [5].

The high M_r values of foam stabilized natural aggregates are attributed to the strong interlocking of angular particles, while the better performance of 100% RAP compared to the 50/50 blend is likely related to the residual bitumen content in the RAP. It should be noted however, these conclusions may only be applicable to mixes compacted to their maximum density of modified Proctor compaction because RAP is known to resist compaction efforts.

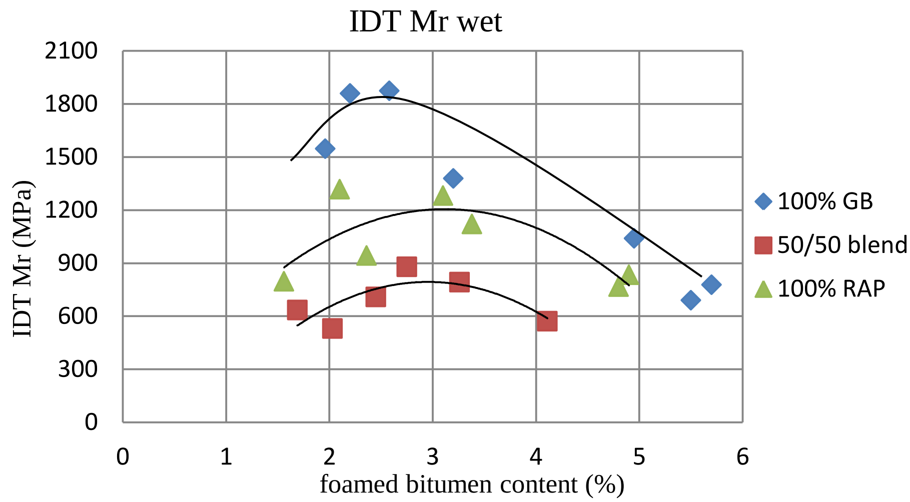


Figure 5.1.6: Indirect tension resilient modulus results (wet)

5.2 Triaxial Tests

In the previous section, the performances of various foam stabilized aggregates (including 100% natural aggregates and 100% RAP) fabricated at the design of the optimum mix, were examined. They were tested for indirect tensile strength and indirect tension resilient modulus. In this section, the efficiency of foam stabilization is further examined by comparing the performances of untreated original aggregates and RAP/aggregate blends with foam stabilized materials of the same parent aggregate blends. To achieve this goal, a series of cyclic triaxial tests were carried out to determine the accumulative deformation induced by repeated loading and the resilient modulus under various conditions. Details about the repeated load permanent deformation tests and the triaxial resilient modulus following AASHTO T307-99 were described in Section 3.5.

5.2.1 Repeated Load Permanent Deformation

5.2.1.1 Loading Pattern Trials

The RLPD testing was completed using the same test sequence on all specimens to compare the difference in permanent deformation based on mix constituents rather than loading type. This was completed for the most part, however the first stabilized specimen (specimen GB2.35) underwent some trial and error to determine the preferred testing sequence. Two different tests were completed on the first specimen. The first test included two sequences at different stress ratios shown in Table 5.2.1, which were selected from the loading sequences for cyclic triaxial resilient modulus tests. The second test was only one sequence which was used for all other specimens. It can be seen in Section 3.5.3.

Table 5.2.1: First RLPD test program

sequence #	σ_3 (kPa)	Deviator load (N)	Deviator stress (kPa)	# of pulses	Pmax (kPa)	q/p
conditioning	103.6	1830	103.6	750	138.1	0.75
1	69	3425	193.8	10000	133.6	1.45
2	137.9	5100	288.6	10000	234.1	1.23

Figure 5.2.1 displays a comparison of deformations based on load history. Loading sequence #2 from Table 5.2.1 is compared to the second load program, which was the same as the aforementioned sequence #2 except with 20000 more cycles. This comparison shows that the sample becomes more resistant to deformation after being loaded. The second loading shows a lower permanent strain after 30000 cycles, compared to the first loading after only 10000 cycles, all under the same stress ratio.

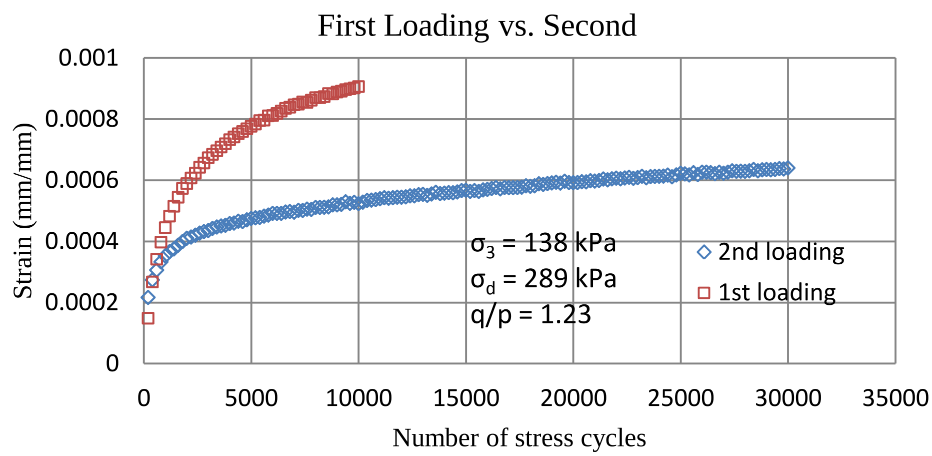


Figure 5.2.1: Comparison of a specimen (GB2.35) loaded under the same conditions in two consecutive trials

The curve of the second loading also appears to flatten out quicker, displaying a reduced rate of deformation when compared to number of cycles. Therefore once the material is subject to a certain load, it is more resistant to loads up to that maximum previously sustained. This can be further seen in Figure 5.2.2 where the first two sequences are from the first test program (first 20000 cycles). The accumulative strains over the first and second sequence were very similar; nearly 0.0014 each, while the maximum load increased from the first to the second sequence. The third curve in the figure is from the second test program. After a day of relaxation the specimen underwent the same maximum load and stress ratio as the first test program, but the permanent deformation was significantly reduced. This is shown in Figure 5.2.1.

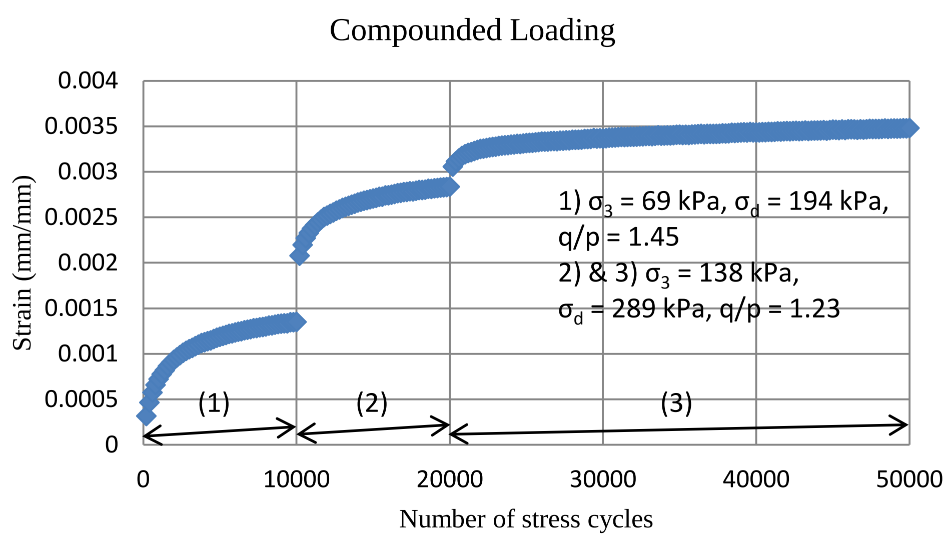


Figure 5.2.2: Compound strain of all RLPD tests on specimen GB2.35

5.2.1.2 Unstabilized Materials

Referring to Section 3.5, the specimens of unstabilized materials tested for permanent deformation induced by repeated loading were all compacted at their optimum bitumen content to the maximum modified Proctor density using a vibratory compactor. The specimen was conditioned at the confining pressure of 103 kPa by applying a cyclic axial stress following AASHTO T307-99. The permanent deformation test was performed at the confining pressure of 138 kPa and the cyclic axial stress amplitude was 289 kPa. Figure 5.2.3 shows all RLPD results from unstabilized materials. For each material, the test was duplicated to examine the repeatability of the test results.

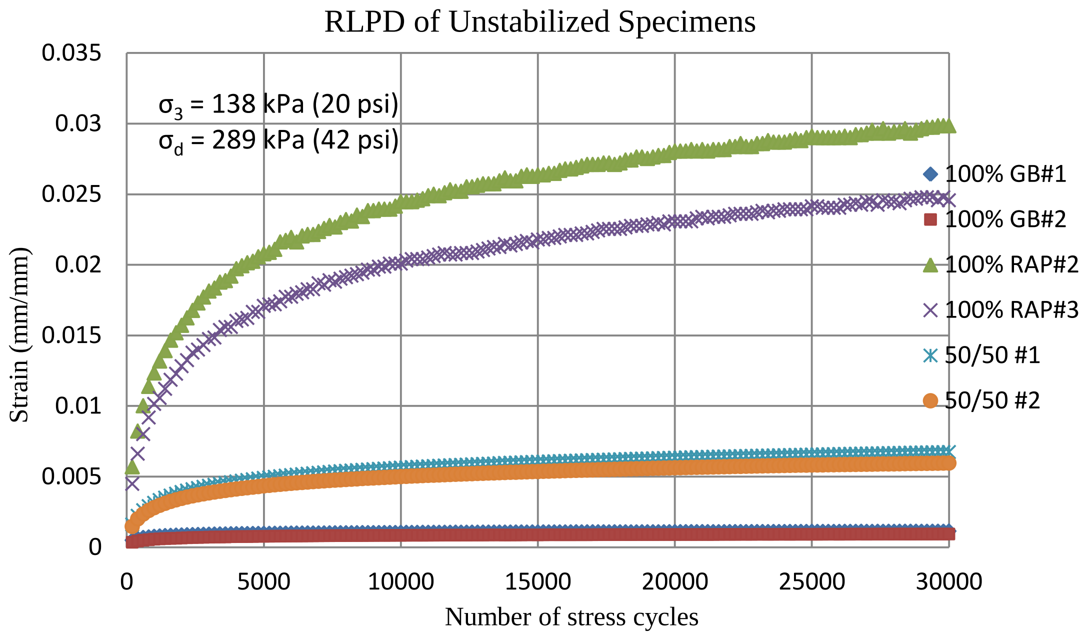


Figure 5.2.3: Repeated load permanent deformation of unstabilized materials

From the figure, it is clear that 100% RAP specimens underwent drastically higher deformation over time compared to 100% natural Granular B. The 50/50 blend also had a much higher permanent deformation than that of the 100% granular B. As summarized in Table 5.2.2, after 30000 stress cycles, the accumulative deformation of the natural aggregates corresponded to an axial strain of approximately 0.1%. The strain of the 50/50 RAP/aggregate blend was 0.63% on average, while the strain of the RAP was as high as 2.72% under the equivalent load sequence. It is also noted that the strain rate increased significantly as the RAP content was increased. After 30000 stress cycles, the deformation of all specimens still tends to increase.

Table 5.2.2: Summary of RLPD tests on unstabilized specimens

Unstabilized Specimen Measurements				
Specimen	Density (kg/m^3)	Fines (%)	Strain @ 30000	Strain rate at 30000 (strain/cycle)
100% GB#1	2308	7	0.00114	4.31E-09
100% GB#2	2262	7	0.00094	3.95E-09
100% RAP#2	1930	1.3	0.02985	1.48E-07
100% RAP#3	1977	1.3	0.02457	8.55E-08
50/50 #1	2155	6.16	0.00674	3.56E-08
50/50 #2	2180	6.16	0.00596	2.72E-08

The results in Figure 5.2.3 also reveal the effect of density on RLPD, particularly for 100% RAP specimens. More specifically, the specimen of lower density tends to have higher permanent deformation. All of these characteristics are representative of previous testing on granular materials containing RAP **[5]**.

5.2.1.3 Repeated Load Permanent Deformation of Stabilized Materials

Four mixes were designed for each aggregate blend. The only significantly differing constituent in each mix for a given aggregate blend was the foamed bitumen content, displayed in the specimen name (1.83 for specimen GB1.83). This content was still only slightly varied since the aim was to have all specimens within $\pm$0.2% by mass compared to the optimum bitumen content determined from ITS testing. The actual results turned out closer to optimum $\pm$0.6% by mass.

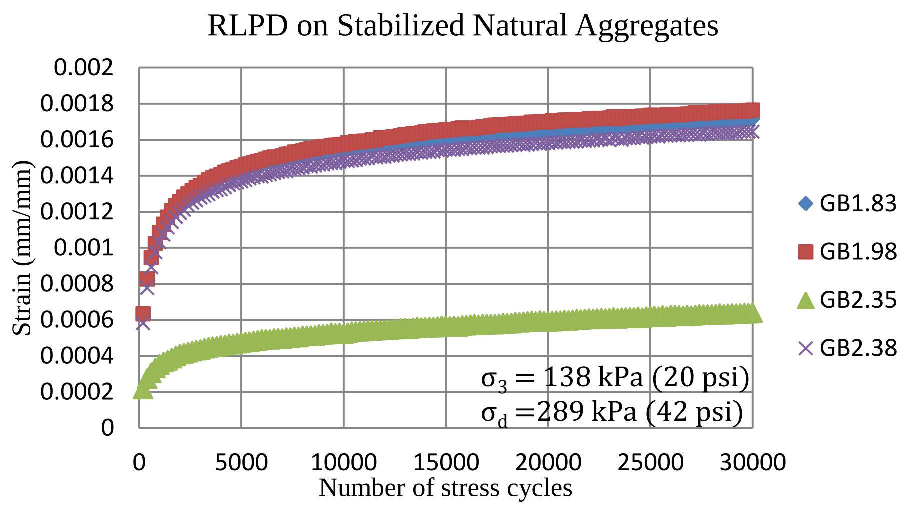


Figure 5.2.4: Summary of RLPD of all natural aggregate specimens with differing AC content

Figure 5.2.4 displays the strain versus cycle number for all foamed asphalt stabilized Granular B aggregate. This graph only includes results from the 30000 cycle test program. The curve for GB2.35 displays much smaller strains because it was the second time that this specimen was loaded, as described in Section 5.2.1.1. It is also notable that such small variation in bitumen content at an already low target bitumen content (2 $\pm$ 0.4%) does not significantly change the permanent deformation of the Granular B material tested. Less than 0.02% strain difference occurred after 30000 load cycles across the range of asphalt binder contents.

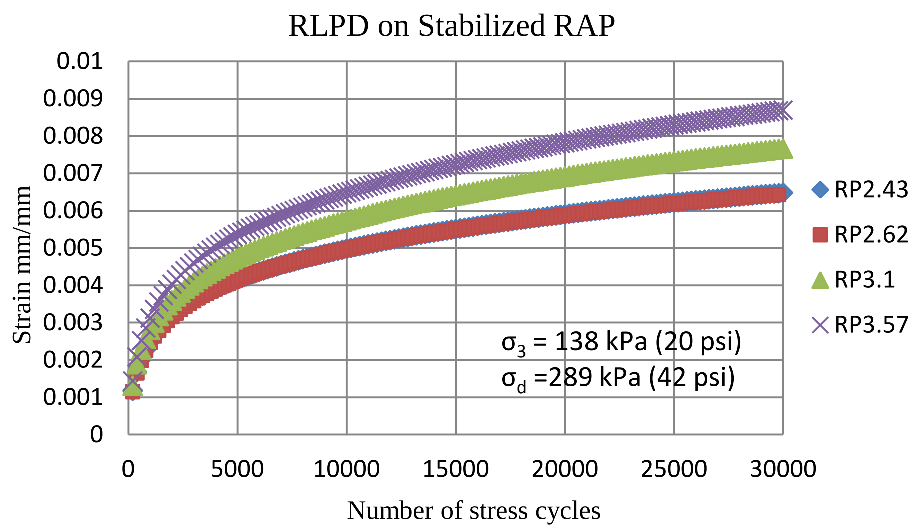


Figure 5.2.5: Summary of RLPD of stabilized RAP specimens with different AC content

Figure 5.2.5 summarizes the development of permanent deformation of stabilized RAP at different foamed asphalt content. The densities of the specimens were 2072±12 kg/m^3. The results show that the permanent deformation tended to increase as the foamed bitumen content increased. The low target density and high target bitumen content in this RAP, compared to the granular B, were likely causes of the high deformation. Higher density increases inter-particle locking, and reduces deformation. Not only did the RAP mixes have lower target density, and higher target optimum bitumen content, they also had residual bitumen which would further increase deformation in the same way as added bitumen.

The RG (50/50 blend) BSM-foam specimen results are portrayed in Figure 5.2.6. The results show that when the foamed bitumen content varies in a small range (2.4% - 3.8%), the permanent deformation after 30000 stress cycles varies in a small range of 0.5% to 0.65%. When comparing the range of accumulated strain values after 30000 cycles for each aggregate type, it appears that mixes with higher RAP content were more significantly affected by changes in bitumen content. The natural Granular B specimens had a range of 2.1 ± 0.3% bitumen and an accumulated strain range of only 0.16% to 0.18% (spans 0.02%), whereas 50/50 RAP/aggregate specimens with bitumen content of 3.1 ± 0.7% had an accumulated strain range of 0.5% to 0.63% (spans 0.13%). Lastly, RAP specimens with bitumen content in the range of 3 ± 0.6% had an accumulated strain range of 0.64% to 0.87% (spans 0.23%). The accumulated strain range of RAP spaned nearly twice that of the 50/50 blend and 18 times that of Granular B over a bitumen content range that was double that of granular B and lower than that of the 50/50 blends. This displays that over a small range of bitumen contents, the permanent deformation characteristics can be significantly affected in the 100% RAP specimens specifically.

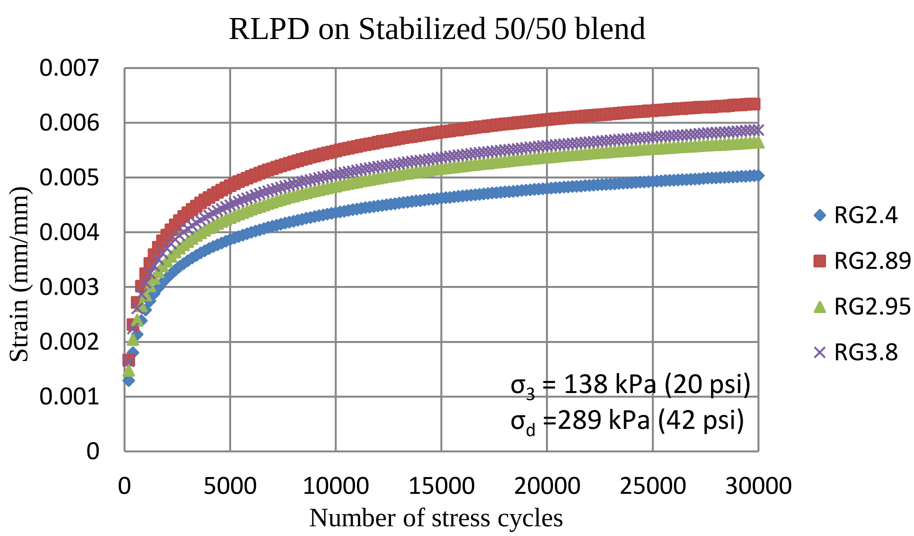


Figure 5.2.6: Summary of RLPD of stabilized 50/50 RAP/aggregate specimens with different AC content

5.2.1.4 *Effectiveness of Foam Stabilization in Permanent Deformation Reduction*

As was found in the literature review, increasing RAP content in a granular material leads to an increase in permanent deformation. The RLPD was compared across aggregate blends in foamed bitumen stabilised materials, displayed in Figure 5.2.7, and tabulated in Table 5.2.3.

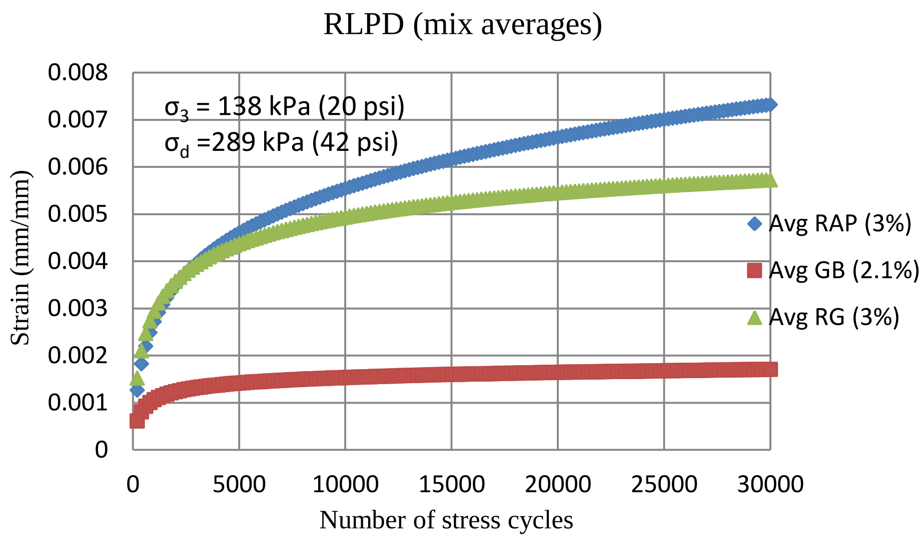


Figure 5.2.7: Comparison of RLPD based on aggregate composition. Average binder content is displayed in brackets.

Figure 5.2.7 displays the average residual vertical strain deformation for a given aggregate blend. All mixes tested with RAP used as aggregate (50% minimum) resulted in far greater deformation after 30000 cycles compared to those containing no RAP. 3.35 times as much deformation occurred in a 50% RAP blend compared to 100% GB. Deformation in 100% RAP after 30000 cycles was only 28% greater than the 50% RAP blend however the rate of deformation was still significantly higher at this point.

Therefore FASB blends containing RAP have an initial deformation resembling that of 100% RAP, but their deformation becomes stabilized earlier than 100% RAP.

Table 5.2.3: Tabulated values of RLPD comparison based on aggregate composition

Summary of RLPD Average Results					
blend	Density (kg/m^3)	Fines (%)	Target bitumen	Strain @ 30000	strain rate @ 30000 (strain/cycle)
100% RAP	2072	1.3	3.30%	0.007321	6.286E-08
50/50	2258	6.16	3%	0.005725	2.406E-08
100% GB	2323	7	2%	0.001707	6.080E-09

When comparing the performances of unstabilized materials against stabilized materials, each aggregate blend was considered separately. The unstabilized materials are labelled as “(0%)” on the graphs to abbreviate. We saw earlier that both stabilized and unstabilized RAP specimens have very different permanent deformation properties. The two are compared with each other in Figure 5.2.8. We can see that stabilization significantly improves the permanent deformation resistance of RAP. The permanent deformation of unstabilized natural aggregates is also depicted for comparison.

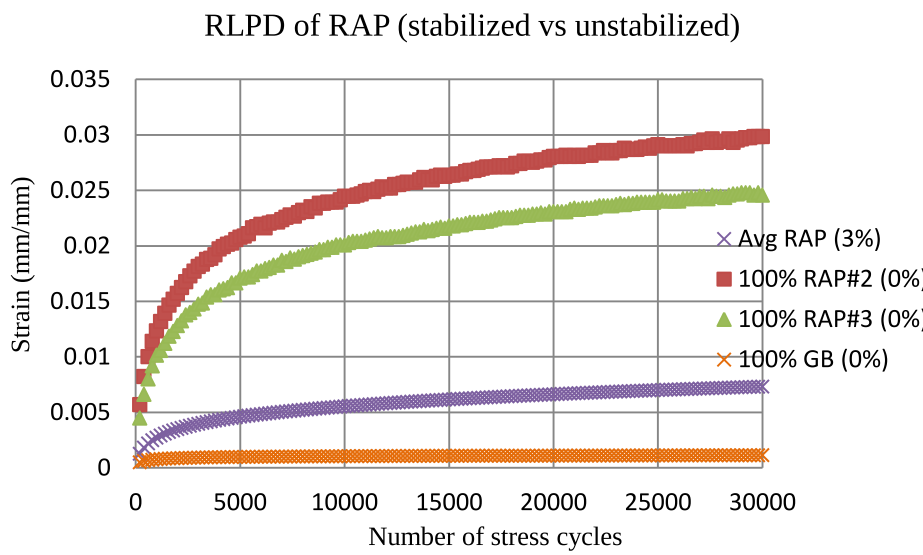


Figure 5.2.8: RLPD of stabilized versus unstabilized RAP

The density of specimens also affects permanent deformation. The specimens of stabilized material had higher density compared to the unstabilized, with density of 2067kg/m^3 and 2072kg/m^3 compared to 1930kg/m^3 and 1977kg/m^3, respectively. Therefore some of the improved resistance to permanent deformation may have been due to the higher density of foam stabilized specimens. An increase in density of 50kg/m^3 in unstabilized material resulted in a reduction of total strain at 30000 cycles by 17%. The stabilized specimens were an additional 90kg/m^3 more dense, and undergwent an additional 63% less strain at 30000 cycles. The benefits of stabilization on permanent deformation resistance may become clearer after examining other blends.

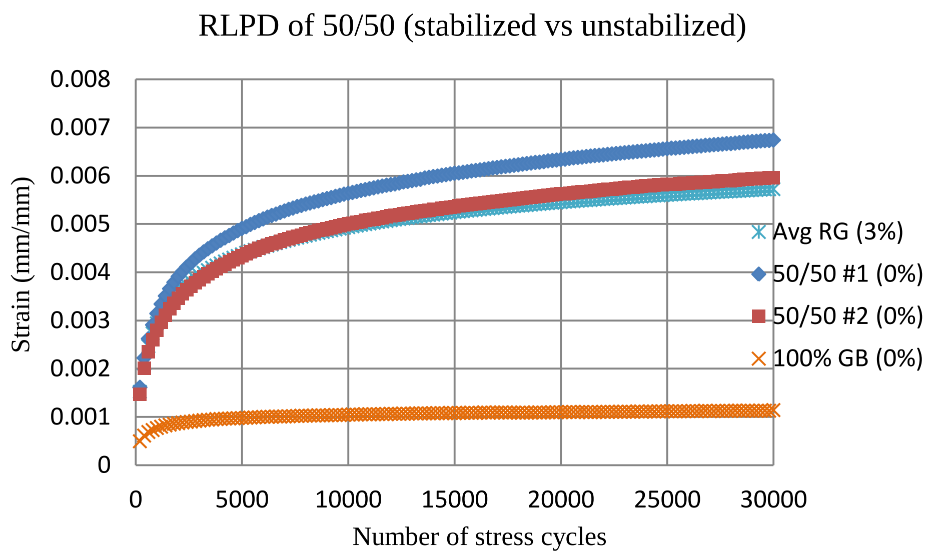


Figure 5.2.9: RLPD of stabilized versus unstabilized 50/50 blends

The differences between RLPD of stabilized and unstabilized 50/50 blends are displayed in Figure 5.2.9. With this material the results were much closer at 30000 cycles compared to RAP. Once again, densities fluctuated by around 100 kg/m^3 between stabilized and unstabilized. The actual values were 2155kg/m^3 and 2180kg/m^3 for unstabilized 50/50 #1 and #2 respectively. The densities of stabilized specimens were 2258kg/m^3 on average. Once again, this difference in strain could be due to the difference in density.

The results in Figure 5.2.9 suggest that foam stabilization of the 50/50 RAP/aggregate blends does not significantly improve repeated loading induced permanent deformation. This trend is generally inconsistent with the observations reported in the literature. It is likely owing to the high compaction effort as well as the fine fractionated RAP with high residual binder content. As a result, the specimen of heavily compacted specimens of RAP is almost like weak asphalt concrete.

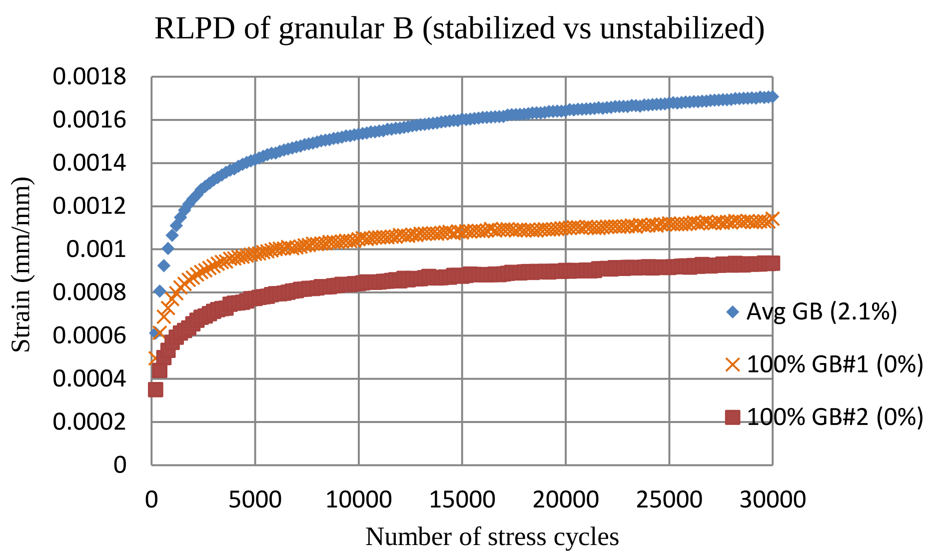


Figure 5.2.10: RLPD results of stabilized vs unstabilized GB

100% GB specimens with and without stabilization were compared for RLPD performance in Figure 5.2.10. It is clear that additional stabilization did not improve permanent deformation resistance of a well compacted natural Granular B material in this study. The unstabilized materials had lower density than the stabilized material by a maximum of 61 kg/m^3 however they still had less permanent deformation at all points throughout the loading. In all cases, the permanent deformation was small comparing with that of 50/50 blends or 100% RAP.

At high RAP content it appears that stabilization can improve resistance to permanent deformation, however as RAP content reduces, stabilization becomes detrimental to permanent deformation resistance, particularly for the 100% Granular B Type II aggregates used in this study. The interlocking structure of aggregates due to their high angularity, builds up enough friction without shearing to cause displacement. Therefore the added bitumen is compressing into void spaces at low bitumen contents, and possibly acting as a lubricant to the aggregates at high bitumen contents. In a 100% RAP specimen, aggregates are more rounded, less densely packed, and coated in an inactive layer of bitumen. They do not build up as high of frictional properties to resist shear. The added bitumen from foam stabilization may improve shear strength of the specimen to resist deformation at high RAP contents. This affect appears to have entirely disappeared at the point of 50% RAP and starts to become detrimental to deformation resistance as the content of natural aggregate increases.

5.2.1.5 RLPD of Soaked and Unsoaked Foam Stabilized Specimens

Foam stabilized specimens, both dry and soaked, were tested to determine the influence of soaking on RLPD. Soaked specimens were tested after having already been tested for triaxial resilient modulus in both the dry and soaked states. Specimens were tested for dry triaxial resilient modulus (TriM_r), then soaked for 24 hours and tested for soaked TriM_r and soaked RLPD immediately after. New specimens were used in RLPD tests on unsoaked specimens. Therefore, some small resistance to deformation may have been built up in the soaked specimens prior to RLPD testing.

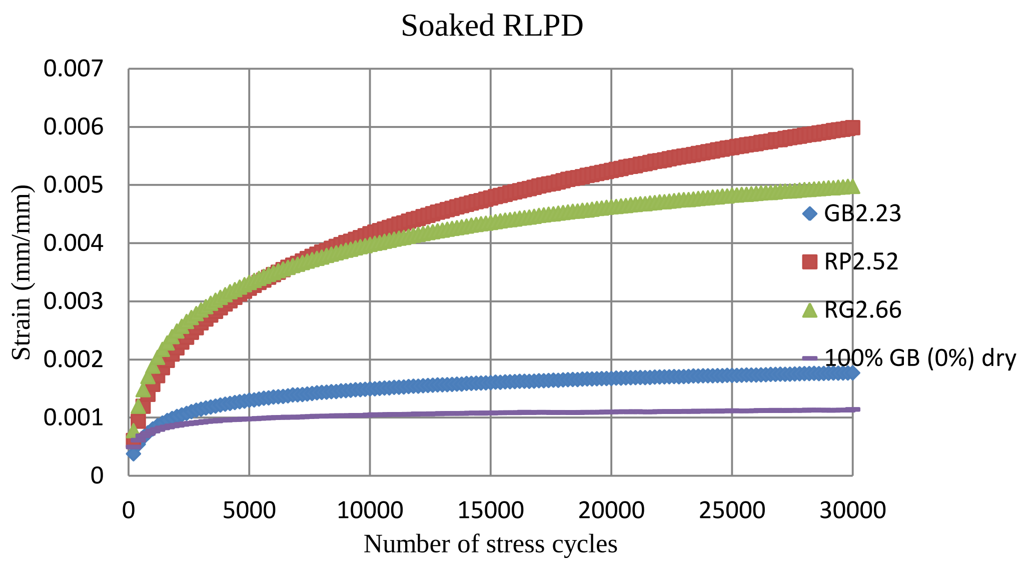


Figure 5.2.11: RLPD of soaked specimens

Soaked RLPD results are displayed in Figure 5.2.11. They are then compared to dry specimens under the same loading conditions in Figure 5.2.12. The same trends occurred in dry and soaked RLPD results. Most noticeably as RAP percentage increased, so did permanent deformation. Also the 50/50 blend developed initial deformation similar to 100% RAP however it built resistance to deformation quicker. When comparing the strain rates between dry and soaked specimens in Table 5.2.4, the wet specimens had higher strain rates at 30000 cycles. This means that they were building up deformation resistance slower than dry specimens. It seems like the non-continuously bound nature of stabilized material trapped some water in the specimen, similar to clay. That water would need to be squeezed out for effective stresses in the sample to increase. Over the course of the testing, water will squeeze out of pores slower than air, increasing the time required for building up deformation resistance.

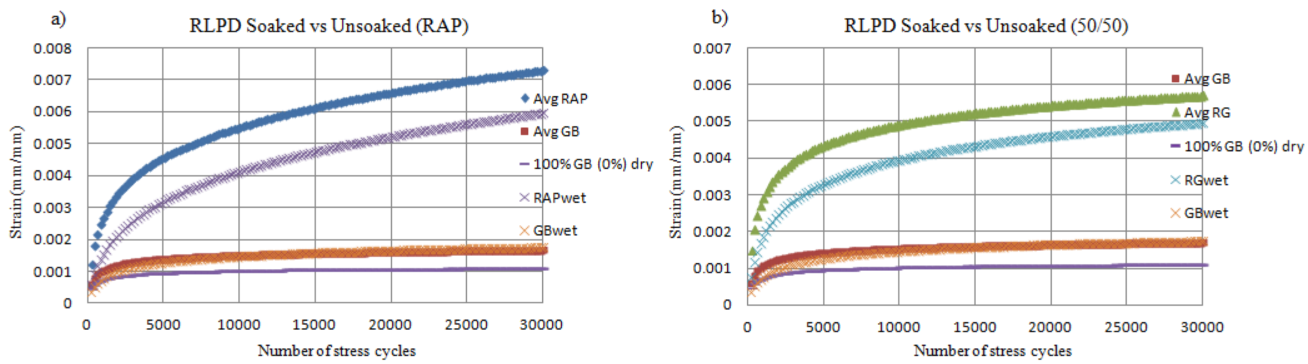


Figure 5.2.12: RLPD of soaked vs. dry specimens a) 100% RAP compared to 100% GB, b) 50/50 blend compared to 100% GB

It is expected that the difference in strain at 30000 cycles was affected by the prior loading of soaked specimens. RLPD of dry specimens was conducted immediately after conditioning however, soaked specimens were used for dry and then soaked TriM$_r$ before being loaded in RLPD. The permanent deformation of soaked specimens without being used for resilient modulus tests would be higher than that shown in Figure 5.2.12.

Table 5.2.4: Summary of dry and soaked RLPD results

Soaked and Average Dry Specimen Results					
Specimen	Density (kg/m^3)	Fines (%)	Bitumen (%)	Strain @ 30000	Strain rate @ 30000 (strain/cycle)
GB2.23 wet	2323	7	2.23	0.00177	8.68E-09
RP2.52 wet	2074	1.3	2.52	0.00599	6.85E-08
RG2.66 wet	2256	6.16	2.66	0.00498	2.99E-08
100% GB dry	2323	7	2	0.00171	6.08E-09
100% RAP dry	2072	1.3	3.3	0.00732	6.29E-08
50/50 dry	2258	6.16	3	0.00572	2.41E-08
100% GB #1	2308	7	0	0.00114	4.31E-09
100% GB #2	2262	7	0	0.00094	3.95E-09

5.2.2 Triaxial Resilient Modulus

Triaxial resilient modulus tests were performed on the final BSM specimens at different points in the loading history. The three soaked specimens were first tested for

$TriM_r$ in the dry state, then they were soaked for 24 hours and tested for soaked $TriM_r$ and soaked RLPD immediately after. Unstabilized tests were all conducted on new specimens with new material, not previously loaded.

5.2.2.1 Resilient Modulus of Unstabilized and Stabilized Aggregates

Resilient modulus tests on unstabilized aggregates were performed following AASHTO T307-99. Figure 5.2.13 shows the variation of $TriM_r$ with the bulk stress for different materials. Two specimens were tested for each blend and the results were close, less than 9% difference at a maximum. Triaxial resilient modulus and measurements of all specimens for each blend are shown in the appendix Section 8.2.

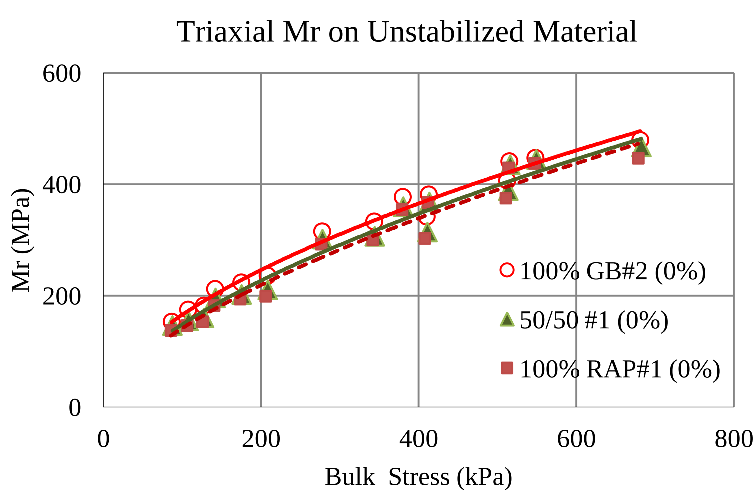


Figure 5.2.13: Triaxial resilient modulus comparison of unstabilized materials with different RAP contents

Table 5.2.5: Measurements of representative unstabilized triaxial resilient modulus specimens

Specimen	Aggregate	Bitumen content	Density (kg/m^3)	Height (mm)
100% GB#2	100% GB	0	2415	307
50/50 #1	50/50 blend	0	2192	313
100% RAP#1	100% RAP	0	2000	316

Figure 5.2.13 shows that RAP content did not significantly affect triaxial resilient modulus for these particular aggregates when all compacted to their maximum density of modified Proctor compaction. The 100% natural Granular B material was marginally stiffer than blends containing RAP at all stress levels. These results are representative of those found in the literature.

Many specimens were also tested for triaxial resilient modulus after foam stabilization. Figure 5.2.14 depicts the triaxial resilient modulus obtained at the highest stress level at σ_3 = 138kPa, σ_d = 288.6 kPa. These specimens were previously loaded in RLPD therefore the resilient modulus values are inflated. However there is a clear trend that increasing foamed bitumen content slightly reduces triaxial resilient modulus for all parent material blends. Referring to Figure 5.2.13, the resilient modulus of unstabilized specimens at the same stress level was approximately 400 MPa. One concludes that foam stabilization tends to increase M_r significantly for all parent materials.

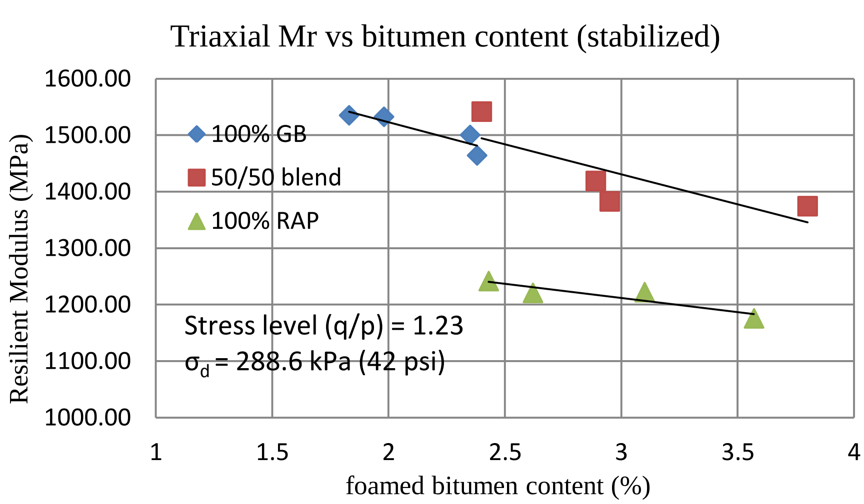


Figure 5.2.14: Triaxial resilient modulus of stabilized specimens at the highest stress level, displayed against foamed bitumen content

5.2.2.2 Stabilized Versus Unstabilized Material

The stabilized specimens used in this comparison were tested for $TriM_r$ before RLPD since the unstabilized materials were also not previously loaded. Figure 5.2.15 shows that bitumen stabilization more than doubled the resilient modulus at all levels of bulk stress for all of the aggregate blends tested.

Referring back to Section 5.2.1.4, it is strange that RLPD test results showed significantly more deformation in stabilized 100% Granular B material compared to the same material without stabilization, yet the resilient modulus showed that stabilized 100% Granular B was more than doubly stiff. The 50/50 RAP/aggregate blend also showed a large difference between the results of the two different tests where it was stiffer in the stabilized state yet it underwent greater permanent deformation. This means that a high resilient modulus of a specimen does not indicate strong permanent deformation resistance of a BSM.

It seems that the foam material in a BSM undergoes very small deformations under loading, which are not significant over a short loading period such as that which is encountered in a triaxial resilient modulus test. However, over long periods of testing encountered in RLPD, the small deformations continue to compound, resulting in large deformations.

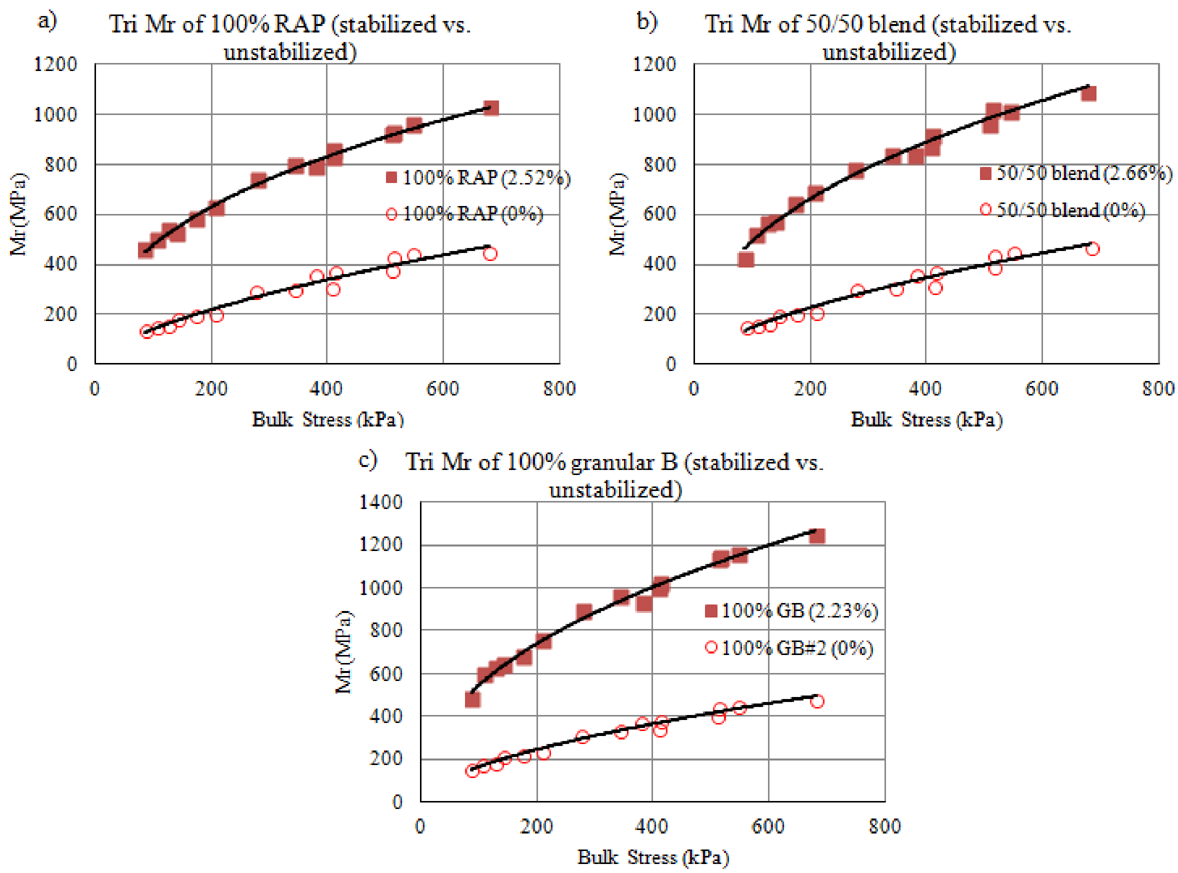


Figure 5.2.15: Stabilized versus unstabilized TriM$_r$ a) 100% RAP, b) 50/50 RAP/aggregate blend, c) 100% natural aggregates

5.2.2.3 Effect of soaking

Results of dry and soaked triaxial resilient modulus testing are displayed in Figure 5.2.16. The figure also shows TriM$_r$ of unstabilized materials for comparison. Soaking of specimens for 24 hours slightly lowered resilient modulus of the mix. The biggest difference was apparent in 100% Granular B, which lost 19% stiffness after soaking. The 100% RAP and 50/50 RAP/aggregate mixes lost 10% and 6% of their stiffness', respectively. With few data points it is hard to justify any difference between stiffness losses based on parent aggregate material, however the loss associated with 100%

Granular B was likely due to the low bitumen contents used. It was determined in the ITS results that bitumen content retains more strength while soaking compared to aggregate friction and weak chemical bonding of aggregates[7].

Figure 5.2.16 also illustrates the improved moisture resistance of stabilized materials over unstabilized materials. The unstabilized materials were tested immediately after compaction at optimum water content by modified proctor testing, therefore they were not soaked. Granular materials typically retain strength during soaking however they would lose some due to reduced frictional properties and pore pressures. The soaked stabilized materials exhibited significantly higher resilient modulus compared to unsoaked, unstabilized materials in all cases.

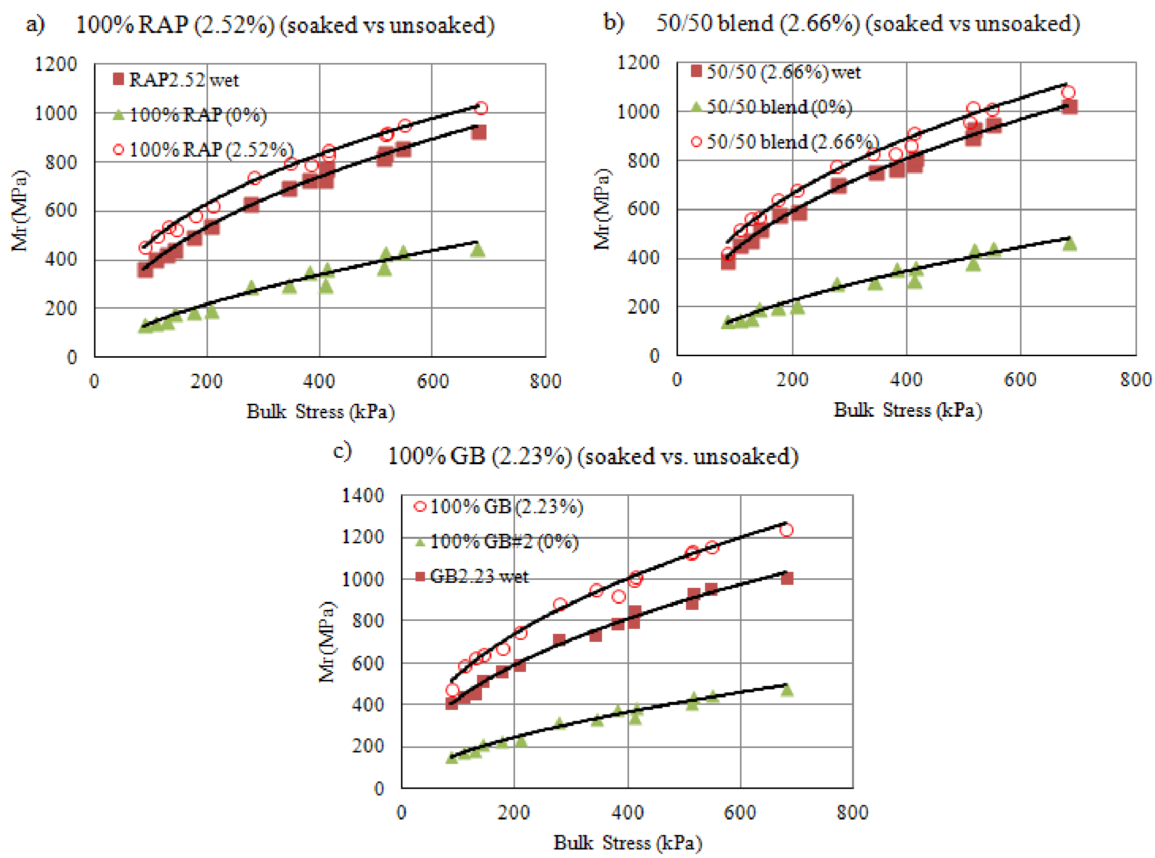


Figure 5.2.16: Dry vs. soaked triaxial resilient modulus by parent aggregate a) 100% RAP, b) 50/50 RAP/aggregate blend, c) 100% granular B

5.3 Unconfined Compression

All unconfined compression (UC) tests in this research program were conducted after triaxial resilient modulus and RLPD tests were completed on the same specimen. Since the strain levels in both the resilient modulus and the RLPD tests are small, it is expected that the unconfined compressive strength is not affected. Figure 5.3.1 displays the unconfined compression strength of all specimens tested in the dry state. These final specimens were created to resemble a target bitumen content however some variation occurred. Regardless, for this particular test the results were nearly identical across the acquired range of bitumen contents obtained for a particular aggregate blend. Figure 5.3.1 demonstrates that RAP content significantly reduced the ultimate failure load in unconfined compression tests.

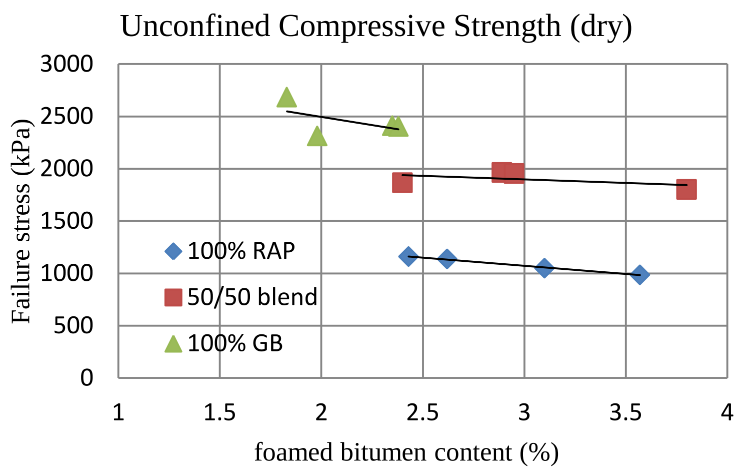


Figure 5.3.1: Unconfined compression strength of dry BSM specimens

Figure 5.3.2 displays the entire stress-strain curve of one (near optimum) specimen from each material blend. The graph starts at contact of the loading ram, passes through the peak stress (failure), and continues to load until the specimen appeared to begin crumbling. From this chart we get a better idea of the ductility of the specimens. The 100% Granular B specimen reached its peak stress and immediately dropped off whereas when 100% RAP was present in the mix, stresses dropped off slower after peaking. This slow drop of stress after the peak indicates a more ductile specimen or a less densely packed specimen. As we know, RAP is more resistant to compaction efforts therefore this is a realistic result.

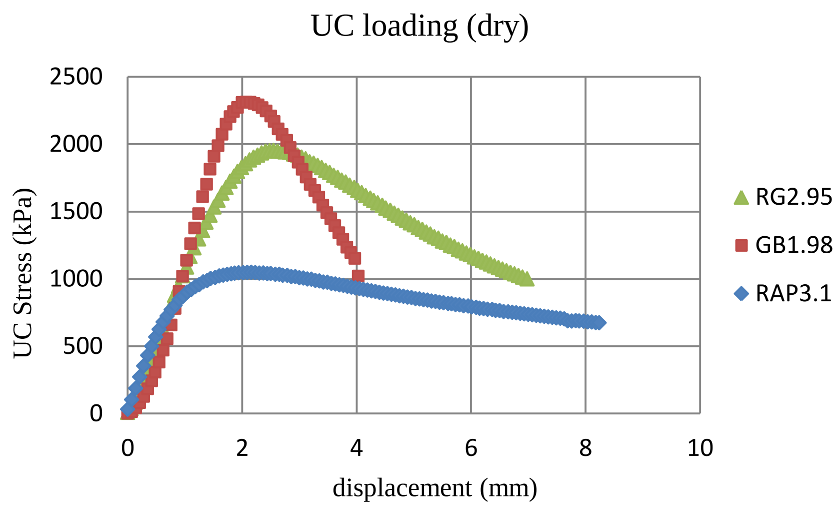


Figure 5.3.2: UC loading curves of stabilized blends

5.3.1 Effect of Soaking

The results of dry and soaked unconfined compression are displayed in Figure 5.3.3. Failure loads did not seem to be strongly affected by soaking however there was some small reduction in strength shown in part a) of the figure. The RAP specimen showed the largest change in peak stress however it was only 13% compared to approximately 7% in the two other blends.

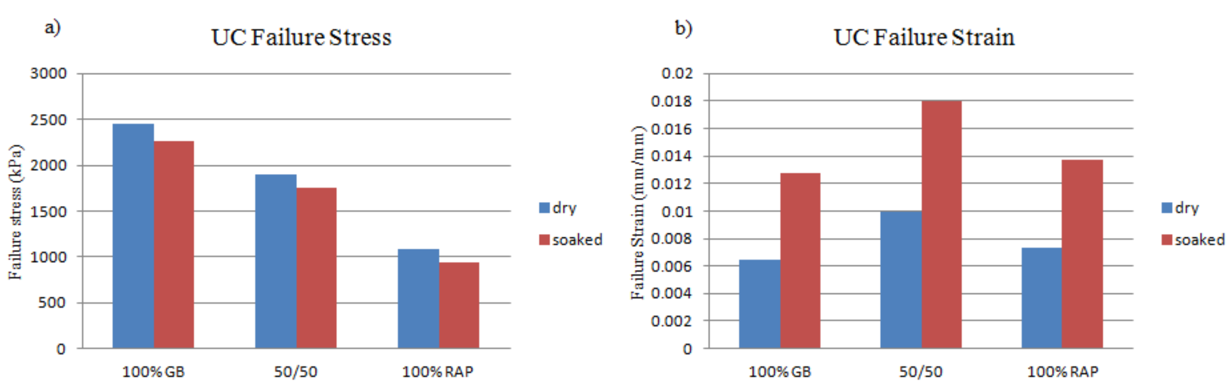


Figure 5.3.3: UC dry vs soaked; a) peak load, b) strain at failure

Changes in strength after soaking are due to reduced adhesive properties of bitumen and inter-particle contact in the mix. It is postulated that increased strain at failure is affected by the same factor. The reduced adhesion would cause the specimen to strain in order to increase contact area between particles in attempt to gain strength before shearing.

It is also possible that internal pore pressures could form in the specimen. Although the specimen was unconfined and readily allowed to drain in all directions, the nature of a BSM is non-continuously bound. Bitumen in the mix is water repellant however aggregates are porous. It is possible that after 24 hours of soaking, water could get into pores where it cannot readily escape. If this happens, pore pressures could build up and reduce effective stresses in the specimen, reducing the ultimate failure load.

It is also apparent in Figure 5.3.3 b) that the 50/50 RAP/aggregate blend had the highest strain at failure for both dry and soaked loading. This was not the case in RLPD testing where 100% RAP exhibited the highest strains. Since density increases and bitumen content decreases as RAP content reduces, the only other altered constituent which could have affected strain was the fines content. This is unlikely since fines were adjusted to resemble those of 100% Granular B (~7%), while maintaining optimum gradation. The frictional properties between rounded particles covered in inactive bitumen in contact with angular clean aggregates, must cause some increase in strain when loading to failure. This is only a postulation.

From the different tests carried out during this research, the effect of soaking appears to act differently in each situation. Soaked test results were compared to dry test results for ITS, IDT M_r, TriM_r, RLPD, and Unconfined compression. The largest effect of soaking was observed in ITS testing where, depending on mix constituents, the difference between soaked and dry specimens was as much as 50% reduced strength. Triaxial resilient modulus only resulted in a maximum of 20% reduction in strength after soaking. RLPD results did not accurately display the direct effect of soaking since soaked specimens had undergone prior loading and the dry ones did not. However, prior loading included two TriM$_r$ tests which are considered repeatable, and the resultant RLPD after soaking was reduced. From these results, it is estimated that RLPD was the least affected by soaking. Unconfined compression also showed small differences from soaking, with a maximum of 13% decrease in strength observed. The maximum difference observed in ITS testing was likely because tensile strength of the specimen is targeted in these tests whereas in all other tests it is not. Binder plays a strong role in developing tensile resistance, and soaking of the specimen results in a weakening of bitumen bonds, ultimately reducing tensile strengths. Therefore, of the tests completed, ITS was the strongest indicator of moisture susceptibility.

6 Summary, Conclusions and Recommendations

6.1 Summary and Conclusions

This study began with a mix design of foamed asphalt stabilized materials using foamed asphalt on specimens incorporating fine fractionated RAP (13.2mm), and crusher run Granular B (19mm). The mix design included aggregate blending and adjustment to densest gradation for RAP/aggregate blends. 100% Granular B and 100% RAP gradations were left as they came. Aggregate blends were tested for maximum dry density and optimum water content using modified Proctor compaction and a standard Proctor compaction test for comparison.

A different type of asphalt foamer was created during this study, in an attempt to reduce the costs associated with foamed bitumen research. The apparatus simply maintained bitumen at the required temperature and used air pressure to force bitumen and water into the expansion chamber on demand. Metering valves were used to control flow rates. It created foam of a sufficient quality however, fine adjustments of bitumen flow rate were irregular, which reduced repeatability in laboratory scale mixing. The mix design process was completed with varying bitumen contents.

The mix design process produced optimum foamed asphalt contents which did not resemble the expected. 100% Granular B had the lowest optimum binder content at 2%, the 50/50 blend optimum was 3.1%, and 100% RAP had the highest binder content at 3.3%. The reason for this difference is the method of optimum binder selection, as well as the type and strength of mixer which was used. The recommended mixer is a pugmill style mixer, which expands and aerates the soil for optimum dispersion of bitumen droplets. The planetary style mixer used for this research had been previously used, however this particular mixer had trouble with mixing at higher material densities. 100% Granular B had much higher density compared to RAP blends, which slowed down the rate of revolution and resulted in poorer mixing, especially at high bitumen contents.

Regardless of the actual optimum bitumen content of the materials, the determined AC values were distinct and precise. Final specimens were created at optimum bitumen content $\pm$0.5% by mass and testing was completed on these stabilized specimens, as well as equivalent unstabilized specimens. ITS and IDT M_r tests were conducted during the mix design stage, and triaxial M_r, RLPD, and unconfined compression in dry and soaked states on final specimens.

From the indirect tensile tests conducted during the mix design stage, the 50/50 blend did not produce any significant improvement over the 100% RAP. ITS_{wet} results were nearly identical, and ITS_{dry} only showed stronger values for the 50/50 blend at optimum bitumen content. Dry ITS displayed the 50/50 blend with an optimum mix around 150kPa, which was 25kPa stronger than 100% RAP, whereas 100% Granular B was around 90kPa stronger than 100% RAP. In the indirect tension resilient modulus results, RAP consistently performed better than the 50/50 blend however more so in the wet testing (+300 to 400MPa). 100% Granular B still outperformed all other results in stiffness.

Triaxial testing was completed next, starting with RLPD of unstabilized materials. These tests showed that unstabilized RAP was extremely susceptible to permanent deformation however it was slightly improved by the addition of Granular B and increased density. RLPD tests on stabilized materials illustrated that the addition of foamed bitumen content reduced permanent deformation at higher RAP contents. 100% Granular B did not show much change in deformation between the different bitumen contents.

Permanent deformation was also compared across aggregate blends with foam stabilization. Stabilized specimens containing RAP appeared to have similar deformation in the initial 5000 cycles, however the 50/50 blend built up deformation resistance quicker. When comparing stabilized to unstabilized RAP, the addition of stabilization improved resistance to permanent deformation when RAP content was high. However it increased susceptibility to permanent deformation when Granular B content was high. The 50/50 blend was nearly unaffected by stabilization. Soaked RLPD of stabilized material exhibited lower initial strain rates but higher final strain rates when compared to unsoaked RLPD. This indicates a reduced ability to build deformation resistance.

Triaxial resilient modulus testing was also completed on stabilized and unstabilized materials for effect of RAP content, foamed bitumen content, effect of stabilization, and effect of soaking. Unstabilized specimens showed marginal differences in $TriM_r$ for increased RAP content, with 100% GB being the stiffest. Around the optimum binder content, increasing foamed bitumen content tended to slightly decrease resilient modulus. When comparing unstabilized blends to stabilized blends, the stabilization more than doubled resilient modulus for all parent materials. The greatly improved M_r but reduced deformation resistance shows that $TriM_r$ is not a good indicator of permanent deformation resistance in stabilized materials. Soaking reduced resilient modulus by 10 to 20%, with 100% GB being the most greatly affected.

The final testing was concluded with a determination of ultimate failure strength in unconfined compression. All specimens had been previously loaded in RLPD and $TriM_r$. It was determined that increasing RAP content reduces ultimate failure stress. The 50/50 blend exhibited double the strength of 100% RAP, and 80% of the strength of the 100% GB material. Increasing RAP content also made a more ductile failure pattern similar to loose sand when compared to dense sand. Soaking reduced the ultimate failure load by 7 to 13%, but greatly increased strain at failure by 80 to 100%. It was postulated that the non-continuously bound nature of BSM-foam materials may trap some water in pore spaces during quick loading, thus reducing the effective strength.

Test results of soaked specimens were compared to those of dry specimens for ITS, IDT M_r, $TriM_r$, RLPD, and unconfined compressive strength. Of the completed tests, the largest effect of soaking was observed in ITS testing where the difference between soaked and dry specimens was as much as 50% reduced strength. Therefore, ITS is the strongest indicator of moisture susceptibility.

6.2 Recommendations

It is recommended that laboratory scale foamed bitumen be produced using a machine which keeps hot bitumen thoroughly mixed, and ideally flowing constantly at a regulated rate to ensure the accuracy of foaming water content, and foamed bitumen content of the specimens created.

Further research is encouraged with regards to experimentation with compaction levels. Material blends with high RAP content (>50%) demonstrated the greatest improvement in RLPD resistance over similar unstabilized blends. The optimum binder content for the 100% RAP blend was chosen particularly high in this research, and higher foamed bitumen content typically increases permanent deformation in the RAP materials tested. It is expected that a lower choice of optimum foamed bitumen content, and further increased compaction levels would reduce the permanent deformation under cyclic loading.

There was further research planned in this program with regards to the effects of compaction efforts, fines contents, and the type of RAP on M_r and RLPD. It is recommended that this further testing be completed since it was only omitted based on time constraints. More detailed information is given in Section 8.1 of the Appendix.

Investigations with regard to freeze-thaw cycles should be conducted. It is speculated that the non-continuously bound nature of foamed bitumen stabilized material can trap moisture for longer periods than in an unbound granular material. Although it is able to retain strength in a warm-wet state, there may be some concern with freeze-thaw damage from the formation of ice lenses in colder climates.

7 Bibliography

[1] J. J. Emery, "Practical Experience with Cold In-Place Asphalt Recycling and Foamed Asphalt Full Depth Reclamation," in *Proceedings of the Fifty-First Annual Conference of the Canadian Technical Asphalt Association*, Charlottetown, PEI, 2006.

[2] A. Copeland, "Reclaimed Asphalt Pavement in Asphalt Mixtures: State of the Practice," FHWA US Department of Transportation, McLean, VA, 2011.

[3] L. Csanyi, "Foamed asphalt for economical road construction," *Civil Engineering,* pp. 66-68, 1962.

[4] Wirtgen Group, Wirtgen Cold Recycling Technology, First Edition, Windhagen, Germany: Wirtgen GmbH, 2012.

[5] C. Luo, *High Performance Granular Base and Subbase Materials Incorporating Reclaimed Asphalt Concrete Pavement,* Hamilton: McMaster University, 2014.

[6] Asphalt Academy, "Bitumen Stabilized Materials - A Guideline for the Design and Construction of Bitumen Emulsion and Foamed Bitumen Stabilized Materials," Asphalt Academy, Pretoria, South Africa, 2009.

[7] P. Fu, "Micromechanics for Foamed Asphalt Stabilized Materials," University of California, Davis, 2009.

[8] W. Chesner, R. Collins, M. MacKay and J. Emery, "User Guidelines for Waste and Byproduct Materials in Pavement Construction," FHWA US Department of Transportation, Washington, D.C., 1998.

[9] P. Ruckel, S. Acott and R. Bowering, "Foamed-Asphalt Paving Mixtures: Preparation of Design Mixes and Treatment of Test Specimens," *Transportation Research Record,* vol. 911, pp. 88-95, 1982.

[10] P. Fu, D. Jones and J. Harvey, "Micromechanics of the Effects of Mixing Moisture on Foamed Asphalt Mix Properties," *Journal of Materials in Civil Engineering,* vol. 22, no. 10, pp. 985-995, 2010.

[11] S. Khosravifar, "Design and Mechanical Properties of Foamed Asphalt Stabilized Base Material," University of Maryland, College Park, Maryland, 2012.

[12] S. Senior, B. Gorman and S. Szoke, "Recycling Road Building Materials with Full Depth Reclamation in the Ontario Provincial Highway System," Materials Engineering and Research Office, Ministry of Transportaion, Downsview, Ontario, 2008.

[13] Ontario Ministry of Transportation, *Construction Specification for Compacting*, Downsview: Ontario Ministry of Transportation, 2013.

[14] Z. A. Khan, H. I. A.-A. Wahab, I. Asi and R. Ramadhan, "Comparative Study of Asphalt Concrete Laboratory Compaction Methods to Simulate Field Compaction," *Construction and Building Materials*, vol. 12, pp. 373-384, 1998.

[15] K. Kuna, G. Airey and N. Thom, "A Laboratory Mix Design Proceedure for Foamed Bitumen Mixtures," *Transportation Research Record: Journal of the Transportation Research Board*, vol. 1, pp. 1-10, 2014.

[16] Ontario Ministry of Transportation, *Construction Specification for Full Depth Reclamation with Expanded Asphalt Stabilization*, Downsview, Ontario: Ontario Ministry of Transportation, 2011.

[17] D. Chen, J. Bilyeu, T. Scullion, S. Nazarian and C. Chiu, "Failure Investigation of a Foamed-Asphalt Highway Project," *Journal of Infrastructure Systems*, vol. 12, no. 1, pp. 33-40, 2006.

[18] C.-T. Chiu and A. Lewis, "A Study on Properties of Foamed Asphalt Treated Mixes," *Journal of Testing and Evaluation*, vol. 34, no. 1, pp. 5-10, 2006.

[19] A. Hodgkinson and A. Visser, "The Role of Fillers and Cementitious Binders When Recycling With Foamed Bitumen or Bitumen Emulsion," in *8th Conference on Asphalt Pavements for Southern Africa*, Sun City, South Africa, 2004.

[20] W. Kim and J. F. Labuz, "Resilient Modulus and Strength of Base Course with Recycled Bituminous Material," Minnesota Department of Transportation, St. Paul, 2007.

[21] A. Nataatmadja, "Some Characteristics of Foamed Bitumen Mixes," *Transportation Research Record*, vol. 1767, pp. 120-125, 2001.

[22] K. Khweir, "Performance of Foamed Bitumen-Stabilised Mixtures," *Proceedings of the Institution of Civil Engineers: Transport*, vol. 2, no. 160, pp. 67-72, 2007.

[23] D. F. Stolle, P. Guo and J. J. Emery, "Mechanical Properties of Reclaimed Asphalt Pavement - natural aggregate blends for aggregate base," *Canadian Journal of Civil*

Engineering, vol. 41, pp. 493-499, 2014.

[24] P. Jitareekul and N. H. Thom, "Experimental Study on Deformation of Foamed Bitumen Bound Base Materials," in *Road Pavement Material Characterization and Rehabilitation: Selected Papers from the 2009 GeoHunan International Conference* , Changsha, Hunan, China, 2009.

[25] Ministry of Transportation, Ontario, "Method of Test for Sieve Analysis of Aggregates (LS-602 R23)," Ministry of Transportation, Toronto.

[26] ASTM International, *ASTM C117-13 Standard Test Method for Materials Finer than 75-μm (No. 200) Sieve in Mineral Aggregates by Washing,* West Conshohocken: ASTM International, 2013.

[27] ASTM International, *ASTM D4318-10e1 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils,* West Conshohocken: ASTM International, 2010.

[28] ASTM International, *ASTM D1557-12e1 Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort,* West Conshohocken: ASTM International, 2012.

[29] Nynas, *Safe Handling of Bitumen: A Practical Guide,* Nynas AB, 2012.

[30] Wirtgen Group, "Laboratory-scale foamed bitumen plant WLB 10," Wirtgen GmbH, Windhagen, Germany, 2001.

[31] Aecon Materials Engineering, "Expanded Asphalt Mix Design Report," Caledon, Ontario, 2014.

[32] ASTM International, *ASTM D7369-11 Standard Test Method for Determining the Resilient Modulus of Bituminous Mixtures by Indirect Tension Test,* West Conshohocken: ASTM International, 2011.

[33] ASTM International, *ASTM D6931-12 Standard Test Method for Indirect Tensile Strength of Bituminous Mixtures,* West Conshohocken: ASTM International, 2012.

[34] Austroads, "Review of Foamed Bitumen Stabilisation Mix Design Methods," Austroads Limited, Sydney, Australia, 2011.

[35] American Association of State Highway and Transportation Officials, *Determining the Resilient Modulus of Soils and Aggregate Materials,* Washington, D.C.: AASHTO, 2000.

[36] ASTM International, *ASTM D2166-13 Standard Test Method for Unconfined Compressive Strength of Cohesive Soil,* West Conshohocken, Pennsylvania: ASTM International, 2013.

[37] I. Sri Sunarjono, *The Influence of Foamed Bitumen Characteristics on Cold-Mix Asphalt Properties,* Nottingham, UK: The University of Nottingham, 2008.

[38] R. Chandra, A. Veeraragavan and K. J. Murali, "Evaluation of Mix Design Methods for Reclaimed Asphalt Pavement Mixes with Foamed Bitumen," in *2nd Conference of Transportation Research Group of India*, 2013.

[39] J.-S. Chen, M. Chang and K. Lin, "Influence of Coarse Aggregate Shape on the Strength of Asphalt Concrete Mixtures," *Journal of the Eastern Asia Society for Transportation Studies,* vol. 6, pp. 1062-1075, 2005.

[40] A. N, K. Mallesh and M. I. Anjum, "Influence of Particle Index of Coarse Aggregate and its Influences on Properties of Asphalt Concrete Mixtures," *International Journal of Research in Engineering and Technology,* vol. 3, no. 6, pp. 304-312, 2014.

[41] H. Y. Ahmed, M. D. Hashem, N. K. Rashwan and S. A. Abdalla, "Investigation of Aggregate Particles Shape on Characteristics of Hot Mix Asphalt," *Journal of Engineering Sciences,* vol. 42, no. 6, pp. 1349-1366, 2014.

8 Appendix

8.1 Extras from Experimental Studies

Proposed comparison testing program (not completed)

Comparison tests were planned in order to determine the influence of compaction level and fines content on the performance characteristics of a foamed BSM incorporating RAP. The IDT tests for resilient modulus and tensile strength ratio would be completed to determine stiffness and moisture susceptibility. RLPD and triaxial shear tests would be performed to find permanent deformation resistance and shear strength of the specimens. For simplicity, only two mixes would be tested for each experiment, including a control mix, and a comparison. The tests would be repeated for validity.

For the compaction experiment, a control mix composed of 50% A1 (≥7% fines) + 50% B4 (blend C2) could be prepared at 105% modified Proctor compaction. The comparison mix would be the same material (blend C2), but prepared at 100% standard Proctor compaction to distinctly display the effect of compaction effort on the specimen performance. This investigation is summarized in Table 8.1.1.

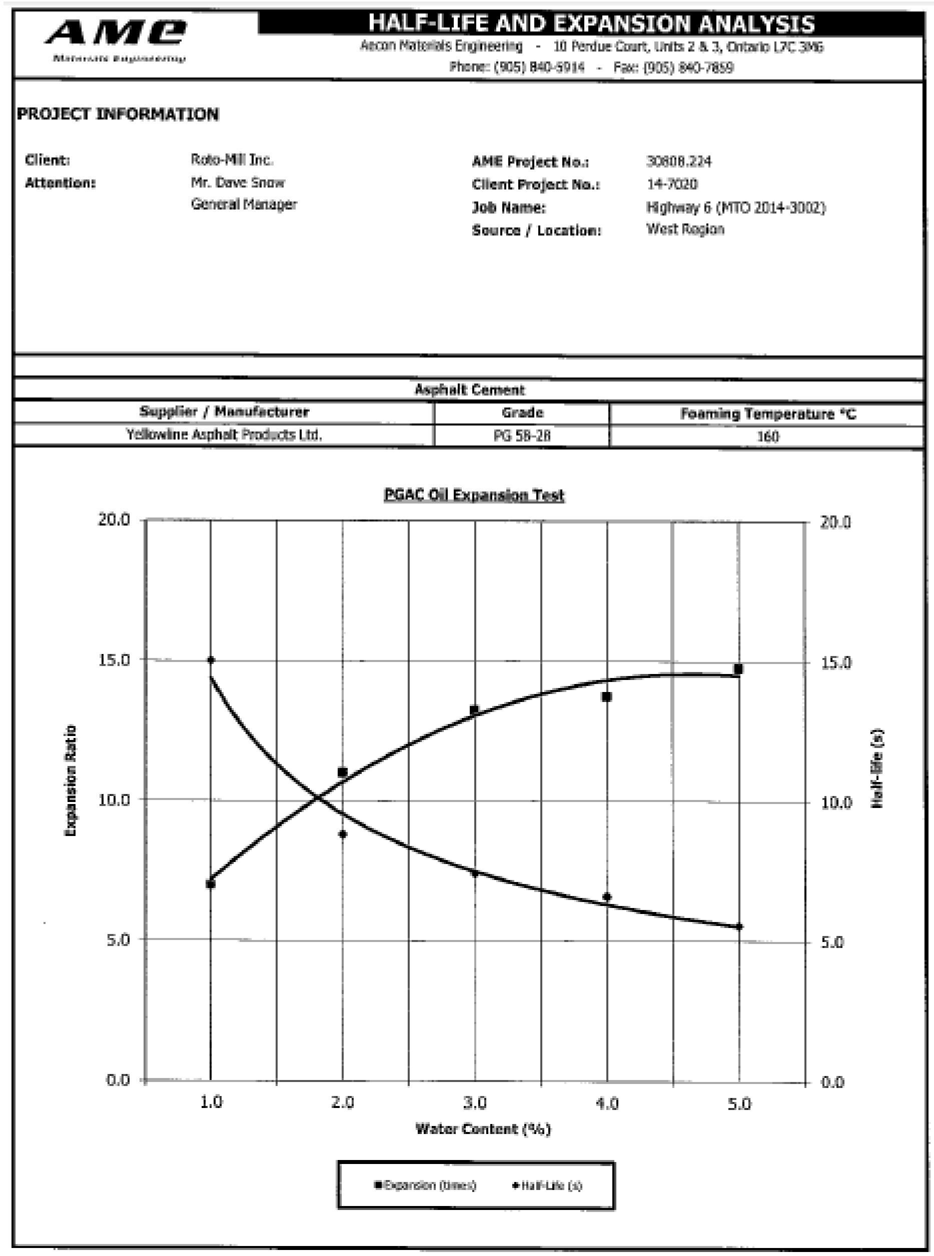
For the fines content experiment, a control mix composed of 50% A1 (≥7% fines) + 50% B4 (blend C2) would be prepared at 100% modified Proctor compaction. The comparison mix has reduced fines content with all other components held equal. It is explicitly composed of 50% A3 (Milton Granular B with 3% fines) + 50% B4 (blend C6), also prepared at 100% modified Proctor compaction to distinctly display the effect that fines content has on the specimen performance. This investigation is summarized in Table 8.1.2.

Table 8.1.1: Summary of tests for effect of compaction level

	IDT tests (M_r and TSR)				
Material	dry	soaked	RLPD	Tri shear	Comments
C2	xx	xx	xx	xx	105% mod. proctor compaction
C2	xx	xx	xx	xx	100% std. proctor compaction

Table 8.1.2: Summary of tests for effect of fines content

	IDT tests (M_r and TSR)				
Material	dry	soaked	RLPD	Tri shear	Comments
C2	xx	xx	xx	xx	Fines content FC1
C2	xx	xx	xx	xx	Fines content FC2



AME Materials Engineering

HALF-LIFE AND EXPANSION ANALYSIS

Aecon Materials Engineering - 10 Perdue Court, Units 2 & 3, Ontario L7C 3M6
Phone: (905) 840-5914 - Fax: (905) 840-7859

PROJECT INFORMATION

Client:	Roto-Mill Inc.	**AME Project No.:**	30808.224
Attention:	Mr. Dave Snow General Manager	**Client Project No.:**	14-7020
		Job Name:	Highway 6 (MTO 2014-3002)
		Source / Location:	West Region

Asphalt Cement		
Supplier / Manufacturer	**Grade**	**Foaming Temperature °C**
Yellowline Asphalt Products Ltd.	PG 58-28	160

Figure 8.1.1: AME half life and expansion analysis on Yellowline Asphalt Procducts' PG58-28 asphalt binder

AME
Materials Engineering

HALF-LIFE AND EXPANSION ANALYSIS

Aecon Materials Engineering - 10 Perdue Court, Units 2 & 3, Ontario L7C 3M6
Phone: (905) 840-5914 - Fax: (905) 840-7859

PROJECT INFORMATION

Client:	Roto-Mill Inc.	**AME Project No.:**	30808.174
Attention:	Mr. Dave Snow	**Client Project No.:**	(12-7002)
	General Manager	**Job Name:**	Highway 17 (MTO)
		Source / Location:	Northwestern Region

Asphalt Cement		
Supplier / Manufacturer	**Grade**	**Foaming Temperature °C**
McAsphalt Industries Limited	PG 58-28	165

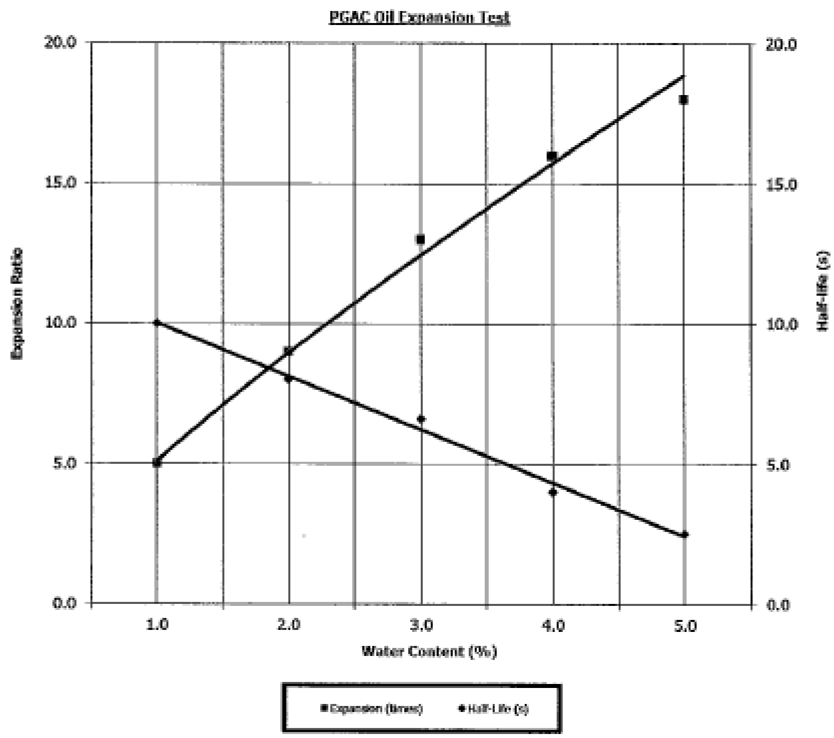


Figure 8.1.2: AME half life and expansion analysis on McAsphalt Industries Ltd.'s PG58-28 asphalt binder

8.2 Extras from Test Results and Analysis

Table 8.2.1: Final BSM specimens with 100% RAP as aggregate

RAP (optimum bitumen content: 3.3% >ended up 3.3+/-0.87)

Specimen	foam content	density target (kg/m3)	dry mass (kg)	diameter (mm)	height (mm)	volume (m^3)	density (kg/m^3)	aggregate comp.	Fines content (%)
RP2.52F1	2.52%	2047	3.641	150	99.79	0.00176	2065	100% RAP	1.30%
RP2.52F2	2.52%	2047	3.655	150	99.52	0.00176	2078	100% RAP	1.30%
RP2.52F3	2.52%	2047	3.654	150	99.45	0.00176	2079	100% RAP	1.30%
RP3.57F1	3.57%	2047	3.643	150	99.96	0.00177	2062	100% RAP	1.30%
RP3.57F2	3.57%	2047	3.656	150	99.66	0.00176	2076	100% RAP	1.30%
RP3.57F3	3.57%	2047	3.659	150	99.77	0.00176	2075	100% RAP	1.30%
RP2.62F1	2.62%	2047	3.635	150	99.91	0.00177	2059	100% RAP	1.30%
RP2.62F2	2.62%	2047	3.646	150	99.7	0.00176	2069	100% RAP	1.30%
RP2.62F3	2.62%	2047	3.649	150	99.57	0.00176	2074	100% RAP	1.30%
RP3.1F1	3.10%	2047	3.638	150	99.54	0.00176	2068	100% RAP	1.30%
RP3.1F2	3.10%	2047	3.657	150	99.64	0.00176	2077	100% RAP	1.30%
RP3.1F3	3.10%	2047	3.657	150	99.4	0.00176	2082	100% RAP	1.30%
RP2.43F1	2.43%	2047	3.644	150	99.78	0.00176	2067	100% RAP	1.30%
RP2.43F2	2.43%	2047	3.655	150	99.7	0.00176	2075	100% RAP	1.30%
RP2.43F3	2.43%	2047	3.649	150	99.75	0.00176	2070	100% RAP	1.30%

Table 8.2.2: Final BSM specimens with 100% granular B as aggregate

GB (optimum bitumen content: 2.0% > ended up 2+/-0.4)

Specimen	foam content	density target (kg/m3)	dry mass (kg)	diameter (mm)	height (mm)	volume (m^3)	density (kg/m^3)	aggregate comp.	Fines content (%)
GB1.98F1	1.98%	2310	4.096	150	99.87	0.00176	2321	100% GB	7%
GB1.98F2	1.98%	2310	4.102	150	99.7	0.00176	2328	100%GB	7%
GB1.98F3	1.98%	2310	4.106	150	99.72	0.00176	2330	100%GB	7%
GB1.83F1	1.83%	2310	4.089	150	100.22	0.00177	2309	100%GB	7%
GB1.83F2	1.83%	2310	4.1	150	99.89	0.00177	2323	100%GB	7%
GB1.83F3	1.83%	2310	4.102	150	99.98	0.00177	2322	100%GB	7%
GB2.35F1	2.35%	2310	4.093	150	99.74	0.00176	2322	100%GB	7%
GB2.35F2	2.35%	2310	4.103	150	99.8	0.00176	2326	100%GB	7%
GB2.35F3	2.35%	2310	4.103	150	99.78	0.00176	2327	100%GB	7%
GB2.38F1	2.38%	2310	4.096	150	99.95	0.00177	2319	100%GB	7%
GB2.38F2	2.38%	2310	4.1	150	99.83	0.00176	2324	100%GB	7%
GB2.38F3	2.38%	2310	4.1	150	99.85	0.00176	2324	100%GB	7%

GB2.23F1	2.23%	2310	4.091	150	99.79	0.00176	2320	100%GB	7%
GB2.23F2	2.23%	2310	4.099	150	99.88	0.00177	2322	100%GB	7%
GB2.23F3	2.23%	2310	4.105	150	99.83	0.00176	2327	100%GB	7%

Table 8.2.3: Final BSM Specimens with 50/50 RAP/GB as aggregate

50/50 RAP/GB (optimum bitumen content: 3.0% >ended up +/-0.8)

Specimen	foam content	density target (kg/m3)	dry mass (kg)	diameter (mm)	height (mm)	volume (m^3)	density (kg/m^3)	aggregate comp.	Fines content (%)
RG2.4F1	2.40%	2254	4.526	150	113.37	0.00200	2259	50/50	6.16%
RG2.4F2	2.40%	2254	4.53	150	113.43	0.00200	2260	50/50	6.16%
RG2.4F3	2.40%	2254	4.531	150	113.4	0.00200	2261	50/50	6.16%
RG3.8F1	3.80%	2254	4.001	150	100.62	0.00178	2250	50/50	6.16%
RG3.8F2	3.80%	2254	4.012	150	100.41	0.00177	2261	50/50	6.16%
RG3.8F3	3.80%	2254	4.017	150	100.17	0.00177	2269	50/50	6.16%
RG2.66F1	2.66%	2254	4.004	150	100.66	0.00178	2251	50/50	6.16%
RG2.66F2	2.66%	2254	4.013	150	100.63	0.00178	2257	50/50	6.16%
RG2.66F3	2.66%	2254	4.018	150	100.63	0.00178	2259	50/50	6.16%
RG2.89F1	2.89%	2254	4.003	150	100.61	0.00178	2252	50/50	6.16%
RG2.89F2	2.89%	2254	4.007	150	100.72	0.00178	2251	50/50	6.16%
RG2.89F3	2.89%	2254	4.019	150	100.27	0.00177	2268	50/50	6.16%
RG2.95F1	2.95%	2254	3.998	150	100.58	0.00178	2249	50/50	6.16%
RG2.95F2	2.95%	2254	4.005	150	100.43	0.00177	2257	50/50	6.16%
RG2.95F3	2.95%	2254	4.014	150	100.32	0.00177	2264	50/50	6.16%

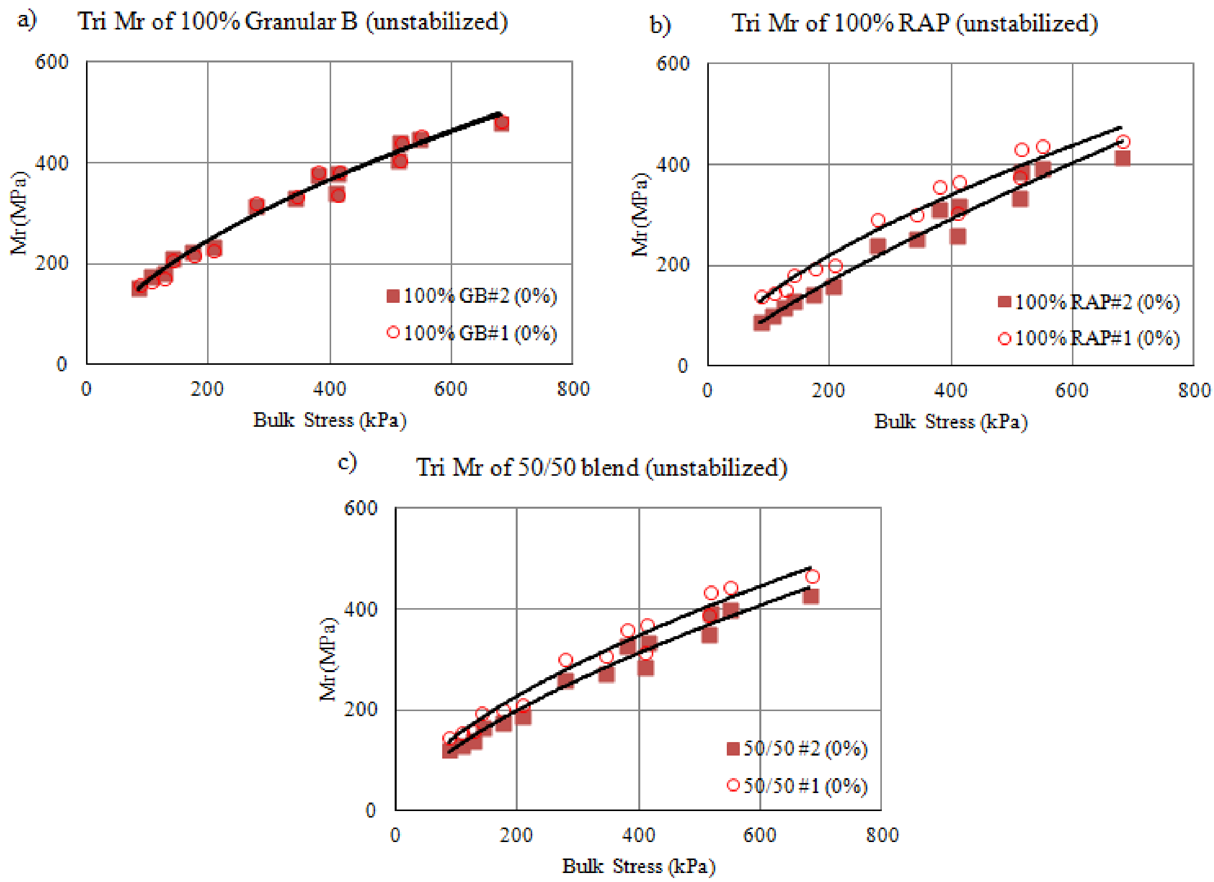


Figure 8.2.1: Triaxial resilient modulus of unstabilized materials by material blend (all tests) a) 100% natural aggregates, b) 100% RAP, c) 50/50 RAP/Aggregate blend

Table 8.2.4: Measurements of all unstabilized specimens tested in triaxial resilient modulus

Specimen	Aggregate	bitumen content	density (kg/m^3)	height (mm)
100% GB#1	100% GB	0	2306	310
100% GB#2	100% GB	0	2415	307
50/50 #1	50/50	0	2192	313
50/50 #2	50/50	0	2225	310
100% RAP#1	100% RAP	0	2000	316
100% RAP#2	100% RAP	0	1977	315