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Full Length Article

The optimal use of crumb rubber in hot-mix asphalt by dry process: A laboratory investigation using Marshall mix design

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ABSTRACT

This paper aims to use the Marshall method to design and evaluate crumb-rubber modified (CRM) and conventional dense-graded (DG) hot mix asphalt (HMA), as there were limited previous experimental investigations that compared their properties by varying crumb rubber (CR) sizes, CR contents and aggregate types. Two types of crushed aggregate – diorite and granite – were mixed with 5.0% to 7.0% bitumen contents and 0%, 1%, 2% and 4% CR contents sized at 0.71 mm, 2 mm and 2.36 mm by dry process. The laboratory test results showed that the optimum bitumen content (OBC) and CR content that commonly satisfied international standards of CRM-DG-HMA by dry process and conventional DG-HMA were 5.5% and 1% respectively. CR content significantly contributed to the swelling of the CRM-DG-HMA, followed by the aggregate type and CR size. CRM-DG-HMA using granite aggregate was slightly more prone to swelling than that with diorite aggregate. Diorite-based CRM-DG-HMA with 2 mm CR size was more durable than conventional DG-HMA, granite-based CRM-DG-HMA and diorite-based CRM-DG-HMA with 0.71 mm CR size, with the last two being less durable than conventional DG-HMA. Granite-based CRM-DG-HMA was more stable and stiffer than conventional DG-HMA, while diorite-based CRM-DG-HMA was less stable but stiffer than conventional DG-HMA. Moreover, granite-based CRM-DG-HMA was more stable and stiffer than diorite-based CRM-DG-HMA with 0.71 mm CR size, with the latter being more stable and stiffer than diorite-based CRM-DG-HMA with 2 mm CR size. The findings from this paper have shown that partial replacement of mineral aggregate with CR in DG-HMA is a sustainable option, which could yield comparable or improved properties over conventional DG-HMA, provided that the CR size, CR content and aggregate type have been determined and selected in the mix design.

1. Introduction

1.1. Crumb-rubber modified asphalt

The utilisation of crumb rubber (CR) in pavement engineering, especially by mixing CR with asphalt to create crumb-rubber modified (CRM) asphalt, has been practised in Sweden and the United States since the 1970s [1]. The addition of CR has been proven to endow asphalt with improved properties and benefits, as presented in Table 1, which also shows that CRM asphalt is a developing area of research and practical application [2–10]. For instance, traffic noise is a crucial factor that contributes significantly to urban noise pollution that can bring about several detrimental effects on humans and the environment [11]. As CRM asphalt also improves the skid resistance of road surfaces, it could be used to enhance road surface condition and hence improve road safety. Moreover, the lifespan of CRM asphalt is expcted to be higher as the rate of accumulated permanent deformations are reduced.

CRM asphalt can be prepared by three processes: dry, wet, and terminal blend. In the dry process, CR acts as a replacement aggregate, essentially functioning as an elastic aggregate [5,6]. The CR and hot aggregate are blended before mixing with bitumen [5]. This process is limited to HMA applications [13]. In the wet process, fine CR is mixed with bitumen at an elevated temperature for a short period of time before mixing with hot aggregate, essentially functioning as a major binder modifier [5] to improve the properties of the bitumen [14]. For the terminal blend, fine-mesh CR from 100% tyre rubber is dissolved/digested with bitumen over an extended period of time [15]. The prominent difference between the wet process and terminal blend is that

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Table 1
Improvements of rubberised asphalt.

Increased resistance to	Refs.	Decreased	Refs.
Rutting	[4,8]	Plastic/permanent deformation	[2,7,12]
Skidding	[5,8, 10]	Ageing effect	[8,12]
Fracture	[4]	Fatigue cracking	[6,10,8, 12]
Oxidation	[8]	Thermal cracking	[6]
Increased		Reflection cracking	[8,2]
Tensile strength	[6]	Traffic noise	[2,4,10, 11]
Resilience	[6,10]	Temperature sensitivity/ susceptibility	[10,12]
Ductility	[10]	Stiffness	[10]
Durability	[5]	Raw material usage	[12]
Elasticity	[12]	Energy consumption	[12]
Flexibility	[10]	Maintenance cost	[2,8,12]
Life expectancy	[2,10]		
Road safety condition	[8,12]		

the former is a 2-phase (non-homogeneous) mixture (consisting of the solid CR and liquid bitumen) while the latter is a 1-phase (homogenous) mixture (CR is dissolved/digested by the bitumen). Between the wet process and terminal blend, CRM asphalt by wet process exceeds CRM by terminal blend in terms of performance and rutting resistance [15]. The terminal blend uses natural rubber for consistency reason, which is not as environmentally beneficial as when using waste rubber in the case of dry and wet processes [16].

Although various literature and research have explicitly expressed that CRM asphalt by dry process is less popular and inferior to that by wet process [9,17], the dry process has several benefits that outweigh the wet process. As CRM asphalt by dry process is similar to the conventional method of asphalt production, there is no need for mechanical modification to the existing manufacturing system, thus making it logistically easier and less costly to implement than the wet process [6,8, 18-20]. Moreover, more bitumen is required to produce CRM binder by wet process, and hence additional binder cost is incurred [13,21,22]. To enhance the bitumen properties, the compatibility of rubber with binder is a critical factor for the wet process. Previously it was considered that such compatibility was less significant in the dry process [13], but it has been proven that it is also a critical factor for fine and ultra-fine rubber particles. CRM asphalt by wet process should not be used in DG-HMA as the rubber particles can create compaction problem due to space/void [15]. CRM asphalt by the dry process does not require special storage and transportation requirements as that by wet process [6]. Ref. [23] further stated that the dry process has higher temperature stability than the wet process.

Refs. [8,19] and [24] highlighted that the dry process exhibits greater potential for recycling end-of-life tyres than the wet process. The dry process consumes CR of 2 to 4 times as much as the wet process [8]. A typical dry process uses about 1% to 5% of rubber by mass of aggregate, while a typical wet process uses about 10% to 30% of rubber by mass of bitumen [25].

1.2. Rubber-bitumen interaction

The dry process substantially contributes to a common but undesired phenomenon associated with CRM asphalt – rubber swelling. CR swelling in asphalt not only alters the proportion of components in the asphalt but also alters the structure of the asphalt; this ultimately alters the properties of the asphalt [26]. The rate of swelling is controlled by the rubber size, shape and content used.

The rubber size used in the dry process (0.4 mm to 10 mm) is generally larger than that used in the wet process (0.075 mm to 1.2 mm) [18,19,27]. The reactivity between rubber and bitumen is inversely proportional to rubber size, i.e., reactivity increases with decreasing rubber size [19]. Rubber size smaller than 1 mm in diameter expedites rubber-bitumen interaction, thus accelerating swelling [19] and is difficult to be separated from the bitumen once added together [26]. The investigation by Ref. [28] showed that rubber size between 0.1 mm and 0.2 mm used in the dry process improved rubber-bitumen interaction. Ref. [16] stated that asphalt prepared by the dry process using rubber size less than 0.6 mm had a similar structural performance to that of asphalt prepared by the wet process. Rubber particles with an irregular shape and high surface area are more likely to interact with bitumen at elevated temperatures than cubical-shaped rubber particles with a lower surface area; the latter is more suited to be used in the dry process [5]. Ref. [5] found that the mechanical properties of dry process rubberised HMA are sensitive to the change in CR content and Ref. [13] stated that the amount of fine rubber introduced to the mix determines the degree of bitumen modification.

Rubber-bitumen interaction is less pronounced in the dry process. It was initially assumed that the interaction between rubber and bitumen in the dry process was insignificant/negligible [5], but this assumption is refuted by other investigations concluding that there are some rubber-bitumen interactions during mixing, transport and placement that result in minor rubber swelling and slight binder modification [18, 21,29,30]. Despite the findings, quantitative physiochemical investigation on rubber-bitumen interaction by dry process remains scarce. However, it is deduced that when more time is given to rubber-bitumen interaction, an equilibrium can be achieved, and at this stage, the rubber is saturated with the absorbed binder and swelling stops [19]. The time taken to achieve the equilibrium depends on the type, size and content of rubber used. Rubber swelling during preparation and post-compaction is often the primary reason why the wet process is favoured over the dry process [14,19,23,24,27,31,32]. Although the swelling behaviour in the dry process is not completely studied and presented, it can be established that the larger size and greater quantity of rubber used in the dry process make swelling more prominent. Rubber swelling can be minimised by manipulating rubber size, shape and content, as well as the mixing time of rubber with bitumen. Apart from swelling, rubber also undergoes rebounding [6,23]. As illustrated by Ref. [33], the rebound effect after compaction showed a uniform dilation in all directions of asphalt samples with CR post-compaction. Swelling and rebounding instigate volumetric variation/instability [6,23,24]. Ref. [6] found that if rubber content is higher than 2% or rubber size is larger than 2.36 mm, the specimen surface could not maintain the moulded shape after compaction as the rubber particles recover their original uncompacted dimensions.

1.3. Use of crumb rubber in dense-graded hot mix asphalt

The dry process considers CR a suitable aggregate replacement for DG-HMA [13]. CRM-DG-HMA by dry process exhibited slightly better fatigue performance and rutting resistance than the wet process and conventional asphalt for the same material and volumetric condition [34]. CRM-DG-HMA is also less vulnerable to moisture damage [35]. Ref. [14] found that for the nominal maximum particle size of 12 mm for CRM-DG, the optimum CR content for DG was 1.5%. The Marshall stability (S) was higher, but the Marshall flow (F) was lower for CRM-DG and the rutting resistance was also better, and the rut depth was less than those for conventional HMA. Ref. [36] found that the S was higher for the nominal maximum particle size of 14 mm for DG, and the F was lesser. The resistance to permanent deformation for DG depends on bitumen, filler and fine mortar stiffness. Results from Ref. [37] concluded that even for conventional HMA, for the same nominal maximum particle size, DG had better stability, stiffness and rutting resistance.

1.4. Aggregate-bitumen stripping and crumb rubber-bitumen stripping

The types of aggregate used with bitumen in asphalt impact stripping, and the aggregate properties are the main contributor to the moisture susceptibility of asphalt [38]. The aggregate can be divided into three types, based on the silicon dioxide (SiO_2) content: alkaline aggregate (SiO_2 is less than 52%), neutral aggregate (SiO_2 is between 52% and 65%) and acidic aggregate (SiO_2 is more than 65%) [39]. It is observed that alkaline aggregate has a higher resistance to asphalt stripping than acidic aggregate [40]. This is because acidic (igneous) rocks tend to be negatively charged, and such aggregate with a considerable amount of albite, k-feldspar and quartz in the form of large crystals do not bind well with bitumen [41–43]. The effects of physical and chemical characteristics of aggregate on bonding with bitumen are discussed by Ref. [40]. Ref. [44], in their investigation, measured the surface free energy (SFE) of granite, limestone and bitumen to determine the adhesion bond strength between the two types of aggregate and bitumen. They deduced that siliceous aggregate, such as granite, is an acidic aggregate with more polar components and fewer non-polar components.

In comparison, calcareous aggregate such as limestone is an alkaline aggregate with more non-polar and less polar components. On the other hand, bitumen is acidic and possesses more non-polar components than polar ones. The adhesion between aggregate and bitumen is formed by covalent (non-polar) bond. Siliceous aggregate (having few non-polar components) when interact with bitumen (having higher non-polar components) form a weaker covalent bond than that between calcareous aggregate (having numerous non-polar components) with bitumen (having numerous non-polar components too). The non-polar components allow the covalent bonds to remain stable in the presence of water.

There is a gap in the quantitative physiochemical investigation of rubberised HMA that determines the SFE of aggregate, rubber and bitumen by dry process; thus, the adhesion and debonding strengths between aggregate-bitumen and rubber-bitumen are relatively unknown. However, physiochemical theories could be used to explain the bonding mechanism between rubber and bitumen. Ref. [45] exhibited several electron microscopy images showing that the smooth surface of rubber is less conducive for bitumen coating, thus inferior mechanical adhesion would form between rubber and bitumen at the interface. The elasticity of rubber, which allows it to deform upon loading and recover upon unloading, can damage its bitumen coating [45]. Therefore, the addition of rubber changes the structure of asphalt and the contact state between its materials, causing the CRM asphalt to be more readily loosened than conventional asphalt [45]. Past and present literature reiterated that severe rubber-bitumen stripping is attributed to the weak adhesion bond between rubber and bitumen [9,46].

1.5. Use of hydrated lime in dense-graded hot mix asphalt

The concerns related to bitumen stripping from aggregate and/or CR can be addressed by adding hydrated lime (HL) ($Ca(OH)_2$) to the asphalt mixture. Initially, HL was introduced in the 1970s to asphalt as an asphalt modifier to resist moisture damage, but over time, it has proven to be a powerful anti-oxidant and an effective filler [47]. When added to aggregate, HL is regarded as an anti-stripping agent, but when added to bitumen, it is regarded as an anti-oxidant [48]. It has been found to be a suitable addition for CRM-DG-HMA [47]. Ref. [38] identified several key advantages of adding HL to HMA. The filler effect of HL in HMA reduces its potential for deformation at high temperatures, especially during its early life when it is most susceptible to rutting. HL stiffens the bitumen film and reinforces it, thus making HMA less sensitive to moisture damage as the aggregate-bitumen bond is improved; this reduces rutting. For the same filler mass, HL imparts a greater stiffening effect than cement or stone dust [49]. HL also reduces the rate of bitumen oxidation as bitumen ageing hardens the bitumen excessively, thus making HMA more susceptible to fatigue cracking and low-temperature cracking. The use of HL increases the indirect tensile strength of HMA, which is attributed to an increased adhesion bond between aggregate and bitumen [50]. Ref. [51] showed that to obtain a tensile strength ratio of approximately 85%, either 2% HL content by dry replacement or 2.5% HL content by wet replacement was required.

The HL content used in HMA is generally between 1% to 3% by mass, with a typically agreed range of around 1% to 2% [49,51-55]. Excessive HL often decreases P_a and S [54,56].

1.6. Asphalt mix design for dense-graded hot mix asphalt

When designing conventional DG-HMA for the asphaltic wearing course, Marshall mix design is the principal design method [57–59] and is endorsed by Asphalt Institute. It is still considered the standard method of asphalt mixture design for practical engineering applications [60]. The asphalt samples are designed based on empirical laboratory procedures in Marshall mix design. It enables rapid testing with minimal effort, is easy to conduct, and requires inexpensive equipment [60]. In tropical countries, it is the most commonly used design method [61]. Stability and permanent deformation (i.e., S and F) provide the performance prediction measure for this design method.

The Marshall method has to be used with caution and consideration for the design of CRM-HMA by dry process. Ref. [1] discussed the critical mix design considerations involved. The authors highlighted that the selection criteria for OBC for CRM-HMA are different from those for conventional HMA. The primary criteria for the design of conventional HMA are sample bulk density (G_{mb}), percentage voids in asphalt (P_a), S and F. However, the S of CRM-HMA could be lower than that of conventional HMA, while the F for CRM asphalt could be greater than that of conventional HMA. Therefore, the S and F values obtained from the Marshall method could only be regarded as secondary criteria for the design of CRM-HMA. Different criteria should be considered for the design of CRM-HMA. The authors underlined that the critical factor for successful CRM-HMA, based on experiments and experiences, is for the asphalt mixture to have low P_a.

The biggest scepticism concerning the use of CRM-HMA by dry process stems from two conflicting outcomes from past investigations. While it is expected that S of CRM-HMA underperformed that of conventional HMA [1,2,4,62-64], there are those whereby S of CRM-HMA outperformed that of conventional HMA [8,14,65-68]. Similarly, there are contradictions in the outcomes for F. While it is expected that F of CRM-HMA to be higher than that of conventional HMA [8,64-66,68], there are those whereby F of CRM-HMA are lower than that of conventional HMA [2,4,14,62]. The identifiable variables that could contribute to such conflicting outcomes are aggregate type (e.g., granite and limestone), aggregate gradation (e.g., gravel-to-sand (G:S) mass ratio ranging from 0.95 to 3) and CR size (e.g., less than 1 mm, between 1.18 mm to 6.3 mm and greater than 9.5 mm). Two common findings observed amongst those past investigations were that the optimum CR content lay between 1% and 2% by mass of asphalt mixture, and samples of small-sized CR (less than 1 mm) had high S.

1.7. Contribution to knowledge and scope of work

While considerable past research has been conducted in integrating rubber with bitumen using the wet process, there has been limited research on the performance of CRM-DG-HMA against conventional DG-HMA by the dry process. There are insufficient experimental investigations that examined the impact of CR size, CR content, aggregate type and bitumen content on the volumetric and Marshall properties of CRM-DG-HMA by dry process compared to those of conventional DG-HMA. Therefore, to address this knowledge gap, the objective of this paper is to use the Marshall method for the design of CRM-DG-HMA, in which two types of crushed aggregate – diorite and granite, were mixed with 2% HL, 5.0% to 7.0% bitumen contents and 0%, 1%, 2% and 4% CR contents sized 0.71 mm, 2 mm and 2.36 mm by dry process. The results in the expressions of sample height (H), G_{mb}, P_a, percentage voids in mineral aggregate (VMA), percentage voids filled with bitumen (VFB), percentage of absorbed bitumen (P_{ba}), S and F were recorded, calculated, interpreted and compared.

2. Materials

2.1. Aggregate

Two types of aggregate were selected for this investigation – diorite and granite. Both aggregates are intrusive igneous rocks. Table 2 shows their silica and quartz contents, which are used to classify the aggregates. The specific gravities (SG), Los Angeles abrasion values (LAAV), aggregate crushing values (ACV) and aggregate impact values (AIV) of the aggregates were experimentally measured and are shown in Table 3. The physical and mechanical quality requirements of the aggregate used in the investigation complied with Brunei Darussalam's specification – GS1: 1998 Flexible Pavement [69]. The average gradation (W/14 [avg]) in the grading envelope from Ref. [69], as presented in Fig. 1, was selected to prepare the asphalt samples. The selected gradation (i.e., W/14 [avg]) has a low gravel content, i.e., more sand than gravel in the mixture. The G:S mass ratio for this gradation is 48:52 or 0.92.

2.2. Bitumen

Bitumen graded 60/70 penetration was used as the asphalt binder. It was procured from a major local supplier in Brunei Darussalam. The investigated range of bitumen contents for the selected wearing course gradation for aggregate type diorite was between 5.0% to 7.0% by mass of the mixture. The investigated bitumen content for the selected wearing course gradation for aggregate type granite was 5.5% by mass of the mixture. The rationale for this is to compare Marshall properties of the diorite-based asphalt mixture to those of the granite-based asphalt mixture, which is more commonly used than the former.

2.3. Crumb rubber and hydrated lime

The CR used in this investigation was procured from a tyre recycling plant in Malaysia. The origin of the CR was mainly from truck and passenger car tyres. As per the supplier's specifications, the CR has a density between 1.1 g/cm^3 to 1.2 g/cm^3. Table 4 shows the physical and chemical properties of CR used in this investigation.

For the diorite aggregate, CR retained on the 0.71 mm BS sieve was used to replace aggregate particles retained on the 1 mm BS sieve in one CR modifying scenario. In another modifying scenario, CR retained on the 2 mm BS sieve was used to replace aggregate particles retained on the 2.36 mm BS sieve. For the granite aggregate, CR retained on the 2.36 mm BS sieve was used to replace aggregate particles retained on the 2.36 mm BS sieve as a third modifying scenario. The proposed CR contents were 0%, 1%, 2% and 4% by the weight of the total mix.

The powdered HL used in the investigation was added as a dry additive that acted as an anti-stripping agent and filler. The HL content used was 2% by mass of the mixture.

Table 2
Basic geological classification of diorite and granite based on silica and quartz contents [70].

Aggregate type and parameter	Diorite	Granite
Silica content (%)	52 – 66	> 66
Main silica mineral	plagioclase	orthoclase
Quartz (%)	0 – 10	> 10
Classification	Intermediate Between mafic gabbro and felsic granite	Acidic Felsic granite

3. Sample preparation and test methods

The Marshall mix design method in ASTM 1559 [75] was used to prepare and test the CRM-DG-HMA samples. The dry ingredients, i.e., aggregate, CR and HL, were weighed and set aside while the bitumen was brought to approximately 160 °C ± 1 °C. The dry ingredients were then heated up to 160 °C ± 1 °C, and bitumen was subsequently added to the heated dry ingredients. The dry and wet ingredients were mixed rapidly and thoroughly until the dry ingredients were coated with bitumen. Thirty-two sample sets were prepared using the diorite aggregate, 4 CR contents of 2 different CR sizes and 4 bitumen contents for each CR content and size. Four sample sets were prepared using the granite aggregate, 4 CR contents of 1 CR size and 1 bitumen content. Each sample set was composed of 3 identical samples, resulting in 108 diorite- and granite-based samples. The samples were then placed into pre-heated moulds with filter papers on each of the two compacting ends. Since the mix design was specific to heavy traffic, each end was compacted with 75 blows. After compaction, the samples were kept overnight at room temperature.

3.1. Sample heights and densities

The samples' H and G_{mb} were measured 24 h ± 0.5 h after compaction. H was measured using vernier callipers, while G_{mb} was usually determined by weighing the samples in air and water. G_{mm} was measured by weighing the loose sample (not compacted) and then determining its volume by calculating the volume of water it displaces.

3.2. Volumetric properties

The volumetric properties P_a, VMA, VFB and P_{ba} were calculated using formulae provided in Refs. [58] and [76].

3.3. Marshall properties

Before the Marshall stability test, the samples were immersed in a water bath at 60 °C ± 1 °C for 35mins ± 5mins. The test machine is a compression testing device designed to apply load diametrically along the circumference. Loading was applied until the samples failed. The force, denoted as S, is measured in newtons (N) or kilonewtons (kN) and the magnitude of displacement/deformation, denoted as F, is measured in millimetres (mm) at failure were recorded.

S is defined as the asphalt's resistance to deformation under load and has been proven dependant on density [77]. In principle, S increases with density until a critical air void content is reached, after which S starts to decrease with increased density for certain bitumen contents [77]. Asphalt with high S does not translate to a superior mix; asphalt with high S may still distort longitudinally by the compaction loads during construction, and longitudinally and transversely by the in-service traffic loads [78]. F is defined as the total movement or displacement that occurred in the test samples between no load and the maximum load applications during the Marshall stability test, i.e., the total deformation corresponding to the maximum load applied at sample failure [73,78].

3.4. Optimum bitumen content (OBC) and optimum crumb rubber content (OCRC)

This is accomplished by identifying asphalt samples that are within the minimum and maximum permissible limits of Marshall parameters in Table 11 (Section 4.4).

Table 3
SG, LAAV, ACV, AIV, flakiness and water absorption of diorite and granite and their typical ranges.

Parameter	Aggregate type and source				Limit for usability wearing course [Ref.]
	Diorite (Sabah, Malaysia)	Typical range [Ref.]	Granite (Karimun, Indonesia)	Typical range [Ref.]	
SG	2.620	2.87 [70]	2.632	2.67 [70]	–
LAAV (%)	14	22 [71]	32	27 [71] 31.19 [62]	< 50 [69]
ACV (%)	16	15 – 20 [71]	23	20 – 25 [71] 36.80 [62]	< 25 [69,72,73]
AIV (%)*	12	13 – 20 [71]	24	17 – 21 [71]	< 25 [73] < 30 [59]
Flakiness (%)	20	–	18	–	< 30 [69]
Water absorption (%)**	1.638	–	1.356	–	< 2 [58,59,69,72]

* The AIV is normally 105% of ACV [72].
** If water absorption is greater than 4%, it becomes unsuitable as a road material [74].

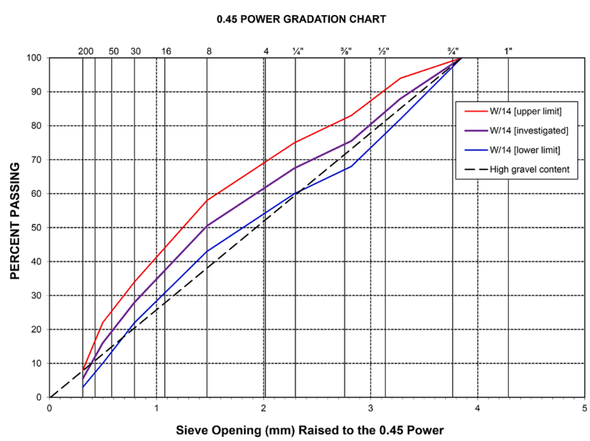

Fig. 1. 0.45 Power gradation curves for aggregate use in asphalt mixtures.

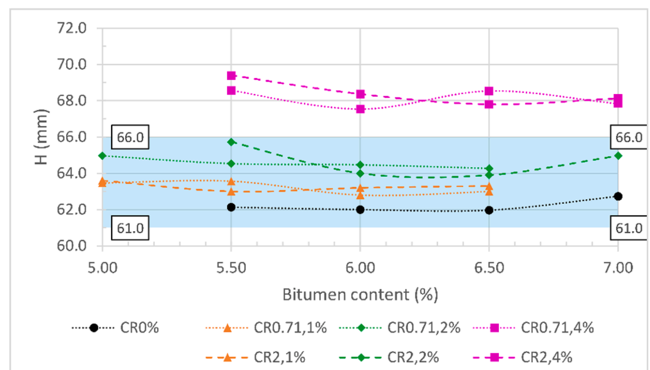

Fig. 2. Height of diorite-based samples (H) using CR sizes 0.71 mm and 2 mm and CR contents 0%, 1%, 2% and 4% for various bitumen contents.

Table 4
Physical and chemical properties of CR as per supplier.

Physical property	Test standard	Value
Passing (sieve size of 3 mm)	ASTM D5644; 5603	> 80%
Heat loss	ASTM D1509	< 1.5%
Steel content	ASTM D5603	< 0.5%
Fibre content	ASTM D5603	< 1%
Chemical property	Test standard	Value
Acetone extract	SMR Bulletin No. 7 '92	8% – 22%
Ash content	SMR Bulletin No. 7 '92	< 8%
Carbon black	TGA	26% – 38%
Rubber hydrocarbon content	TGA	> 42%

4. Results and discussion

4.1. Sample height and densities

This section investigates the influence of 4 key variables: aggregate type, CR size, CR content and bitumen content on CR swelling in CRM-DG-HMA, expressed by the rise in H. This section also investigates how the sample densities (G_{mb}) are affected by the same 4 variables. For this investigation, diorite and granite aggregates were used for the preparation of CRM-DG-HMA samples using 0.71 mm, 2 mm and 2.36 mm CR sizes. CR content ranges from 0% to 4% and bitumen content ranges from 5% to 7%.

4.1.1. Sample height (H)

The absorption of the lighter bitumen fraction (i.e., maltene) had caused the CR to swell and thus increased in H, as illustrated in Fig. 2. For the diorite-based samples with the same CR size and content, the increase in bitumen content resulted in only a small average change in H of less than 1%. CR content has a much more significant effect on swelling – for 1%, 2%, and 4% CR contents, the rise in H averaged about 1.8%, 3.9% and 9.7%, respectively, compared to 0% CR content ($H_{CR0\%}$). The consequent change in V_{mb} impacted the volumetric properties, which is discussed in Section 4.2.

Fig. 3 illustrates how different aggregate types, CR sizes and CR contents lead to increase in H at 5.5% bitumen content. Without CR, H of diorite-based samples exceeded that of granite-based samples by 1.5%. For diorite-based samples with 0.71 mm CR size, H increased compared to the conventional HMA by 2.3%, 3.9% and 10.4% at 1%, 2% and 4% CR contents, respectively. For diorite-based samples with 2 mm CR size, H increased compared to the conventional HMA by 1.4%, 5.8% and 11.7% at 1%, 2% and 4% CR contents, respectively. For granite-based samples with 2.36 mm CR size, H increased compared to the

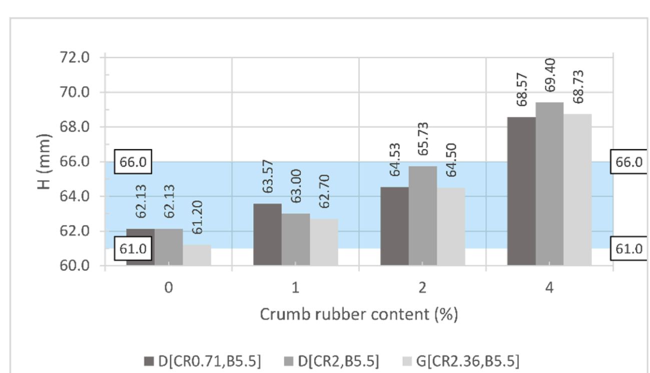

Fig. 3. Height of diorite-based samples (H) using CR sizes 0.71 mm and 2 mm, granite-based samples using CR size 2.36 mm and CR contents 0%, 1%, 2% and 4% at 5.5% bitumen content.

conventional HMA by 2.5%, 5.4% and 12.3% at 1%, 2% and 4% CR contents, respectively. As expected, increases in CR content had resulted in CR swelling, i.e., volumetric increases, in both the granite-based and diorite-based samples.

4.1.2. Sample density (G_{mb})

When CR – less dense with high elasticity – was used to replace aggregate – denser with high rigidity – G_{mb} decreased, as illustrated in Fig. 4. For 1%, 2% and 4% CR contents, the fall in G_{mb} averaged about 2%, 4% and 9% respectively, when compared with 0% CR content ($G_{mb,CR0\%}$). For the samples with the same CR size and content, increasing the bitumen content by 1% beyond the 5.5% bitumen content resulted in an average decrease in G_{mb} of 0.6% at 1% CR content and an average increase in G_{mb} of 1.3% at 2% and 4% CR contents. For diorite-based samples at 4% CR content, the values of G_{mb} for all investigated bitumen contents were consistently below the blue band in Fig. 4, representing reductions in $G_{mb,CR0\%}$ of less than 7%.

Fig. 5 illustrates how different aggregate types, CR sizes and CR contents lead to different G_{mb} at 5.5% bitumen content. For the conventional HMA without CR, G_{mb} of diorite-based samples exceeded that of granite-based samples by 0.5%. For diorite-based CRM HMA with 0.71 mm CR size, G_{mb} decreased below that of the conventional HMA by 2.1%, 4.1% and 9.4% at 1%, 2% and 4% CR contents, respectively. For diorite-based CRM HMA with 2 mm CR size, G_{mb} decreased below that of the conventional HMA by 1.0%, 5.0% and 10.3% at 1%, 2% and 4% CR contents, respectively. For granite-based CRM HMA with 2.36 mm CR size, G_{mb} decreased below that of the conventional HMA by 1.8%, 3.9% and 9.1% at 1%, 2% and 4% CR contents, respectively.

4.2. Volumetric properties

4.2.1. Percentage voids in asphalt (P_a)

As per Ref. [1], the critical factor for successful CRM DG-HMA, based on experiments and experience, is that P_a should be kept low within the permissible limits. The purpose of air voids in asphalt is to allow the asphalt to undergo additional compaction under traffic loads while providing space for bitumen to flow without flushing onto the pavement surface [79]. Ref. [80] explained a high P_a allows oxygen to penetrate the asphalt, thus, making it brittle and prone to fatigue cracking. On the other hand, a low P_a promotes rutting when the air temperature is high, whereby the bitumen expands and saturates the voids. Though high P_a reduces the tensile strength of asphalt [21], high P_a coupled with low density produces asphalt with comparably a short fatigue life [77]. If properly compacted, the asphalt will provide an almost impermeable surface for water runoff [79]. Furthermore, compacted asphalts to high P_a are generally more likely to experience stripping than those compacted to low P_a [38].

When HL is used, P_a decreases with increasing HL content. The investigation by Ref. [81] found that at 2% HL, Pa was 2.8%; at 4% HL, P_a decreased to 2.5%. On the other hand, when CR is used, P_a increases with increasing CR content. CR content greater than 1% causes a considerable increase in P_a and a substantial loss of cohesion between aggregate and bitumen [12].

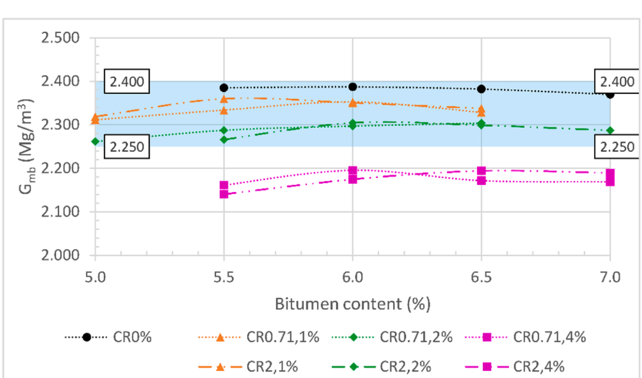

Fig. 4. Density of diorite-based samples (G_{mb}) using CR sizes 0.71 mm and 2 mm and CR contents 0%, 1%, 2% and 4% for various bitumen contents.

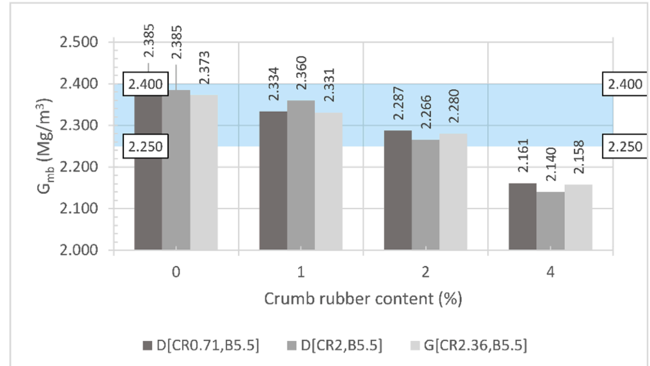

Fig. 5. Density of diorite-based samples (G_{mb}) using CR sizes 0.71 mm and 2 mm, granite-based samples using CR size 2.36 mm and CR contents 0%, 1%, 2% and 4% for bitumen content 5.5%.

Table 5 presents the permissible ranges of P_a for conventional asphalt based on various standards. The table shows that the minimum permissible P_a is 2%, and maximum permissible P_a is 6%. P_a less than 2% tend to produce low stability asphalt [77], which is unacceptable. P_a greater than 8% will allow unimpeded moisture penetration into the asphalt [82], which is also unacceptable. Ref. [83] cited that an OBC of 5.5% using bitumen penetration grade (PG) 60/70 should be sufficient to enhance moisture resistance of DG-HMA of siliceous aggregate. The blue bands (3% – 6%) in Fig. 6 and Fig. 7 represent the permissible ranges of P_a based on the various standards presented in Table 5.

Fig. 6 illustrates the P_a of diorite-based samples using 0.71 mm and 2 mm CR sizes and 0%, 1%, 2% and 4% CR contents for various bitumen contents. It is observed that the greater the CR size and content, the higher P_a at the same bitumen content and P_a for all samples with and without CR decreased with increasing bitumen contents. The only exception was at 1% CR content and 5.5% bitumen content, where P_a of the samples with 0.71 mm CR size was greater than that of the samples with 2 mm CR size. The higher the bitumen content, the less the difference between P_a values related to 0.71 mm and 2 mm CR sizes at the same CR content. This is particularly evident at 2% and 4% CR contents. The CR in the samples made compaction difficult as the high elasticity and swelling of the CR pushed the aggregate apart.

Table 5
Permissible ranges of percentage voids in asphalt (P_a).

Condition	Permissible range	Ref.
For asphalts with non-conventional materials e.g., polymeric wastes		[1]
Light traffic	2% – 3%	
Medium traffic	3%	
Heavy traffic	4%	
For heavy traffic roads in hot climate regions	4% – 6%	[49]
Laboratory-compacted asphalt using the Marshall method at standard temperature		[58, 80]
Optimum P_a	4%	
Normally accepted P_a	3% – 5%	
For heavy traffic roads	3% – 5%	[69]
For roads with slow moving heavy vehicles on climbing lanes and approaching lanes to intersections	3% at 'refusal'/ 'residual' density	[72]
For roads with ESAL 1 – 5 × 10^6	3.5% – 4.5%	[72]
For roads with ESAL greater than 5 × 10^6	4.5% – 5.5%	
For roads impervious to water (infiltration)	Less than 5%	[82]
For roads with minimum fatigue during service	No less than 3% and no greater than 8%	[84]

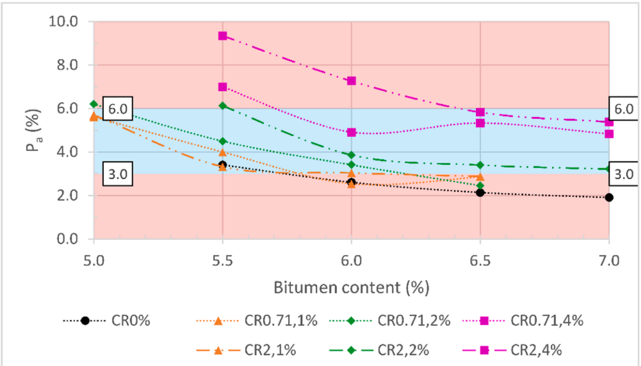

Fig. 6. Percentage voids in asphalt (P_a) of diorite-based samples using CR sizes 0.71 mm and 2 mm and CR contents 0%, 1%, 2% and 4% for various bitumen contents.

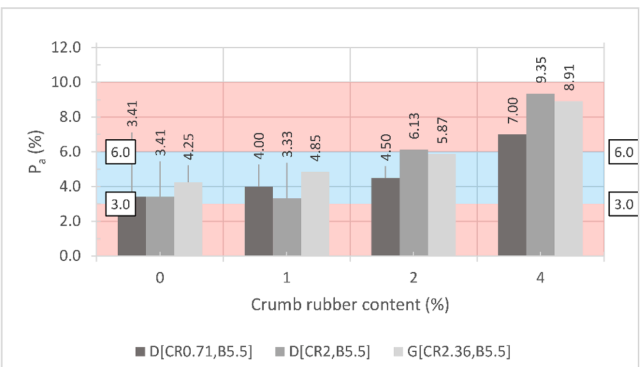

Fig. 7. Percentage voids in asphalt (P_a) of diorite-based samples using CR sizes 0.71 mm and 2 mm, granite-based samples using CR size 2.36 mm and CR contents 0%, 1%, 2% and 4% for bitumen content 5.5%.

Fig. 7 illustrates the P_a of diorite-based samples using 0.71 mm and 2 mm CR sizes, granite-based samples using 2.36 mm CR size and 0%, 1%, 2% and 4% CR contents for 5.5% bitumen content. For the conventional HMA without CR, the P_a of the granite-based sample exceeded that of the diorite-based sample by 25%. For diorite-based samples with 0.71 mm CR size, P_a increased beyond that of the conventional HMA by 17.4%, 32.0% and 105.4% at 1%, 2% and 4% CR contents, respectively. For diorite-based samples with 2 mm CR size, P_a increased beyond that of the conventional HMA by 79.9% and 174.4% at 2% and 4% CR contents, respectively and decreased by 2.4% at 1% CR content. For granite-based samples with 2.36 mm CR size, P_a increased beyond that of the conventional HMA by 14.2%, 38.0% and 109.6% at 1%, 2% and 4% CR contents, respectively. For both diorite- and granite-based samples, P_a exceeded the permissible ranges in Table 5 at 4% CR content and all the investigated CR sizes.

4.2.2. Percentage voids in mineral aggregate (VMA)

VMA represents the intergranular air voids between aggregate particles in a compacted asphalt. VMA consists of the volume of air voids and the volume of effective bitumen [85]. Refs. [58] and [86] explained that VMA must remain high enough to achieve rich bitumen film formation that provides asphalt with better durability. However, if VMA is exceedingly high, it can pose stability and economic problems. Asphalt with below minimum VMA has thin bitumen film that reduces durability and results in dry asphalt. Ref. [85], in their investigation, found that asphalts using either granite (VMA = 12.2%) or coarse limestone (VMA = 13.0%) aggregates have better resistance to both longitudinal and alligator cracking than asphalts using either sandstone (VMA = 9.2%) or fine limestone (VMA = 10.5%) aggregates. For instance, after 10 million equivalent single axial loads (ESALs), the longitudinal cracking increased by 73%, while alligator cracking increased by 24%, after a decrease of 3.8% in VMA when coarse limestone was replaced with sandstone in the asphalt. The larger the nominal maximum aggregate size used, the lower the minimum permissible VMA [57]. Sufficient VMA can be achieved by moving the aggregate gradation some distance away from the MDL on a 0.45 Power curve [87].

Table 6 presents the permissible ranges of VMA for conventional asphalt based on various standards. The table shows that the minimum permissible VMA is 11%, and the maximum permissible VMA is 16%. The blue band (11% – 16%) in Fig. 8 and Fig. 9 represents the permissible ranges of VMA based on various standards, as presented in Table 6.

Fig. 8 illustrates that the VMA for diorite-based samples with no CR but with 2% HL increased with increasing bitumen content. In contrast, the VMA for samples with CR decreased with increasing bitumen content before increasing again. At CR contents of 1%, 2% and 4%, VMA increased by an average of 1.7%, 6.4% and 19.9% beyond that of samples with no CR but 2% HL, respectively. The response of CR to the impact compaction causes the changes in VMA. It is observed that the higher the bitumen content, the less the difference between VMA values for 0.71 mm and 2 mm CR sizes at the same CR content. This is particularly evident at 2% and 4% CR contents.

Fig. 9 illustrates how the aggregate type, CR size, and content influenced the VMA of samples at 5.5% bitumen content. For the conventional HMA without CR, the VMA of granite-based samples exceeded those of diorite-based samples by 9.2%. For diorite-based samples with 0.71 mm CR size, VMA increased compared to the conventional diorite-based HMA by 3.2%, 5.7% and 20.8% at 1%, 2% and 4% CR contents, respectively. For diorite-based samples with 2 mm CR size, VMA increased compared to conventional diorite-based HMA by 16.4% and 36.2% at 2% and 4% CR contents, respectively and decreased by 1.1% at 1% CR content. For granite-based samples with 2.36 mm CR size, VMA increased compared to conventional granite-based HMA by 2.9%, 8.3% and 25.0% at 1%, 2% and 4% CR contents, respectively.

4.2.3. Percentage voids filled with bitumen (VFB)

VFB is defined as the percentage by volume of VMA filled with effective bitumen [58]; thus, VFB is inversely proportional to P_a [73]. Ref. [88] explained that a sufficient VFB ensures that the bitumen coat on aggregate is not too thin to be susceptible to damage by oxidation (air) and moisture (surface runoff). On the contrary, high VFB can cause bleeding and flushing. Often, asphalt that satisfies VFB permissible ranges does not satisfy those of VMA. The U.S. Army Corps uses VFB as one of their Marshall mix design parameters instead of VMA [89]. However, the U.S. Department of Transportation (DOT) views VFB as an important design property, i.e., specification requires VFB of between 70% and 80% during the design stage; it is not a production requirement [90].

Table 6
Permissible ranges of percentage voids in mineral aggregate (VMA).

Condition	Permissible range	Ref.
For heavy traffic roads in hot climate regions using max. aggregate size larger than 12.7 mm but less than 19.1 mm	14% – 15%	[49]
For heavy traffic roads (ESAL > 1 × 10^6) using max. aggregate size larger than 12.5 mm but less than 19.0 mm		[58]
Design P_a at 3%	12% – 13%	
Design P_a at 4%	13% – 14%	
Design P_a at 5%	14% – 15%	
For wearing course	12% to 16% (depending on max. particle size)	[57]
For roads with ESAL less than 0.4 × 10^6, between 0.4 × 10^6 and 5 × 10^6 and greater than 5 × 10^6	11% – 16% (depending on max. particle size)	[72]

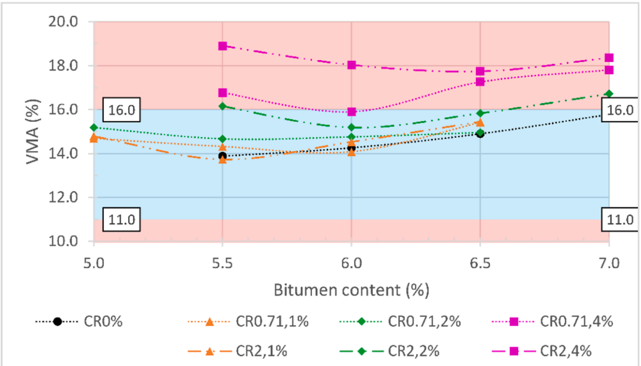

Fig. 8. Percentage voids in mineral aggregate (VMA) of diorite-based samples using CR sizes 0.71 mm and 2 mm and CR contents 0%, 1%, 2% and 4% for various bitumen contents.

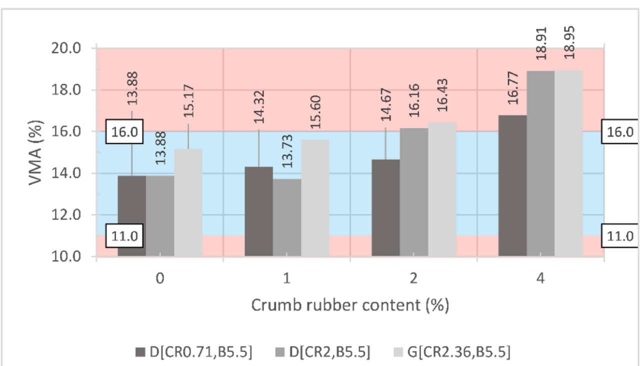

Fig. 9. Percentage voids in mineral aggregate (VMA) of diorite-based samples using CR sizes 0.71 mm and 2 mm, granite-based samples using CR size 2.36 mm and CR contents 0%, 1%, 2% and 4% for bitumen content 5.5%.

Table 7 presents the VFB for conventional asphalt based on various standards. The table shows that the minimum permissible VFB is 65%, and the maximum permissible VFB is 80%. It is noted that the permissible minimum limit of VFB typically varies from 65% to 75%, as shown in Table 7. For countries with high rainfall intensity, high ambient temperature and high ultraviolet index, the VFB should be as close to 70% [88]. VFB of 75% to 85% would not be a practical specification for production [89].

For all the diorite-based samples with or without CR, the increase in bitumen content increased the percentage of voids filled with bitumen (VFB), as illustrated in Fig. 10. At the same bitumen content, the addition of CR increased P_a (Fig. 7), thus, decreased VFB. Increasing the bitumen content increased VFB by an average of 2.8%, 7.5% and 22.0% beyond the samples with no CR but 2% HL at CR contents of 1%, 2% and 4%, respectively. This behaviour is attributed to the probable absorption

Table 7
Permissible ranges of percentage voids filled with bitumen (VFB).

Condition	Permissible range	Ref.
For heavy traffic roads	75% – 80%	[69]
For heavy traffic roads (ESAL > 1×10^6)	65% – 75%	[58]
For roads with ESAL 1 – 5×10^6	65% – 75%	[72]
For roads with ESAL greater than 5×10^6	65% – 73%	
For satisfactory asphalt	68% – 77%	[89]
For fair or good asphalt	68% – 83%	
Bituminous concrete mix (source: MoRTH, India)	65% – 75%	[91]
Bituminous wearing course (source: JKR/SPJ/2008-S4, Malaysia)	70% – 80%	[92]

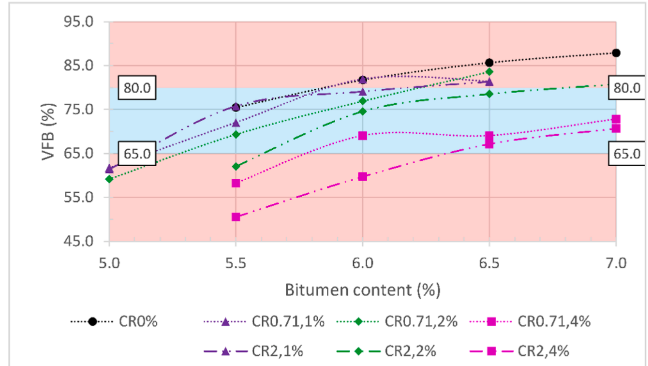

Fig. 10. Percentage voids filled with bitumen (VFB) of diorite-based samples using CR sizes 0.71 mm and 2 mm and CR contents 0%, 1%, 2% and 4% for various bitumen contents.

of bitumen by CR, which increases the need for more bitumen to achieve the permissible P_a ranges [93]. The higher the bitumen content, the less the difference between VFB values for 0.71 mm and 2 mm CR sizes at the same CR content. This is particularly evident at 2% and 4% CR contents. At 4% CR content, the VFB dropped below the permissible ranges in Table 7, which are depicted by the blue band (65% – 80%) in Fig. 10.

Fig. 11 illustrates the influence of aggregate type, CR size and CR content on the VFB of samples at 5.5% bitumen content. For the conventional HMA without CR, the VFB of diorite-based samples exceeded those of granite-based samples by 4.8%. For diorite-based samples with 0.71 mm CR size, VFB decreased compared to that of the conventional diorite-based HMA by 4.5%, 8.1% and 22.8% at 1%, 2% and 4% CR contents, respectively. For diorite-based samples with 2 mm CR size, VFB decreased compared to that of the conventional diorite-based HMA by 17.8% and 33.0% at 2% and 4% CR contents, respectively and increased by 0.4% at 1% CR content. For granite-based samples with 2.36 mm CR size, VFB decreased compared to that of the conventional granite-based HMA by 4.3%, 10.7% and 26.4% at 1%, 2% and 4% CR contents, respectively.

4.2.4. *Percentage of absorbed bitumen (P_{ba})*

Aggregate with high P_{ba} absorbs a large proportion of bitumen into the pores of the aggregate, which leaves less bitumen for film formation and thickness; this makes the asphalt drier and stiffer [78]. In this case, if bitumen content is not increased for compensation, then the amount of compactive effort needs to be increased to achieve the desired G_{mb}; otherwise, the HMA would ravel under traffic loads [78]. There is a high consensus that aggregate water absorption should be less than 2% [49, 57,58,69]. Bitumen absorption is typically between 40% and 80% of the

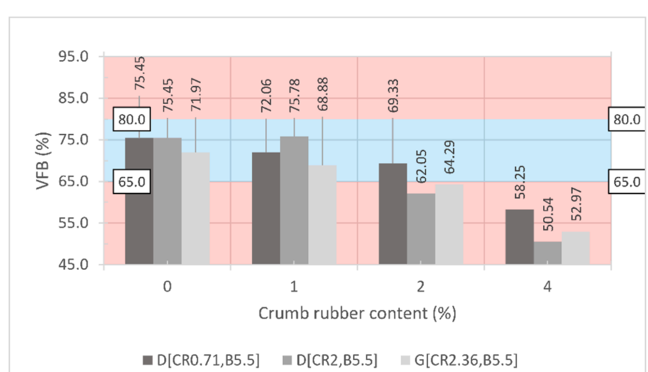

Fig. 11. Percentage voids filled with bitumen (VFB) of diorite-based samples using CR sizes 0.71 mm and 2 mm, granite-based samples using CR size 2.36 mm and CR contents 0%, 1%, 2% and 4% for bitumen content 5.5%.

water absorption rate [58]. The permissible ranges of P_{ba} for conventional asphalt based on various standards are shown in Table 8.

The diorite and granite used in the investigation had water absorption of 1.638% and 1.356%, respectively; thus, diorite-based HMA had higher P_{ba} than granite-based HMA at 5.5% bitumen content, which is illustrated in Fig. 12. For the conventional HMA without CR, the P_{ba} of diorite-based samples exceeded that of granite-based samples by 28.5%. For diorite-based samples with 0.71 mm CR size, P_{ba} decreased compared to the conventional diorite-based HMA by 3.2%, 5.8% and 13.7% at 1%, 2% and 4% CR contents, respectively. For diorite-based samples with 2 mm CR size, P_{ba} decreased compared to conventional diorite-based HMA by 1.9%, 3.4% and 7.7% at 1%, 2% and 4% CR contents, respectively. For granite-based samples with 2.36 mm CR size, P_{ba} decreased compared to the conventional granite-based HMA by 1.3%, 4.3% and 7.1% at 1%, 2% and 4% CR contents, respectively.

4.3. Marshall properties

4.3.1. Marshall stability (S)

The permissible ranges of S for conventional asphalt based on various standards are shown in Table 9. The blue bands (8 kN – 18.5 kN) in Fig. 13 and Fig. 14 represent the permissible ranges of S based on the various standards presented in Table 9.

Fig. 13 illustrates that the S of the diorite-based samples with no CR but 2% HL decreased linearly from 17.80 kN to 11.49 kN with increasing the bitumen content from 5.5% to 7%, which is similar to the trend of percentage voids in asphalt (P_a) illustrated in Fig. 6. When CR was added at 5.5% bitumen content, the higher the CR content, the less the values of S. When the bitumen content increased to 6%, S of most of the samples with CR exceeded that of the samples without CR, expect for the samples at 4% CR content of sizes 0.71 mm and 2 mm and the samples at 1% CR content of 2 mm CR size. Beyond 6% bitumen content, S for all samples continued to decrease linearly until they converged at 6.5% bitumen content. By 7% bitumen content, S for all samples were 12 kN and less. For conventional and CRM HMA with all investigated CR sizes and contents, S of the samples at all investigated bitumen contents was within the blue band in Fig. 13 and did not exceed the permissible ranges of S in Table 9.

Fig. 14 illustrates the influence of aggregate type, CR size and CR content on the S of samples at 5.5% bitumen content. For the conventional HMA without CR, S of diorite-based samples exceeded that of granite-based samples by 12.5%. For diorite-based samples with 0.71 mm CR size, S decreased below that of the conventional HMA by 1.6%, 5.2% and 28.7% at 1%, 2% and 4% CR contents, respectively. For diorite-based samples with 2 mm CR size, S decreased below that of the conventional HMA by 7.4%, 17.0% and 33.6% at 1%, 2% and 4% CR contents, respectively. For granite-based samples with 2.36 mm CR size, S increased beyond that of the conventional HMA by 14.5% and 1.5% at 1% and 2% CR contents, respectively and decreased below that of the conventional HMA by 16.6% at 4% CR content.

4.3.2. Marshall flow (F)

The permissible ranges of F for conventional asphalt based on various standards are shown in Table 10. The blue bands (2 mm – 5 mm)

Table 8
Permissible ranges of percentage of absorbed bitumen (P_{ba}).

Condition	Permissible range	Ref.
For heavy traffic roads in hot climate regions to reduce (bitumen) bleeding using aggregate with water absorption less than 2%	< 2%	[49]
For heavy traffic roads (ESAL > 1×10^6) using aggregate with water absorption less than 2%	< 1%	[58]
Highly absorptive aggregate (P_{ba} > 2%) may not be economical as relatively high bitumen is needed	< 2%	[57]

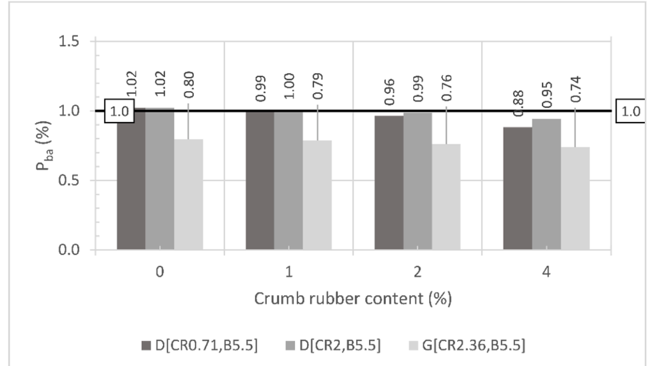

Fig. 12. Percentage of absorbed bitumen (P_{ba}) of diorite-based samples using CR sizes 0.71 mm and 2 mm, granite-based samples using CR size 2.36 mm and CR contents 0%, 1%, 2% and 4% for bitumen content 5.5%.

Table 9
Permissible ranges of Marshall stability (S).

Condition	Permissible range	Ref.
For heavy traffic roads in hot climate regions	Min. 6.8 kN Max. 12.5 kN	[49]
For heavy traffic roads	Min. 9.0 kN	[69]
For heavy traffic roads (ESAL > 1×10^6)	Min. 8.006 kN	[58]
For roads with ESAL 1 – 5 × 10^6	Min. 8.0 kN	[72]
For roads with ESAL greater than 5 × 10^6	Min. 9.0 kN	

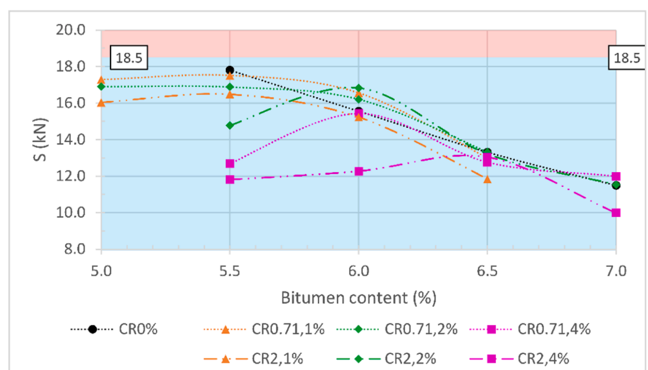

Fig. 13. Marshall stability (S) of diorite-based samples using CR sizes 0.71 mm and 2 mm and CR contents 0%, 1%, 2% and 4% for various bitumen contents.

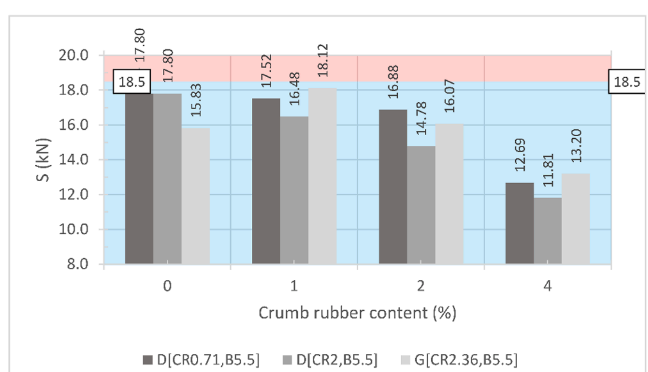

Fig. 14. Marshall stability (S) of diorite-based samples using CR sizes 0.71 mm and 2 mm, granite-based samples using CR size 2.36 mm and CR contents 0%, 1%, 2% and 4% for bitumen content 5.5%.

Table 10
Permissible ranges of Marshall flow (F).

Condition	Permissible range	Ref.
For heavy traffic roads in hot climate regions	2.0 mm – 4.0 mm	[49]
For heavy traffic roads	2.0 mm – 4.0 mm	[69]
For heavy traffic roads (ESAL > 1 × 10^6)	2.032 mm – 3.556 mm	[58]
For roads with ESAL 1 – 5 × 10^6 For roads with ESAL greater than 5 × 10^6	2.0 mm – 3.5 mm	[72]
For asphalts with non-conventional materials e.g., polymeric wastes and etc	2.0 mm – 5.0 mm	[62, 94-96]

in Fig. 15 and Fig. 16 represent the permissible ranges of F based on the various standards presented in Table 10.

Fig. 15 illustrates that the F of the diorite-based samples with no CR but 2% HL was about 4 mm, which was one of the maximum permissible F limits in Table 10. F of this sample increased by 38% with increasing the bitumen content by 18% from the OBC. At 5.5% bitumen content, the average F (for both CR sizes) were 3.3 mm, 3.4 mm and 3.9 mm at 1%, 2% and 4% CR contents respectively. As the bitumen content increased from 5.5% to 6.0%, the average F increased by 8.4%, 34.2%, 29.3% and 27.6% at 0%, 1%, 2% and 4% CR contents respectively. At 6.0% bitumen content, the average F (for both CR sizes) were 4.3 mm, 4.4 mm, 4.3 mm and 4.9 mm at 0%, 1%, 2% and 4% CR contents respectively. As the bitumen content increased from 6.0% to 6.5%, the average F increased by 26.9%, 41.6%, 19.9% and 24.5% at 0%, 1%, 2% and 4% CR contents respectively. At 6.5% bitumen content, the samples for all investigated bitumen content, CR sizes and CR contents were greater than 5 mm.

Fig. 16 illustrates the influence of aggregate type, CR size and CR content on the F of samples at 5.5% bitumen content. For the conventional HMA without CR, F of granite-based samples exceeded that of diorite-based samples by 5.8%. For the diorite-based samples with 0.71 mm CR size, F decreased below that of the conventional HMA by 13.9%, 19.6% and 22.2% at 1%, 2% and 4% CR contents, respectively. For the diorite-based samples with 2 mm CR size, F decreased below that of the conventional HMA by 17.3% and 9.4% at 1% and 2% CR contents, respectively, and increased above that of the conventional HMA by 19.0% at 4% CR content. For the granite-based samples with 2.36 mm CR size, F decreased below that of the conventional HMA by 2.1% and 6.4% at 1% and 2% CR contents, respectively, and increased above that of the conventional HMA by 12.1% at 4% CR content.

4.4. Selection based on Marshall parameters optimisation

Table 11 is a cumulation of Marshall parameters' minimum and maximum permissible limits from Tables 5–10. Using the limits/ranges from Table 11, an optimum mix design that fulfilled P_a, VMA, VFB, P_{ba}, S

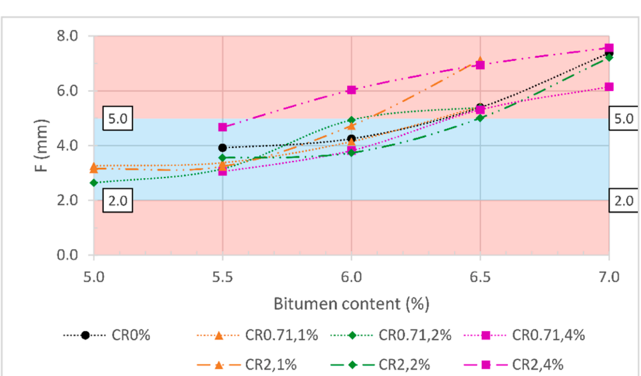

Fig. 15. Marshall flow (F) of diorite-based samples using CR sizes 0.71 mm and 2 mm and CR contents 0%, 1%, 2% and 4% for various bitumen contents.

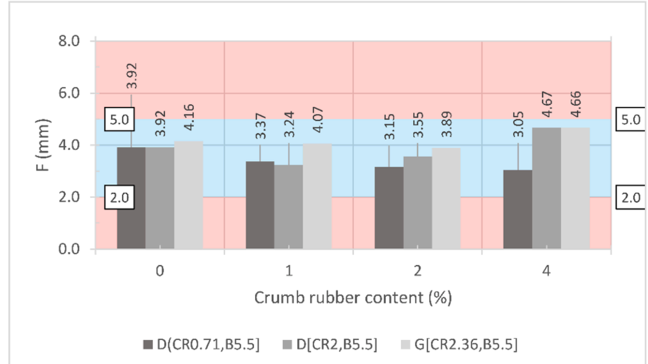

Fig. 16. Marshall flow (F) of diorite-based samples using CR sizes 0.71 mm and 2 mm, granite-based samples using CR size 2.36 mm and CR contents 0%, 1%, 2% and 4% for bitumen content 5.5%.

Table 11
Permissible minimum and maximum limits of Marshall parameters.

Marshall parameter	Specifications	
	Permissible min.	Permissible max.
P_a	3%	6%
VMA	11%	16%
VFB	65%	80%
P_{ba}	1%	2%
S	8.0 kN	–
F	2.0 mm	5.0 mm

and F within the permissible limits was determined. Table 12 shows the samples that satisfied the permissible limits of Marshall parameters presented in Table 11.

Table 13 compares Marshal properties of this investigation's conventional HMA to relevant past investigations' HMA without HL that used different aggregate types, aggregate G:S mass ratio, bitumen content and bitumen type. The table shows that G_{mb} values from past investigations, particularly with high G:S mass ratios, were greater than that of this investigation, except for Ref. [97]. Although S from this investigation outperformed past investigations, the F from this investigation generally underperformed those from past investigations. This indicated that adding HL to the asphalt samples of this investigation improved stability significantly but did not help much with the permanent deformation.

Table 14 compares Marshal properties of this investigation's CRM-HMA to relevant past investigations' CRM-HMA without HL that used different CR size, CR content, aggregate types, aggregate G:S mass ratio, bitumen content and bitumen type. The table shows that most of the G_{mb} of CRM asphalt samples from past investigations were lower than that of this investigation, except for those by Ref. [64] at 8% and 10% CR contents. The S and F of CRM asphalt samples from this investigation generally outperformed those from past investigations. This indicated that adding HL to the CRM asphalt samples of this investigation improved the stability and reduced permanent deformation.

Table 12
Samples that satisfy permissible limits of Marshall parameters.

Agg. type: Diorite CR size: 0.71mm	Agg. type: Diorite CR size: 2mm	Agg. type: Granite CR size: 2.36mm
BC = 5.5%, CR content = 0%, 1% and 2%.	BC = 5.5%, CR content = 0% and 1%.	BC = 5.5%, CR content = 0% and 1%.
BC = 6.0%, CR content = 2% and 4%.	BC = 6.0%, CR content = 1% and 2%.	

BC: bitumen content.

Table 13
G_{mb}, P_a, VMA, VFB, S and F of conventional asphalt by dry process from relevant past investigations.

Aggregate type and G:S mass ratio	Bitumen content (%) and type	G_{mb}	P_a (%) *3–6*	VMA (%) *11–16*	VFB (%) *65–80*	S (kN) *> 8.0*	F (mm) *2–5*	Remark / Ref.
Diorite (G:S = 0.92)	5.5* PG 60/70 *OBC	2.385	3.41	13.88	75.45	17.80	3.92	This investigation and with 2% HL
G:S = 1.63	4.46*	2.346	3.84	–	72.0	9.9	2.8	[97] (2016)
G:S = 1.38	4.46*	2.348	3.86	–	74.8	10.6	3.3	
G:S = 1.17	4.42*	2.356	3.62	–	74.0	11.5	3.0	
G:S = 1.00	4.57*	2.336	4.20	–	65.0	9.9	3.1	
G:S = 0.85	4.46*	2.342	3.88	–	72.2	9.7	2.7	
Unnamed crushed aggregate	VG-30 *OBC							
G:S = 2.6	5.22*	2.396	3.8	16.21	–	10.07	3.36	[98] (2017)
G:S = 1.6	5.26*	2.400	3.5	16.05	–	13.55	3.30	
G:S = 1.3	5.37*	2.407	2.8	15.63	–	15.50	2.65	
G:S = 2.9	5.30*	2.390	3.7	16.26	–	13.05	2.50	
G:S = 1.7	5.23*	2.393	3.8	16.10	–	11.95	2.90	
G:S = 0.9	6.06*	2.395	2.8	15.74	–	13.30	3.40	
Unnamed crushed aggregate	PG 80/100 *OBC							
Unnamed crushed aggregate (G:S = 1.8)	4.5	2.390	7.69	14.15	45.88	12.7	2.8	[91] (2018)
	5.0	2.420	5.83	13.58	57.09	14.2	3.2	
	5.5*	2.440	4.28	13.31	67.97	15.0	3.8	
	6.0	2.450	3.11	13.39	77.24	12.5	4.3	
	VG-30 *OBC							

OBC: Optimum bitumen content.

Table 15 compares Marshal properties of this investigation's conventional HMA to relevant past investigations' HMA with HL that used different aggregate types, aggregate G:S mass ratio, bitumen content and bitumen type. The table shows that G_{mb} values from past investigations were less than that of this investigation. Although S from this investigation outperformed past investigations, the F from this investigation generally underperformed those from past investigations. This indicated that the conventional HMA from this investigation is more prone to permanent deformation than those from past investigations.

5. Conclusions

The following are the general conclusions obtained from this experimental investigation:

5.1. Effects of CR sizes

- CR sizes had more prominent effects on air void properties – P_a, VMA and VFB and Marshall parameter – F, especially at 2% and 4% CR contents. At these contents, P_a, VMA and F of modified asphalt using 0.71 mm CR size were lesser than those using 2 mm and 2.36 mm CR sizes, whereas VFB of modified asphalt using 0.71 mm CR size were more than those using 2 mm and 2.36 mm CR sizes, regardless of aggregate types used.
- When large-sized CR (2.36 mm or 2 mm) replaces the equivalent mineral aggregate size (2.36 mm) by mass, it has undesirable CR swelling due to bitumen absorption (by CR). CR being elastic, when compressed, would decrease in volume – it was deduced that the presence of air void mainly caused this. When a small-sized CR (0.71 mm) replaces the equivalent mineral aggregate size (1 mm), the undesirable effect is reduced as the small-sized CR fills the air voids between the mineral aggregate. Thus, the aggregate resists the applied force when the modified asphalt is compressed.

5.2. Effects of CR contents

- The effects of CR contents on most of the asphalt properties were more significant as the properties of dry process rubberised HMA were more sensitive to the change in CR content than CR size. As the CR content increased, H, P_a, VMA and F increased, whereas G_{mb}, VFB, P_{ba} and S decreased. At 4% CR content, H, G_{mb}, P_a, VMA and VFB exceed their respective permissible ranges.
- As the increasing CR content absorbs more bitumen, CR swelling becomes more significant, and the amount of effective bitumen between the mineral aggregate becomes lesser/thinner. Similarly, the bulk density decreases as more CR replaces more mineral aggregate. Thus, CR content between 1% and 2% is highly recommended for small-sized CR to reduce both absorption and swelling by CR.

5.3. Effects of aggregate types

- The effects on aggregate types were studied at 5.5% BC only. Only H and G_{mb} were not affected by the aggregate types used. At 0% CR content, P_a, VMA and F of diorite-based asphalt were less than those of granite, whereas VFB and S of diorite-based asphalt were greater than granite. Ultimately, the CR size and content were the dominant factors that affected the air void and Marshall parameters and, to a lesser extent, the aggregate type.
- Since diorite has higher bitumen absorption than granite, thus, for all investigated CR contents, P_{ba} of diorite-based asphalt were more than those of granite, regardless of CR size.

5.4. Effects of bitumen contents

The effects of bitumen content were observed for both unmodified and modified asphalt, regardless of CR sizes and contents for diorite aggregate, except for H, G_{mb} and P_{ba}. As bitumen content increased, VFB and F increased, whereas P_a, VMA and S decreased. The minimum bitumen required was between 5.5% and 6.0%.

5.5. Summary of results

Based on the results and outcomes from the laboratory investigation, the bitumen content and CR content, at which P_a, VMA, VFB, P_{ba}, S and F were within the permissible ranges for diorite-based samples using 0.71 mm and 2 mm CR sizes and granite-based samples using 2.36 mm CR size, were 5.5% – 6.0% and 1% – 2%, respectively.

Table 14
G_{mb}, P_a, VMA, VFB, S and F of CRM asphalt by dry process from relevant past investigations.

CR content (%)	CR size (mm)	Agg. Type and G:S mass ratio	Bitumen content (%) and type	G_{mb}	P_a (%) *3–6*	VMA (%) *11–16*	VFB (%) *65–80*	S (kN) *> 8.0*	F (mm) *2–5*	Remark / Ref.
0	–	Diorite	5.5*	2.385	3.41	13.88	75.45	17.80	3.92	This investigation and
1	0.71	(G:S = 0.92)	5.5*	2.334	4.00	14.32	72.06	17.52	3.37	with 2% HL
1	2		5.5*	2.360	3.33	13.73	75.78	16.48	3.24	
			PG 60/70							
			*OBC							
0	–	Spain crushed ortho-	5	2.310	4.9	16.36	–	17.78	2.89	[4] (2009)
1	0.5–1	quarzite	5	2.310	4.9	16.10	69.6	17.20	2.70	
		(G:S = 2.33)	PG 60/70							
0	–	Unnamed Malaysia	5.3	2.329	4.2	–	80	12.65	3.15	[2] (2014)
2	0.15	crushed agg. (G:S 1.86)	5.6	2.206	3.0	–	67	10.69	2.90	
2	0.425		5.55	2.258	3.0	–	70	10.59	2.95	
2	1.18		5.5	2.323	3.6	–	80	9.43	5.10	
			PG 80/100							
0	–	Nigeria crushed granite	6.22	2.354	3.23	–	81.19	7.24	4.04	[62] (2015)
2	2.36	(G:S = 0.96)	6.22	2.288	3.05	–	81.92	5.47	3.71	
4	2.36		6.22	2.276	3.53	–	79.45	4.95	4.05	
2	0.6		6.22	2.300	3.02	–	81.79	6.20	3.74	
4	0.6		6.22	2.246	2.28	–	85.82	4.97	4.22	
			PG 60/70							
0	–	Palestine crushed granite	5	2.350	4.60	16.00	–	14.70	2.35	[8] (2016)
5	0.063	(G:S = 3)	5	2.150	4.48	14.75	–	14.71	2.55	
10	0.063		5	1.725	3.00	11.50	–	3.1	2.20	
15	0.063		5	1.590	2.79	10.50	–	3.0	2.15	
			PG 60/70							
0	–	Jordan crushed limestone	5.5	2.415*	7.37	14.35	–	4.5	2.0	[65] (2016)
20*	2.36	(G:S = 2.33)	5.5	2.030*	2.71	33.21	–	1.5	3.0	
30*	2.36		5.5	1.990*	4.08	26.87	–	9.0	2.5	
40*	2.36		5.5	1.830*	4.97	33.23	–	1.0	3.5	
20*	0.3		5.5	1.963*	3.51	27.26	–	9.0	6.0	
30*	0.3		5.5	1.821*	6.04	34.29	–	3.0	2.0	
40*	0.3		5.5	1.780*	6.40	36.01	–	8.5	2.0	
*by mass of bitumen			PG 80/100	*G_{mm}						
0	–	Nigeria crushed granite	5.9	2.240	4.0	17.5	60.5	12.65	3.41	[64] (2018)
2	9.5–12.5 (coarse	(G:S = 1)	5.9	2.265	4.6	18.1	64.0	11.82	3.67	
4	rubber)		5.9	2.310	6.2	19.3	70.0	11.02	3.82	
6			5.9	2.315	6.3	19.4	70.5	9.35	3.98	
8			5.9	2.345	8.0	21.0	76.0	8.25	4.18	
10			5.9	2.365	9.0	21.8	80.0	8.01	4.37	
			PG 60/70							
0	–	Unnamed Indonesia	6	1.737	3.5	18.63	66.0	13.25	3.55	[68] (2019)
0.5*	0.149	crushed agg.	6	–	3.5	18.75	66.0	13.50	4.32	
1*	0.149	(G:S = –)	6	–	3.6	18.87	66.5	14.65	4.40	
1.5*	0.149		6	–	4.4	19.00	64.5	13.05	4.35	
3*	0.149		6	–	4.5	19.25	64.5	10.90	4.46	
4.5*	0.149		6	–	7.4	20.50	59.0	8.90	4.60	
6*	0.149		6	–	12.6	22.74	52.0	7.40	4.85	
0.5*	0.297		6	–	3.5	18.75	66.0	12.90	4.35	
1*	0.297		6	–	4.0	18.87	65.0	13.50	4.40	
1.5*	0.297		6	–	3.8	18.75	65.5	11.50	4.35	
3*	0.297		6	–	4.0	19.00	65.0	10.00	4.45	
4.5*	0.297		6	–	8.6	20.87	57.5	8.60	4.60	
6*	0.297		6	–	11.5	22.10	53.5	8.50	4.82	
*by mass of bitumen			PG 60/70							

PG: Bitumen penetration grade.
G_{mm}: Theoretical maximum specific gravity.

6. Recommendations

A future experimental investigation is recommended to compare the performance of CRM-DG-HMA by the dry process to that of conventional HMA in terms of rutting and fatigue resistances, stiffness, moisture susceptibility and ageing. This may confirm the optimum CR content, CR size and bitumen content identified in this research for preparing CRM-DG-HMA by dry process. It is also recommended for further investigations to adopt an experimental design that allows for sufficient sample sizes to perform statistical analysis. Based on the comparison presented in this paper of volumetric and Marshall properties between CRM-HMA and those of conventional HMA, further research is also recommended to investigate the modification of the Marshall Mix Design method to suit the application of dry CR to dense-graded HMA. Further research is recommended into adopting the same CR size for samples based on the diorite and granite aggregates to investigate whether the comparison of the various properties presented in this paper is still valid.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Table 15
G_{mb}, P_a, VMA, VFB, S and F of conventional asphalt treated with HL by dry process from relevant past investigations.

HL content (%)	Aggregate type and G:S mass ratio	Bitumen content (%) and type	G_{mb}	P_a (%) *3–6*	VMA (%) *11–16*	VFB (%) *65–80*	S (kN) *> 8.0*	F (mm) *2–5*	Remark / Ref.
2	Diorite (G:S = 0.92)	5.5* PG 60/70 *OBC	2.385	3.41	13.88	75.45	17.80	3.92	This investigation and with 0% CR
2	Unnamed crushed aggregate (G:S = –)	6.5 PG 60/70	2.352	–	–	–	8.69	3.5	[54] (2016)
10	Unnamed crushed aggregate (G:S = 0.56)	6.5 PG 60/70	2.360	3.8	–	79.0	8.20	3.4	[99] (2016)
2 4	Crushed mixture of limestone, sandstone and granite (G:S = 1.9)	4–5.5 (PG -)	2.115* 2.130* *G_{sa}	2.8 2.5	–	–	9.20 11.91	3.40 3.10	[81] (2017)

G_{sa}: Apparent specific gravity.

Data availability

The authors do not have permission to share data.

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