KWAME NKRUMAH UNIVERSITY OF SCIENCE AND TECHNOLOGY,

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TOWARDS IMPROVED QUALITY AND PERFORMANCE OF COLD-MIX ASPHALTS FOR BITUMINOUS PAVEMENT MAINTENANCE IN GHANA

By

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DECLARATION

I hereby declare that this submission is my own work towards the PhD and that, to the best of my knowledge and belief, it contains no material previously published or written by another person nor material which, to a substantial extent, has been accepted for the award of any other degree or diploma at the Kwame Nkrumah University of Science and Technology, Kumasi, or any other educational institution, except where due acknowledgement is made in the thesis.

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DEDICATION

This work is dedicated to my wife, Doreen Osei Amponsh, my kids, Michael Antwi Boateng, Mercedes Serwaa Boateng, Francis Osei Boateng and Ferdinand Osei Boateng for their love and support.

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ABSTRACT

Premature failure and poor performance of cold-mix asphalts (CMAs) used in pothole patching and sectional repairs on bituminous roads are very common in Ghana, and yet, for a long time, the problem has remained unaddressed while the material continues to be used. There is, therefore, the need to improve the quality of cold-mix asphalts used in the country in order to achieve a more successful and durable product in road maintenance. To this end, Ghana's Ministry of Transportation (MOT) Standard Specification for Road and Bridge Works (2007) was reviewed alongside three foreign specifications on CMAs, namely, Asphalt Institute MS 19 (1997), Chevron USA Incorporated Procedure and Nikolaides Specification (1994), to establish possible commonalities and areas of deviations that could impact the field performance of the material. Also, samples from ready-to-use CMA stockpiles at six contractor sites across the country, and failed road patches were investigated. Further, a series of dense-graded cold-mix asphalts, with simplified material proportioning ratios, were proposed and investigated for adoption and use. Lastly, potential improvement in CMA properties, using montmorillonite nanoclay filler, was explored. It was established that Ghana's MOT Specification lacked specificity and clarity on the subject of CMAs. In the case of the other Specifications, though variations in mixture design and testing protocols existed, there appeared to be a general consensus on the use of different aggregate gradation structures that lead to high field performance. The asphalt contents of the cold-mix asphalt samples taken from the field tended to range between 3% and 6%, with emulsion mixes having the lower values. In addition, the aggregate structures of the mixes were poor, consisted essentially of single-size aggregates with uniformity coefficient values that ranged between 1 and 4. Some specimens compacted in the laboratory disintegrated during conditioning for stability and flow test, suggesting a lack of stickiness and cohesion within the compacted matrix. It, therefore, appears that the early failures characterizing cold-mix asphalts used in maintenance works in the country could be due principally to inadequate aggregate structure and low binder content of the mixes. For improved cold-mix quality, five aggregate gradation blends that meet GHA dense grading requirements, together with the corresponding emulsion-to-aggregate ratio, for easy material batching in the field, have been proposed. Montmorillonite nanoclay used as filler increased the optimum bitumen content and improved the stability of the cold-mixes but resulted in significantly high flows. This suggests that their use in cold-mix asphalts has the potential to induce plastic behaviour and render the mixes more rut-susceptible, especially at high temperatures.

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CHAPTER 1: INTRODUCTION

1.1 Problem Statement

In Ghana, the road transport sector accounts for approximately 98% of passenger traffic and about the same percentage of freight transport (GIPC, 2012). This places a big responsibility on the road agencies existing under the Ministry of Roads and Highways to maintain and keep the country's roads in pristine condition at all seasons. However, for the most part, the country's road network is fraught with premature pavement failures and a generally poor road condition mix. In particular, most chip-sealed roads are notorious for pothole development to the extent that they may come under several cycles of maintenance interventions in the form of pothole patching and/or sectional repairs before reaching the end of the design life. The many cycles of pothole patching derive, in part, from patches that fail shortly after placement and, in part, from new ones that develop subsequently. If potholes remain unpatched for a long time or re-patching of failed patches is delayed, the quantity of patching work to undertake will quickly increase in size, and make repair works more costly and difficult at a later date. Prevalence of premature pothole patching and sectional repair failures in bituminous pavement maintenance in Ghana is a major concern as these rampant patch failures call for repeated patching in the face of limited maintenance budget.

The need to achieve a long-lasting pothole patch cannot be overemphasized given the fact that potholes have been associated with accident causation, ride discomfort, traffic congestion, increase in air pollution from idling engines during traffic congestion, fuel wastage and wear and tear on vehicles (Goliya *et al*., 2016).

Kuennen (2004) and Chan *et al*. (2011) have established that using high quality materials is one of the most important ways to ensure patch success. For pothole and similar repairs on bituminous roads in the country, cold-mix asphalt (CMA) is the preferred material used extensively. It is produced at ambient temperature as a mixture of liquefied bitumen, aggregates and filler (Fang *et al*., 2016). Like any bituminous patching material, CMA is expected to have some special and important properties to make a longer lasting patch. Kuhn *et al.* (2004) have advocated for CMA use due to its versatility and reduced environmental impact, lower start-up and equipment installation costs and lower energy consumption. Despite this appeal and potential benefits, the use of CMA in pothole patching maintenance in Ghana has generally not resulted in a satisfactory patch performance.

A careful observation of most pothole patches that have failed reveal three significant defects types, namely, dishing, shoving and matrix disintegration (see Plates 1.1-1.3).

Plate 1.1. Dishing

Plate 1.2. Shoving

Plate 1.3. Disintegration

In general, exhibition of those three types of defects in patches is suggestive of deficiency in patch mixture quality. Such deficiencies may be traced to aggregate structures which are unable to give the required rigidity and stability, adequate cohesion within the compacted matrix due to insufficiency of binder, poor matrix compaction, high-porosity matrix and a host of other factors associated with the aggregates. In fairness, the quality concerns raised cannot be dissociated from the appropriateness and robustness of the local specifications that guide the formulation of the cold-mix patching material and the level of quality the mixes are able to attain.

Given the convenience, environmental friendliness and low cost associated with CMA production, the use of the material in Ghana is likely to continue for a long time to come. Therefore, to justify its continual use in road maintenance, especially pothole patching, the material must be improved in quality considerably beyond what currently pertains to achieve a more successful product.

One such means of improving bituminous mixtures is the use of wastes, polymers, composites and fillers (Jahromi & Khodaii, 2009; Moussa et al., 2021; Notani et al., 2019). Fillers used in bituminous mixtures have been identified as having the potential to improve mixture properties positively (Abdullah et al., 2015). Nanoclays (pulverized clay particles ranging in size 1-100 nm by nanotechnology) have emerged as the newest modifiers with the potential to improve bituminous mixture properties when incorporated (Abdullah et al., 2016; Abdullah et al., 2016; Ameri et al., 2017; Gardênia et al., 2016; Mansourian et al., 2019; Mohammadiroudbari et al., 2016; Mousavinezhad et al., 2019; Siddig et al., 2018). In April 2012 the Asphalt Institute (AI) published an article titled "Introducing Nanoclay as an Asphalt Modifier" (Walker, 2012). The article gave insights on the improvements associated with using asphalt-modified binders which included the capacity to reduce fatigue and cold temperature cracking and improve rutting resistance, as Ameri et al. (2017) and Yao et al. (2013) also discovered later. These performance improvements have led to widespread use of modified asphalt and have also sprouted various researches into using potential additives as modifiers in asphalt. Also, a team of researchers at Michigan Technological University explored the improvements or otherwise in integrating nanoclay as a filler material in asphalt. The results showed that the nanoclay-modified binders were more viscous or stiffer which is an indication of improved performance of rutting resistance (You et al., 2011). To this end, potential improvements attainable in incorporating nanoclay filler CMA would be explored.

Also, the local specifications available need to be interrogated and reviewed alongside some foreign specifications in countries where CMA patches are performing satisfactorily to establish commonalities and areas of deviations that could impact material quality and performance in the field.

This study, therefore, sought to investigate and then improve the quality of cold-mix asphalt used in bituminous pavement maintenance works in Ghana in order to have a more robust mix that will achieve long-lasting field performance.

1.2 Research Objectives

The main objective of the study was to develop stable and long-lasting cold-mix asphalts for pothole patching. The specific objectives were:

1. To review existing local and other specifications for cold emulsified asphalt mixes for pothole patching.
2. To establish the characteristics of typical cold mix asphalts used by local contractors in pothole maintenance in Ghana.
3. To design a resilient dense-graded cold-mix asphalt with simplified material proportion ratios.
4. To explore the potential improvements in property attainable by incorporating nanoclay filler in cold-mix asphalts.

1.3 Scope of Study

In order to achieve the above stated objectives the research was divided into phases. The first phase involved desk studies to review existing local specification in Ghana, that is, the Ministry of Transportation (MOT) Standard Specification for Road and Bridge Works (2007) alongside other foreign specifications for cold emulsified asphalt mixes for pothole patching.

The second phase of the study involved conducting laboratory investigating such as Asphalt binder extraction test, sieve analysis etc. to establish the properties of some cold asphalt patching mixtures used by contractors for maintenance of roads in the country.

The third phase entailed laboratory investigation to design of a series of dense-graded cold-mix asphalts of Gradation 0/25 mm with simplified material proportioning ratios to aid ease of formulation by contractors in the field.

The final phase also entailed laboratory studies on the incorporation of montmorillonite nanoclay filler in cold-mix asphalt to explore potential improvements in mix properties.

1.4 Justification for Study

No conscious efforts have been made to address the causes of unsuccessful pothole patches in bituminous pavement maintenance works in the country, even though the problem has persisted for a long time. It is hoped that this research would provide answers that may lead to better mix design and ensure long-lasting patch works.

In addition, the results from this research may motivate and help the Ministry of Roads and Highways to develop simple manuals and easy-to-use guidelines for formulating resilient pothole patching mixes that may be used by contractors engaged in such works. The outcome of the study may also be used to provide the necessary training to contactors' technical personnel and field supervisors engaged on maintenance works involving the use of cold-mix asphalts.

On the knowledge front, this study will hopefully expand the scope of current expertise in the country on cold-mix asphalts, serve as a reference material for future research and studies on the subject in the country and provide a spring board for further research on improving the properties and performance of cold-mix asphalts. Finally, in broader perspectives, this study would potentially lead to a better use of limited routine maintenance budget for bituminous road maintenance in the country.

CHAPTER 2: LITERATURE REVIEW

2.1 Constituents and Production of Cold-Mix Asphalt

Cold-mix asphalt (CMA) refers to an asphalt mixture whose main constituents may comprise some or all of the following:

- Liquefied bitumen (cutback asphalt or bitumen emulsion)
- Graded aggregates
- Water
- Additives (Fillers, etc.)

Mixing of the constituents takes place at ambient temperature (Fang *et al*., 2016). Even though the binder in the mix may be any liquefied bitumen, in practice, emulsion mixes are the most preferred and extensively used for the following major reasons (Roberts *et al*., 1996):

- Asphalt emulsions are relatively pollution-free unlike cutback asphalts which cause relatively minute amounts of hydrocarbon volatiles, other than water, to evaporate into the atmosphere.
- When cutback asphalt cures, the petroleum solvents which are high-price products, are wasted into the atmosphere.
- Emulsions are safer to use as there is little danger of the material catching fire when heated compared to cutbacks.
- Emulsions can be applied to damp aggregates and at relatively low temperatures whereas dry conditions are required for cutbacks.

Mix production entails energy savings of up to 95% of that used in comparable hot-mix asphalt (HMA) manufacture (Chehovits and Galehouse, 2010). This benefit derives from the elimination of heating of aggregates during the production of the mix. As such, CMA is seen as a green bituminous mix for road use. The appeal of the material derives from its versatility and reduced environmental impact in its production, lesser start-up and lower energy consumption (Kuhn *et al.*, 2000). The material is particularly suited for laying during road construction in humid or wet conditions and in isolated and remote areas where plant-produced HMA is not feasible.

CMA is a material used extensively for pothole patching as well as partial-depth and sectional repairs in routine maintenance of bituminous roads. The main concerns associated with the use of the material, however, have centered around the high air voids content when compacted, fragile early life strength due to entrapped water in the mix, long curing times due to the low evaporation rate of volatiles/water in the mix and slow setting of the asphalt emulsion to obtain the required performance (Leech, 1994).

2.2 Influence of Aggregate Gradation on Mix Properties

The properties of bituminous mixes, whether hot-mixed or cold-mixed, are influenced by the aggregate in the mixture. In cold patching mixes, aggregate gradation plays a very major role (Biswas *et al.*, 2015) and influences mix stability (AAPA, 2004). Also, open-graded mixes provide adequate space for thick binder films which in turn provide good workability.

Aggregate gradation influences rutting resistance significantly (Oliver *et al.*, 1997) with the shear resistance of the mix being most greatly related to the grading characteristics of the coarse aggregate fraction (Roque *et al.*, 1997). According to Roberts *et al.* (1996) and Ruth *et al.* (2002), dense-graded asphalt mixes with well-balanced and continuous gradation are mostly regarded as generating the utmost permanent deformation resistance for any given quality and grade of aggregates (see Fig. 2.1). That notwithstanding, Diaz (2016) concluded from the performance of fine-grained Superpave mixes that the maximum size of aggregates also influence mix performance.

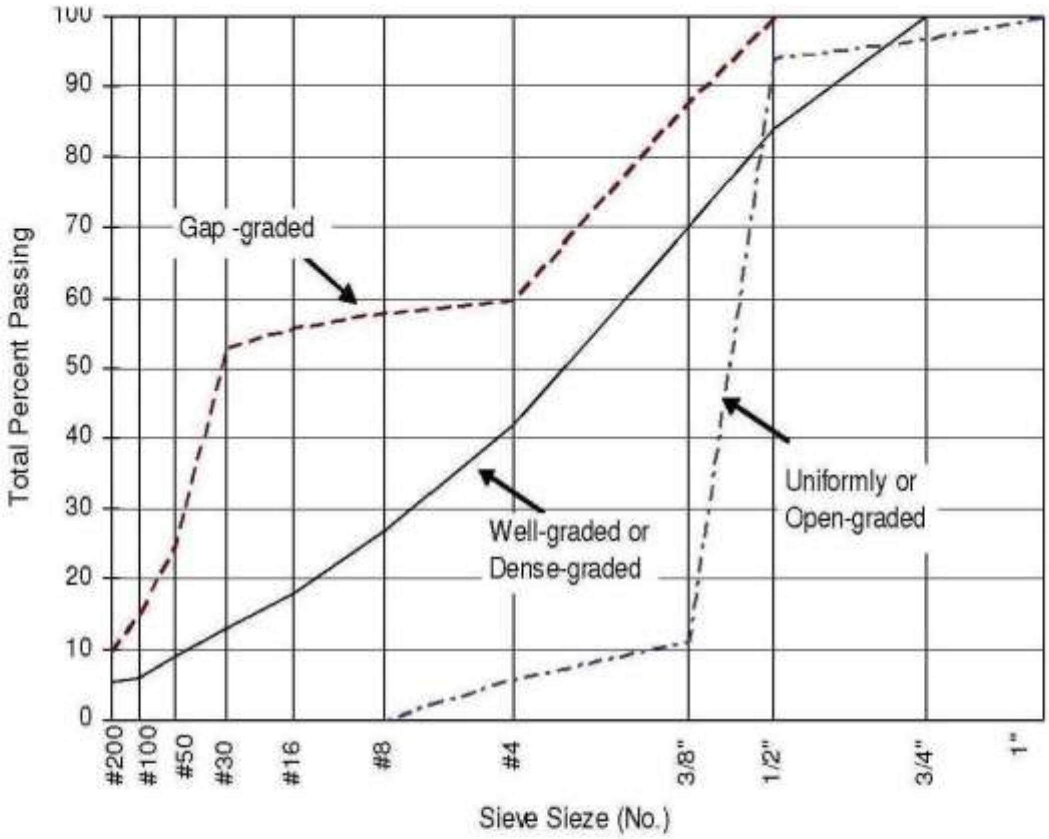


Fig. 2.1. Types of Aggregate Gradation Curves (Lavin, 2003)

A suitable aggregate structure must consist of crushed angular particles and maximum aggregate size not more than 13mm and a maximum of 2% fines (Anderson *et al.*, 1988). The larger particles in dense-graded aggregates help in the development of frictional resistance of the mix to shear failure as well as deformation and are tightly bonded together as a result of the interlocking effect of the smaller particles. Lack of stability can occur when gradations containing excess of particular size fractions are used.

A quantitative parameter that may be used to determine a good representation of different aggregate size ranges (or lack of it) in an aggregate mass or structure is the uniformity coefficient (Cu) defined as;

$$C_u = \frac{D_{60}}{D_{10}} \tag{2.1}$$

where,

C_u=uniformity coefficient (or coefficient of uniformity)

D_{60}=sieve opening size through which 60% of the aggregate sample by mass will pass

D_{10}=effective size, or sieve opening size through which 10% of the aggregate sample by mass will pass.

In practice, D_{60} and D_{10} are obtainable from the particle size distribution chart as the particle size at which the curve crosses the "60% Passing" and "10% Passing" lines, respectively (see Fig. 2.2).

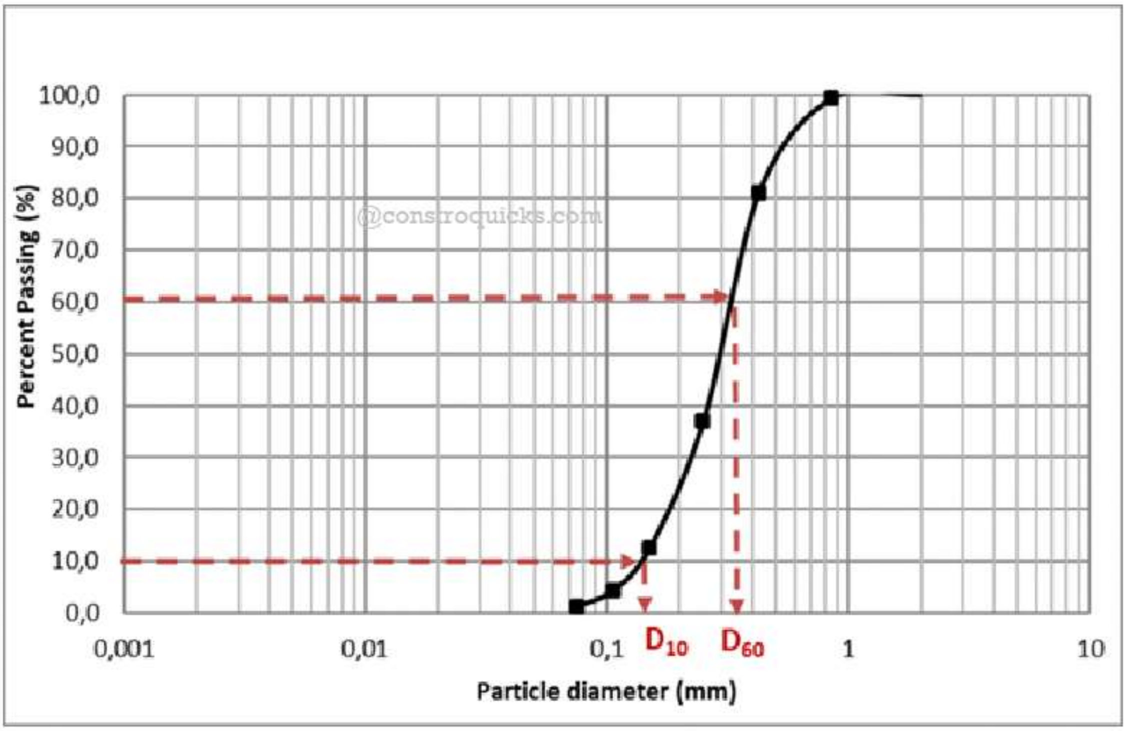


Fig. 2.2. Aggregate gradation curve showing D_{10} and D_{60} (Meininger, 1992)

An aggregate mass that has C_u<4 and approaching unity (the minimum theoretically possible) shows a high degree of particle size uniformity or a structure composed essentially of single-size aggregates. On the other hand, Cu >6 and substantially larger (15 or more) would indicate the presence of a wide range of different particle sizes within the aggregate mass. This would typically be the case of a well-graded or dense-graded aggregate mass.

Dense-graded asphalt mixes may be desirable from the standpoint of maximum stability, however, if higher fraction are mostly well-graded, the required amount of bitumen to provide the required binder coat thickness cannot be accommodated by the voids in mineral aggregate (VMA) since the voids will be too low and hence lead to bleeding. On the other hand, if lower binder content is used, it results in thin binder coat thickness of the aggregate, the asphalt concrete produced would not be durable and may be ravel susceptible (Doyle *et al*., 2013). Consequently, the ultimate aggregate gradation should deviate a little from dense gradation so as to provide sufficient voids space in the mineral aggregates to accommodate the quantity of asphalt cement required for high stability and durability of the asphalt mix.

2.3 Effect of Maximum Aggregate Size on Mix Performance

According to ASTM C125 (ASTM, 2001), maximum aggregate size refers to the smallest sieve aperture through which the whole amount of aggregate passes while nominal maximum size is the smallest sieve aperture that retains not more than 10 percent of the aggregate. More simply put, Superpave mix design system defines nominal maximum size as one sieve size larger than the first sieve to retain more than 10 percent of aggregates, whereas maximum aggregate size is one sieve larger than the nominal maximum size (Fwa, 2005).

According to Evans *et al.* (2006), most cold patching mixes have maximum aggregate size of 9.5mm or 12.5mm. Mixes with larger maximum aggregate size are used for patching deeper potholes due to their enhanced load bearing capability. Using a larger maximum aggregate size also generally decreases the design asphalt content and cost of the mix due to reduced aggregate surface area. However, mixtures formulated with larger aggregate sizes are harder to place and compact to the desired smoothness. When larger aggregates are used in patching shallow holes, they ought to be raked out for a smooth patch finishing thereby affecting the stability of the mixture. The lift thickness also restricts the maximum aggregate size to be used in the asphalt mix. The maximum size is limited to half the lift thickness (Roberts et al, 1991). Therefore, when shallow potholes are to be patched, smaller maximum aggregate size ought to be used and finer mixes are needed when the mix must be feathered at the edges of the hole.

Ideally, two different maximum aggregate sizes of patching mixes should be available for use by the maintenance crew; a portion of aggregates that are of larger sizes to patch deeper holes and another portion that is smaller sized to fill shallow holes.

2.4 Desirable Properties of Asphalt Patching Mixtures

Mixes that will ensure successful and durable patches must have desirable properties, which include the following (Chatterjee *et al*., 2006; Roberts *et al*., 1996):

a) Stability

Stability of an asphalt mix is the ability of the material to resist imposed external loading. A stable asphalt mix maintains the shape and smoothness required under repeated loading. Lack of asphalt mix stability leads to dishing, raveling and shoving under traffic loads (Roberts *et al*., 1996). Structurally, the stability of a mix depends on internal friction and cohesion, with the former derived from the shape and surface texture of the aggregates, while the latter is derived from the bonding ability of the binder. A proper degree of both internal friction and cohesion in asphalt mixes prevents the aggregate particles from sliding over each other by the imposed external loading. Patching mixes with excessive binder content and unwarranted use of medium size sand or rounded aggregate with little or no crushed surfaces tend to have low stability. Such mixes may be characterized by dishing and shoving distresses (see Plate 2.1).

Plate 2.1. Surfacing exhibiting shoving distress

b) Stickiness

Stickiness helps the mix to adhere to the bottom and sides of the pothole which is important when the patching mix is used to repair thin edges. Stickier mixtures are helpful in situations where the maintenance crew does not take time to clean and dry a hole thoroughly so that a proper tack coat could be applied. Stickiness is influenced by the temperature of the mix. Generally, however, hot mixes have relatively better adhesion properties when still hot than cold mixes.

c) Impermeability

This property, is a measure of the resistance of an asphalt mix to the passage of water and air into or through it. Asphalt binder may strip from aggregates, particularly in the presence of water, so it is important to keep water out of mixes to the extent possible (Roberts *et al*., 1996).

If a mixture is liable to stripping, the patch mix may ravel, disintegrate and initiate patch failure. This property is also affected by the type of aggregate and binder. When a water-susceptible mixture is used to patch potholes that are caused by poor water drainage, significant impact of stripping may result. Low binder content may result in thin binder films around the aggregates and cause early aging and raveling. High permeability in patching mixes may be due to low binder content, poor aggregate structure and inadequate compaction. Such mixes may be susceptible to oxidation, brittle fracture and subsequent disintegration (Chatterjee *et al*., 2006).

Durability

Durability measures the ability of a patching mix to resist changes in the binder for the worse due to oxidation and to provide satisfactory resistance to raveling and disintegration under traffic loads (Estakhri and Button, 1995). Higher binder contents increase durability because it results in thick binder films around aggregates which in turn impede oxidative aging, so the binder is able to retain its original characteristics for a longer period (Chatterjee *et al*., 2006). In effect, high binder contents efficiently seals off a higher proportion of interconnected air voids in the asphalt mix, rendering the penetration of air and water difficult. A required percentage of air voids, though, is needed to remain in the asphalt mix to accommodate the expansion of the binder in hot weather. Dense-graded tough and sound aggregate has the propensity to contribute to asphalt mix durability since it provides closer contact between aggregate particles there by enhancing the impermeability of the asphalt mix (Roberts *et al*., 1996).

d) Workability

This property defines the ease associated with the placing and subsequent compaction of an asphalt mix. Workability is generally improved by altering design mix parameters, source of aggregate, and/or gradation (Chatterjee *et al*., 2006). An asphalt mix that contains a high fractions of coarse aggregates is likely to segregate during handling and subsequent compaction may be difficult. Mixes with high fractions of fines, depending on the characteristics of the fines, may render the asphalt mix to become sticky or tough, and make compaction difficult (Roberts *et al*, 1996)

e) Storageability

A stockpile of patching mix is expected to remain workable when stored over a desired period of time (6 to 12 months). Mixes that do not contain the right type of binder, tend to loose volatiles very rapidly becoming harder with time. Using a polyethylene or tarpaulin to cover the stockpile prolongs storage time. The mix should not contain too much asphalt binder, which may drain and settle at the bottom of the stockpile (Chatterjee *et al*., 2006).

2.5 Problems associated with Cold Patching Mixes

Premature failure or poor performance of asphalt paving mixes may usually initiate from either the stockpile, or during handling and placement, or in service. A summary of some of the problems and their possible causes (Anderson *et al*., 1988; Estakhri and Button, 1995) as cited in Chatterjee *et al*. (2006) are listed in Table 2.1.

Table 2.1. Cold Patching Mix problems and probable causes (Anderson *et al*, 1988)

Problem	Probable Causes
In Stockpile	
Hard to work	Binder too stiff; too many fines in aggregate or mix too coarse, dirty aggregate
Binder drains to bottom of pile	Binder too soft; stockpiled or mixed at high temperature
Loss of coating in stockpile	Stripping; inadequate coating during mixing; cold or wet aggregate
Lumps-premature hardening	Binder cures prematurely
Mix too stiff in cold weather	Binder too stiff for climate; temperature susceptibility of excessive binder; too many fines in aggregate, dirty aggregate; mix too coarse or fine
During Placement	
Too hard to shovel	Binder too stiff; too many fines, dirty aggregate; mix too coarse or too fine
Softens excessively upon heating (when used with hot box)	Binder too soft
Hard to compact (appears tender during compaction)	Insufficient mix stability; too much binder; insufficient voids in mineral aggregate; poor aggregate interlock; binder too soft
Hard to compact (appears stiff during compaction)	Binder too stiff; excess fines; improper gradation; Harsh mix-aggregate surface texture and shape
In Service	
Pushing, shoving	Poor compaction; binder too soft; too much binder;

Table 2.1 (Continued)

Dishing	Poor compaction; mixture compacts under traffic
Raveling	Poor compaction; binder too soft; poor cohesion in mix; poor aggregate interlock; stripping; absorption of binder by aggregate; excessive fines, dirty aggregate; aggregate gradation too fine or too coarse
Freeze-thaw deterioration	Mix too permeable; poor cohesion in mix; stripping
Poor skid resistance	Excessive binder; loss of texture due to polishing of aggregates; gradation too dense

2.6 Use of Fillers in Cold Emulsified Asphalt Mixes

Fillers used in asphalt mixes appear to have a very intricate role. It may be active or inert depending on its surface characteristics and fineness. The percentage of fillers in asphalt mixes largely depends on the gradation and type of mix, typically ranging between 4% to 5% in dense-graded mixtures and 10% to 15% in open graded mixtures (Thanaya, 2003). Reactive fillers react when they come into contact with, say, bitumen emulsion whereas non-reactive fillers will not.

Mineral fillers possess a wider range of particle sizes that can significantly influence asphalt mix properties. Fillers with the size of their particles larger than the bitumen film thickness (from about 10 to 100 microns) usually tend to improve the interlocking behavior of the compacted aggregates, whereas fillers with particles that are finer than the bitumen film thickness mostly suspend in the binder constituting part of the bitumen in the mix (Abdullah et al., 2016; Mastoras et al., 2021).

With regards to particle sizes, fillers such as limestone appear to have relatively larger sizes (3500 nanometers) compared to others like carbon black with a diameter between 100 to 150 nanometers (nm) that is suspended and embedded in bitumen films when used. Such fillers, like carbon blacks are referred to as “micro fillers” (Hans *et al*., 2019).

The influence of mineral fillers on binder in bituminous mixes is to increase binder viscosity (Harvey, 2000), particularly if the filler has a large surface area. Binders that have high filler contents are less temperature sensitive (Choudhary *et al*., 2020). When compacted, mixes with fillers that are smooth, slippery, and spherical require less compaction effort. In comparison fillers with angular and rough surface texture generates friction resistance when being compacted (Moeini *et al*., 2020).

Thanaya *et al.* (2009) found that partially replacing limestone dust with Portland cement or hydrated lime could improve rutting resistance. Similar results have been obtained by Maher (2000) for calcite and quartz fillers. In general, fillers tend to improve the properties of asphalt mixtures in diverse ways and, therefore, a given filler type may under-perform in some aspects but perform better than others for a particular property. Hence, a specific study is essential on each filler type to find the best outcomes.

2.7 Use of Nanoclay Fillers in Asphalt Mixtures

Nanoclays are tiny particles of layered mineral silicates which include bentonite, montmorillonite and kaolinite (Aragao *et al*., 2011). These materials may be used as asphalt modifiers (Prowell and Franklin, 2020). Generally, asphalt mixes formulated with modified binders tend to be characterized by reduced fatigue cracking and improved rutting resistance (Ashish & Singh, 2021).

Even though nanoclay-modified asphalt is in its early stages of research (Gardênia et al., 2016), there is some degree of optimism that it could improve heat resistance, mechanical properties and biodegradability of hybrid materials (Abdullah *et al*., 2016; Ameri *et al*., 2017; Mansourian *et al*., 2019; Mohammadiroudbari *et al*., 2016; Mousavinezhad *et al*., 2019; Siddig *et al*., 2018). It has already been established by Thanaya *et al.* (2009) that nanoclay in bituminous mixes improves properties such as resilient modulus, stability, indirect tensile strength and results in better performance compared to the unmodified bituminous mixtures under dynamic creep. However, it is also known that the use of the material does not appear to have a favorable effect on fatigue behavior nor thermal cracking at low temperature (Wilson and Romine, 1993).

Traditionally, clay has been used as an emulsifier, but the new trend in clay application in mixes is to use nanoclay to modify asphalt residue properties. Nanoclay-stabilized emulsion could be produced by dispersing the nanoclay in water and then mixing with hot asphalt to form an emulsion with tiny (10-12 micron) particles of asphalt surrounded by tinier clay particles (AEMC,2010). This mixing is done at 100^{o}C, making the emulsion easy to handle. Eventually, after application when the emulsion dries up, it leaves a residue comprising an asphalt and clay with significantly altered properties (AEMC, 2010).

2.8 Basic Considerations in Cold-Mix Asphalt Design

Xiao (2009) and Estakhri and Button (1995) are convinced that the following factors with their respective summaries outline pertinent details to be considered when designing cold bituminous mixes;

Binder consistency

Hard binders would usually not enhance proper coating which could possibly result in bituminous mix that are too stiff, making the mix hard to shovel and compact at ambient temperatures. On the other hand, mixes formulated with soft bitumen are most likely to experience stripping in stockpile, tenderness during placement and compaction as well as rutting, shoving, bleeding and loss of skid resistance in service.

Binder content and Anti-stripping additive

The binder content in the mix has to be maximized to improve workability although excess amount would cause drain-down in stockpile, bleeding, rutting and shoving. Mixes with insufficient binder would exhibit poor cohesion and be susceptible to moisture damage, raveling and eventually suffer matrix disintegration. If added in the right proportion, anti-stripping additives reduce moisture damage (Estakhri and Button, 1995).

Aggregate shape and texture

Rough surfaced and angular aggregates are highly resistant to shoving and rutting but hard to work with. Smooth and rounded aggregates enhance workability but are less resistant to shoving and rutting.

Aggregate gradation

For mixes with fines content not exceeding 2%, mix workability is enhanced while for those with coarse aggregate fractions greater than 25mm in size, spreading and shovelling becomes difficult (Xiao, 2009). Curing is very rapid in mixes that are are open-graded while stability is enhanced if mixes have well-graded aggregates. Dense gradation mixtures must be carefully considered to provide more voids to accommodate binder, thus potentially reducing the tendency for bleeding to enhance durability.

2.9 Procedures for Design of Cold-Mix Asphalts

A major aim of the design of cold-mixes is to optimize the water and bitumen emulsion content for the aggregates in the blend (Kandhal, 1981). The water is used to wet the surface of the aggregates while the emulsified bitumen is used to coat the aggregates. For the study at hand, four different procedures for cold mix design were reviewed:

1. Ministry of Transportation Standard Specification for Road & Bridge Works Ghana (2007).
2. Asphalt Institute Manual Series MS-19 (1997)
3. The Chevron USA Incorporated Procedure (1977)
4. The Nikolaides Specification (1994)

1. Ministry of Transportation Standard Specification for Road & Bridge Works Ghana (2007)

(a) Aggregate Strength Requirements for Cold Mix Asphalt

According to the specification, coarse aggregate (material retained on 4.75 mm sieve) to be used shall comprise of crushed stone produced from boulders or rocks, the smallest size of which is at least 4 times the maximum size of the final crushed aggregate. It is expected that, the fine and coarse aggregates shall be free from silt, organic matter, clay and other deleterious elements. Table 2.2 spells out the strength requirements which the aggregates are to meet. Material inferior to Aggregate Class C shall be rejected.

Table 2.2: Aggregate strength requirement (MOT, 2007)

Coarse Aggregate (size greater than 4.75 mm)				
Aggregate Class A B C		A	B	C
LAA	Max	30	35	40
FI (%)	Max	20	25	30
10% Fines Dry (kN)	Min	160	160	160
Wet/Dry ratio	Min	0.75	0.75	0.75

b) Aggregate Gradation for Cold Asphalt

Section 17.5.3 of the Specifications further specifies that aggregate used in the mix is expected to be within and approximately parallel to the gradation requirement envelope to be specified in the Special Specification supplied by the Engineer.

(c) Binder Content

Section 17.5.4 of the Specification states that the proportion, by mass of total mix, of bitumen shall be stated in the Special Specification. This shall be referred to as the nominal binder content and it is expected that the Engineer provides this information following site trials and laboratory tests.

(d) Density and Stability of Cold Mixes

Section 17.5.4 of the Specifications further specifies that the density and stability of the cold-mix asphalt shall be as specified in the Special Specification or as instructed by the Engineer.

(e) Compaction of Cold Asphalt

Section 17.5.6 specifies that the average density of the 100 mm diameter cores cut from the cold asphalt shall not be less than 98% of the average density obtained from Marshall Specimen (2 x 50 blows) made during laboratory tests on the mix used for site trials. No individual density shall be below 95% of the average of the laboratory determined density.

(f) Curing of Mix

The Specification is silent on the time allowed for the cold-mix asphalt to cure after placement before traffic use.

2. Asphalt Institute's Manual Series MS-19 (1997)

This design procedure/specification basically consists of the following steps;

a) Aggregate Gradation

The Asphalt Institute distinctly specifies two gradation categories: **Dense Gradation** and **Open Gradation.** Table 2.3 provides the requirements for dense gradation for five nominal maximum aggregate sizes.

Table 2.3. Aggregate gradation for dense-graded emulsion mixtures (Asphalt Institute Manual Series MS-19, 1997)

Sieve Sizes (mm)	% Passing				
	Nominal Maximum Size (mm)				
	37.5	**25**	**19**	**12.5**	**9.5**
50	-	-	-	-	-
37.5	100	100	-	-	-
25	90 – 100	90 – 100	100	-	-
19	60 – 80	-	90 – 100	100	-
12.5	-	60 – 80	-	90 – 100	100
9.5	-	-	60 – 80	-	90 – 100
4.75	20 – 55	25 – 60	35 – 65	45 – 70	60 – 80
2.36	10 – 40	15 – 45	20 – 50	25 – 55	35 – 65
1.18	-	-	-	-	-
0.600	-	-	-	-	-
0.300	2 – 16	3 – 18	3 – 20	5 – 20	6 – 25
0.150	-	-	-	-	-
0.075	0 - 5	1 - 7	2 - 8	2 - 9	2 - 10

Similarly, Table 2.4 details the gradation requirements for open-graded cold mixes for four nominal maximum aggregate sizes.

Table 2.4: Aggregate gradation for open-graded emulsion mixtures (Asphalt Institute Manual Series MS-19, 1997)

Sieve Size (mm)	**% Passing**			
	Nominal Maximum Size (mm)			
	25	**19**	**12.5**	**9.5**
37.5	100	-	-	-
25	95 – 100	100	-	-
19	-	90 – 100	-	-
12.5	25 – 60	-	100	-
9.5	-	20 – 55	85 – 100	100
4.75	0 – 10	0 – 10	-	30 – 50
2.36	0 – 5	0 - 5	0 - 10	5 – 15
1.18	-	-	0 - 5	-
0.075	0 - 2	0 - 2	0 - 2	0 – 2

The Asphalt Institute MS-19 further details the following requirements to be met as shown in Table 2.5.

Table 2.5: Emulsified asphalt aggregate mixture design (Asphalt Institute Manual Series MS-19, 1997)

Test Property	Minimum	Maximum
Soaked Stability at 25±1ºC (kN)	2.224	-
LAA (%)	-	40
Aggregate Coating (%)	85	-

b) Determination of Initial Residual Asphalt Content (IRAC) and the Initial Emulsion Content (IEC)

Once the gradation type to be used is established, the Initial Residual Asphalt Content is calculated utilizing the empirical formula below, referred to as the Centrifuge Kerosene Equivalent Method.

$$P = (0.05\,A + 0.1\,B + 0.5\,C) \times (0.7) \qquad (2.2)$$

where,

P = Percentage of Initial Residual Asphalt Content by mass of total mixture,

A = Percentage of aggregate retained on Sieve 2.36 mm,

B = Percentage of aggregate passing Sieve 2.36 mm and retained on Sieve 0.075 mm, and

C = Percentage of aggregate passing Sieve 0.075 mm.

The initial emulsion content is evaluated as;

$$IEC\,(\%) = 100\frac{P_b}{X} \qquad (2.3)$$

where,

IEC = Initial Emulsion Content (% by mass of total mixture)

P_b = Initial Residual Asphalt Content (in %) by mass of total mixture

X= % bitumen residue in emulsion.

c) Coating Test

The coating is carried out by utilizing the IEC value. This is done by mixing all batches of dry aggregates and filler and then pre-wetting with varied amounts of water. The bitumen emulsion is then added and mixed for about two to three minutes for uniform coating to be achieved. The optimum pre-wetting water content (OPWwc), which gives the best bitumen coating on the aggregates, can then be determined. By visual observation, the degree of coating should not be less than 85%.

d) Determination of Optimum Residual Asphalt Content (ORAC)

The Optimum Residual Asphalt Content is determined by the optimization of parameters such as air voids, flow value, soaked stability for soaked samples for all residual asphalt content variations. Even though all other requirements should meet CMA design requirements, the main parameter is the minimum soaked stability.

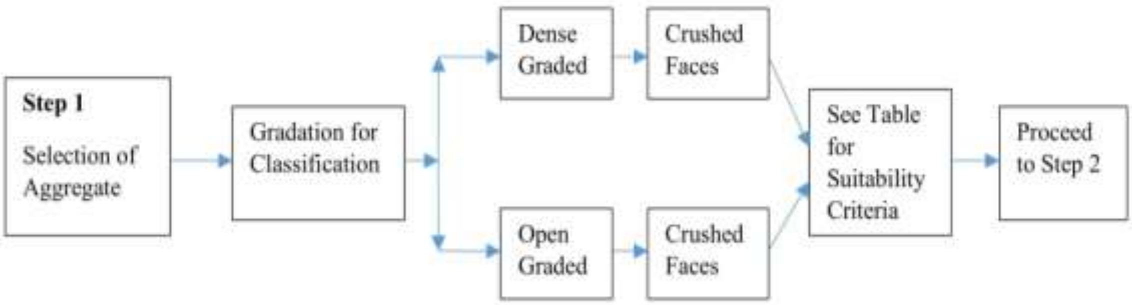
3. The Chevron USA Incorporated Procedure (1977)

The Chevron Method for Cold-Mix Asphalt covers the selection, proportioning, selection and testing of emulsified asphalt mixtures.

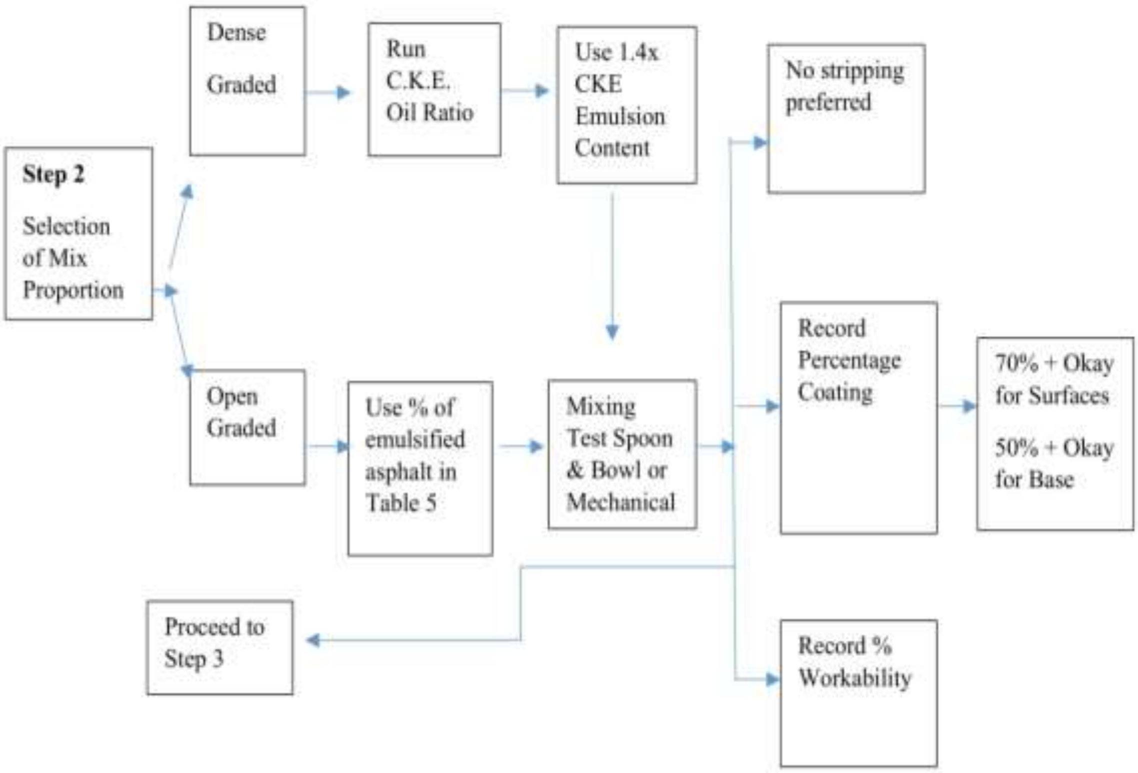
Testing schedule for Emulsified Asphalt Mixtures

The Chevron procedure follows the following steps:

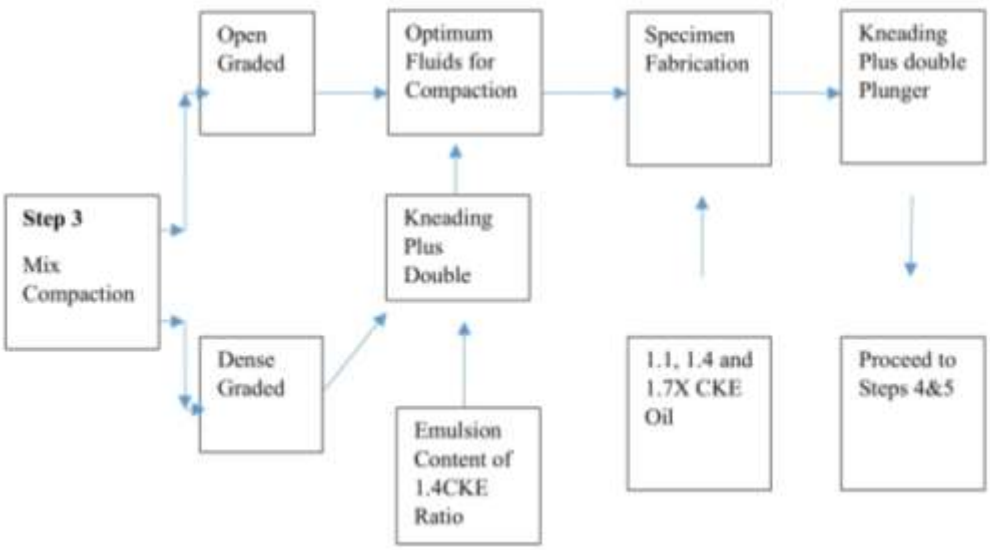
Step 1: Selection of Aggregates



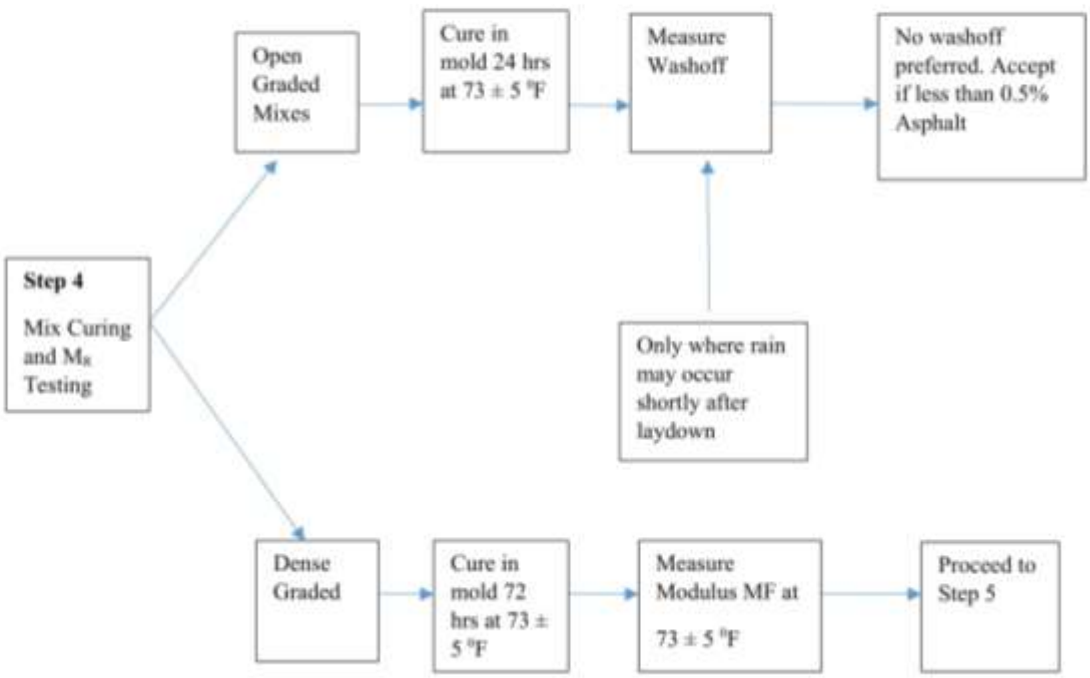
Step 2: Selection of Mix Proportion



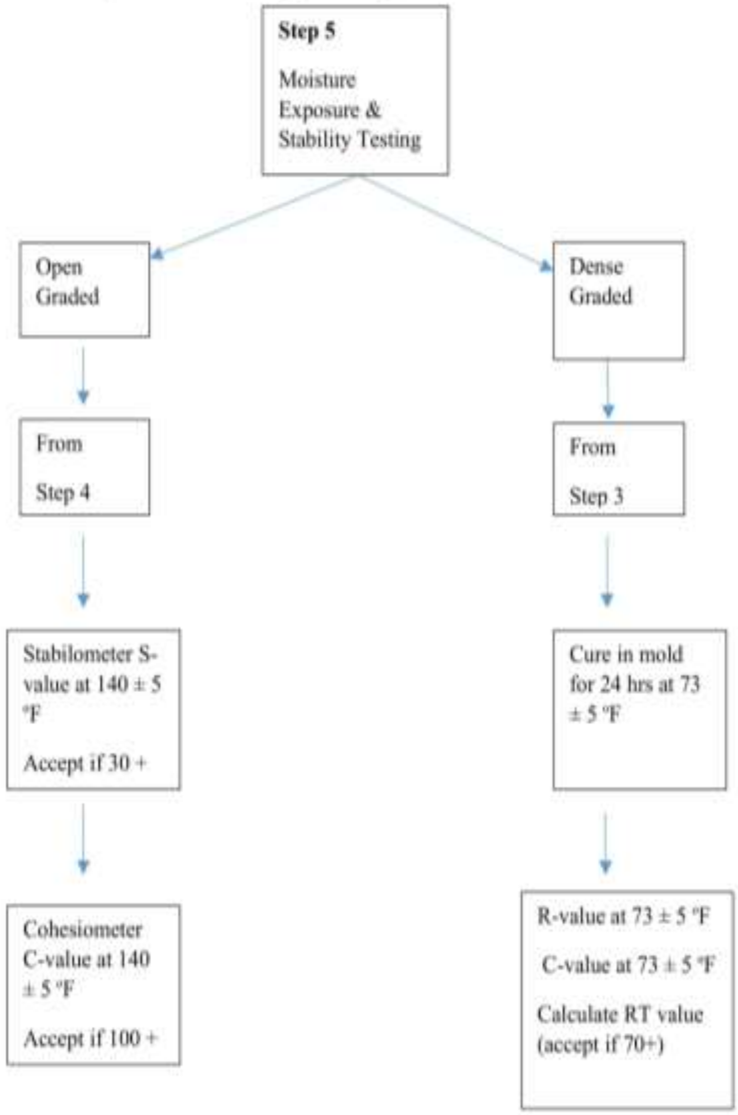
Step 3: Mix Compaction



Step 4: Mix Curing & Testing



Step 5: Moisture Exposure & Stability Testing



4. The Nikolaides Specification (1994)

Nikolaides Specification covers nine gradation types for cold dense-graded emulsion mixtures and provides the nominal bitumen content for each gradation regime as well as the minimum acceptable stability, coating and void content. The mixture gradation types and properties are given in Table 2.6.

Table 2.6: Gradation and Mixture Property Requirements (Nikolaides Specification, 1994)

Sieve Size (mm)	Percent Passing				
	Type 1	Type 2	Type 3	Type 4	Type 5
50	100	-	-	-	-
38.10	90 – 100	-	-	-	-
25.40	-	90 – 100	100	-	-
19.10	60 – 80	-	90 – 100	100	-
12.70	-	60 – 80	-	90 – 100	100
9.52	-	-	60 – 80	-	90 – 100
4.76	20 – 55	25 – 60	35 – 65	45 – 75	60 – 80
2.36	10 – 40	15 – 45	20 – 50	25 – 55	35 – 65
0.60	-	-	-	-	-
0.30	4 – 15	5 – 18	5 – 20	5 – 20	5 – 25
0.074	0 – 5	0 - 5	-	3 - 8	3 – 11
Nominal Bitumen Content (%)	4	4.5	5	5.5	6
Soaked Stability (N)	1335	1335	1335	1335	1335
Min. Retained Stability after 48hrs soaking (%)	50	50	50	50	50
Min. Total Void Content (%)	6	6	6	6	6
Max. Total Void Content (%)	12	12	12	12	12
Max. Water Absorption after 48 hrs soaking (%)	4	4	4	4	4
Bitumen Film Thickness (µm)	6	6	6	6	6
Min. Degree of Coating (%)	85	85	85	85	85

Table 2.6 (Continued)

Sieve Size (mm)	Percent Passing			
	Type 6	Type 7	Type 8	Type 9
50	-	-	-	-
38.10	-	100	-	-
25.40	-	90 – 100	100	100
19.10	-	65 – 98	97 – 100	97 – 100
12.70	100	50 – 75	70 – 100	77 – 98
9.52	-	40 – 60	56 – 80	67 – 85
4.76	75 – 100	35 – 50	50 – 60	60 – 70
2.36	60 – 80	30 – 48	45 – 60	55 – 70
0.60	-	10 – 45	15 – 60	18 – 70
0.30	15 – 65	-	-	-
0.074	5 – 20	3 - 6	3 – 8	3 - 8
Nominal Bitumen Content (%)	6.5	6.5	6	5.5
Soaked Stability (N)	1110	1335	1335	1335
Min. Retained Stability after 48hr soaking (%)	50	50	50	50
Min. Total Void Content (%)	6	6	6	6
Max. Total Void Content (%)	12	12	12	12
Max. Water Absorption after 48 hrs soaking (%)	4	4	4	4
Bitumen Film Thickness (μm)	6	6	6	6
Min. Degree of Coating (%)	85	85	85	85

2.10 Common Tests for Cold Emulsified Asphalt Mixes

Most of the common tests conducted on cold-mixes are meant to ensure that the mixes have adequate mechanical strength in terms of stiffness or stability to provide load spreading ability, adequate fatigue cracking resistance and ability to resist excessive deformation. Tests performed for mechanical properties commonly include the following:

a) Marshall Stability and Flow Test ASTM D6927 (2015)

The test for stability and flow on cold-mix asphalt utilizes the Marshall Stability apparatus for testing hot bituminous mixtures. The compacted specimens are conditioned for thirty minutes in a water bath maintained at a temperature of $60^{o}C$. At the end of the conditioning period, the specimen is removed from the bath and carefully blotted dry, and positioned in the testing head. A compressive load is applied at a constant rate until failure. The load that causes the specimens to fail is the Marshall stability and the deformation from the start until specimen failure is the flow.

b) Stiffness Test ASTM D4123 (1997)

The stiffness of a compacted bituminous mixture indicates the load spreading ability expressed as the elastic modulus. The property can be subdivided as a viscous stiffness and elastic stiffness. Elastic stiffness is obtained at low temperatures when specimen is subjected to short loading. On the contrary, viscous stiffness is tested at longer loading times at higher temperatures and is fundamentally used for assessing resistance to permanent deformation. The mechanical behavior of asphalt mixtures is considered to be visco-elastic and depends on both temperature and loading time. According to Usman *et al.* (2021) in visco-elastic materials, with any stress application the resultant strain is out of phase. That is, applied stress lags behind the strain by a parameter known as the phase angle. A typical visco-elastic response to a low pulse in asphalt mixtures is shown in Fig. 2.2.

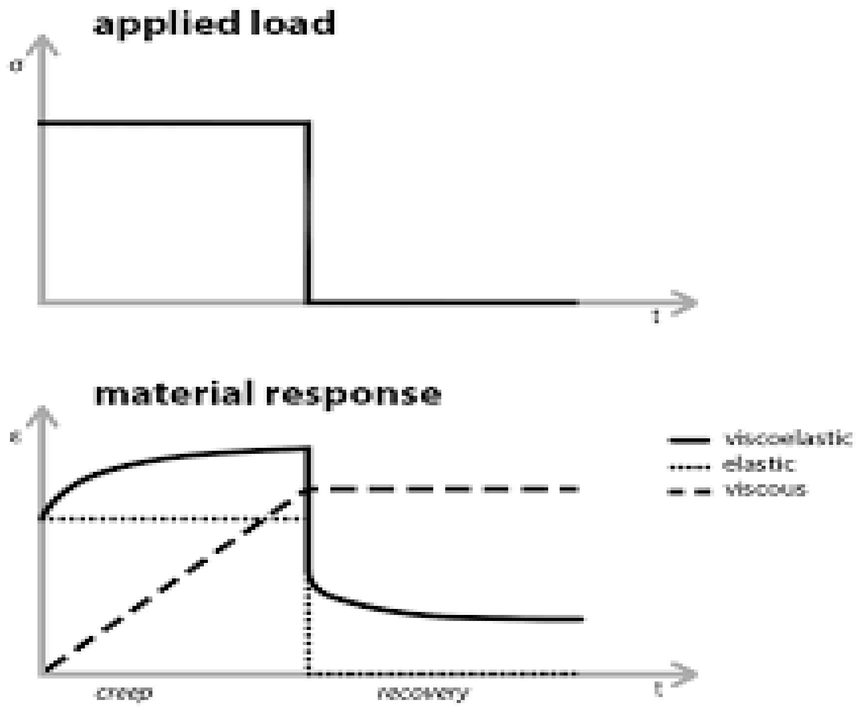


Fig. 2.2 Visco-elastic response to loading (Read and Collop, 1997)

c) Indirect Tensile Stiffness Modulus (ITSM) Test ASTM 4123-82 (1997)

Bituminous mixtures are more sensitive to tensile stresses than to compressive stresses and, therefore, it is the tensile stiffness which is routinely measured. Testing bituminous samples in an indirect tensile mode is more practicable than direct tension tests. This stiffness test is used for quantifying minor recoverable strains on bituminous specimens and consists of a set of pulse loadings applied on two diametrically opposed generating lines of a cylindrical specimen.

The current method of specifying the indirect tensile stiffness modulus is based on British Standard Draft for Development BS DD213: 1993, Method for determination of the indirect tensile stiffness modulus of bituminous mixtures. The indirect tensile stiffness modulus (S_m) is calculated using the equation below:

$$S_m = \frac{L(\mu + 0.27)}{tD} \tag{2.4}$$

where,

S_m =indirect tensile stiffness modulus (MPa),

L =peak value of the applied vertical load (N)

D = mean amplitude of the horizontal deformation obtained from 2 or more applications of the load pulse (mm)

t = mean thickness of the test specimen (mm)

μ = Poisson's ratio

d) Deformation Resistance Test ASTM 4535 (1997)

The resistance of bituminous mixtures to deformation largely relates to the stiffness of the mixture, mostly influenced by the loading and temperature. Factors such as binder grade, aggregate gradation, degree of interlock, aggregate shape and texture also have an influence on stiffness. The unconfined uniaxial creep test is used to test the resistance to deformation of bituminous mixtures. Fig. 2.3 is a setup for creep test (Gandi *et al*., 2019).

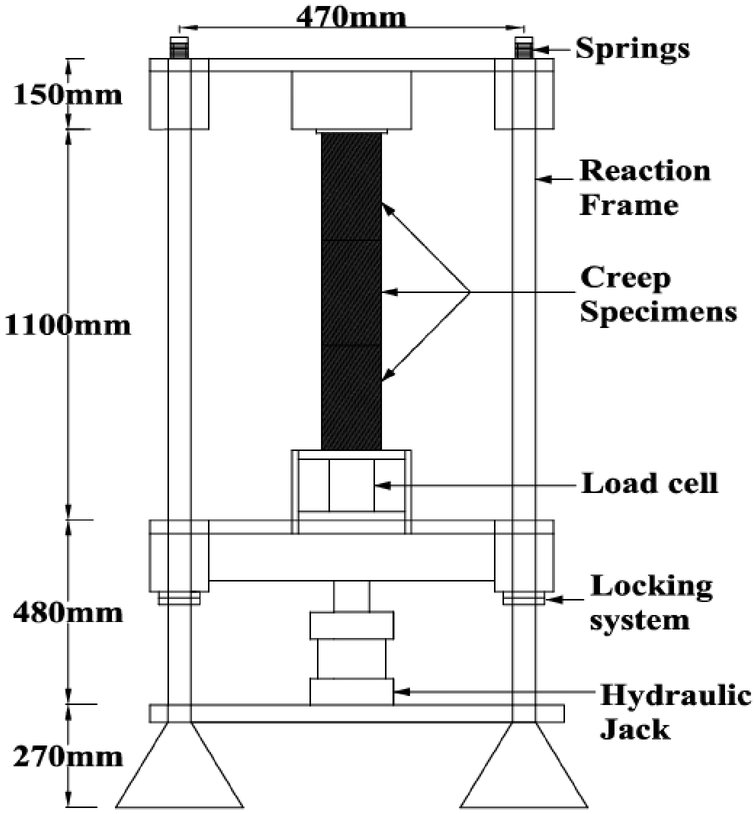


Fig. 2.3 Schematic representation of the creep test configuration (Gandi et al, 2019)

e) Static Creep Test ASTM 4680 (1997)

This test involves subjecting a cylindrical sample to a static axial stress (100KPa) at a loading time of one hour, whiles simultaneously measuring the axial deformation. On completion of the test, adequate recovery time (at least 15 minutes) is allowed during which sample recovery is observed. The mixture stiffness (S_{mix}) at any loading time is then determined from the axial strain per the following relationship:

$$S_{mix} = \frac{\delta}{\varepsilon} \qquad (2.5)$$

where,

δ = applied stress

ε = axial strain

The mixture stiffness values obtained from static loading provides an avenue for creep performance comparison of various bituminous mixtures to give a foresight of the long term performance prediction of these mixtures.

f) Fatigue Test (Indirect Tensile Fatigue Test) ASTM D8237 (1997)

Fatigue is a phenomenon of fracture associated with repeated load whose magnitude may be less than the material's tensile strength (Gandi *et al.*, 2019). Fatigue life of bituminous mixtures depends on the loading frequency, stress magnitude and duration of rest period in between load applications. Cooper and Pell (1994) established that, the main factor that affects fatigue performance includes binder type and content. Fatigue life is increased when binder content increases. However, porosity reduction as a result of compaction and aggregate gradation modification have no significant effect on fatigue life (Rogue and Buttlar, 1992).

Studies by Hasanuzzaman *et al.* (2017) on the influences aggregate gradation has on asphalt concrete mixtures established that mixtures with finer gradation exhibited better fatigue performance than those with coarser gradation while harder binders performed much better in thicker pavements. In the laboratory, fatigue life is obtained using the relationship;

$$N_f = c\left(\frac{1}{\varepsilon_t}\right)^m \tag{2.6}$$

N_f =number of load applications to initiate a fatigue line.

ε_t = maximum value of applied tensile strain

m =slope of strain – fatigue line

c = factor dependent on the composition properties

The Indirect Tensile Fatigue Test (ITFT) is a typical test set up as shown in Plate 2.2 is used for measuring the fatigue life of a bituminous mixture.

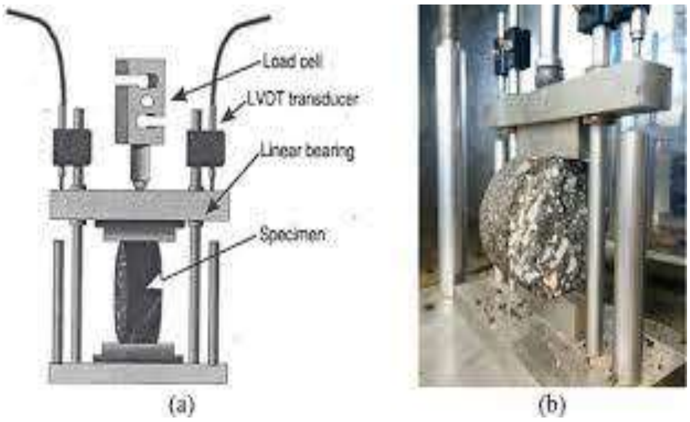


Plate 2.2 Indirect Tensile Fatigue Test set up (Thanaya, 1997)

In the Indirect Tensile Fatigue Test, a cylindrical specimen is subjected to loading vertically diametrically which, in turn, produces a stress along the horizontal diameter. The test is able to give the fatigue life characteristics of bituminous mixtures by testing few specimens, usually less than 10 specimens at high stress levels (greater than 450kPa) at high temperatures (above 25°C). The load applied vertically generates both a horizontal tensile stress and a vertical compressive stress along the diameters of the tested specimen. The stress strain condition is calculated by the following assumption;

1. The Poisson's Ratio (υ) for the material is known.

2. The cylindrical specimen is subjected to a stress $\delta z = 0$

3. The material specimen is isotropic, homogenous and behaves in a linear elastic manner.

4. A force (P) is applied.

After assuming the conditions above, the maximum horizontal tensile stress δmax and strain ε_{xmax} at the center of the specimen is calculated as follows;

$$\delta_{x\max} = \frac{2P}{\pi Dt} \tag{2.7}$$

$$\varepsilon_{x\max} = \frac{\delta_{x\max}(1+3\mu)1000}{S_{mix}} \tag{2.8}$$

where,

δ_{xmax} = maximum horizontal tensile stress at the center of specimen (kPa)

ε_{xmax} = maximum initial horizontal tensile strain at the center of the specimen

S_{mix} = the indirect tensile stiffness modulus (MPa)

D= diameter of test specimen (mm)

t = thickness of test specimen (mm)

P = applied vertical force (kN)

μ= Poisson’s ratio (assumed to be 0.35)

g) Durability Test

Bituminous mixtures are considered durable when they are able to endure the effects of variations in environmental conditions, such as water, ageing and temperature variations without significant deterioration when subjected to a given amount of traffic loading over an extended period of time (Yang *et al.*, 2021). Generally, it is assumed that age-hardening and moisture damage are the primary factors that affect durability of asphalt mixtures. Water can damage the structural integrity of bituminous mixtures in two ways. Firstly, it can cause stiffness of the bitumen and loss of cohesion. Secondly, it weakens the adhesive bond between the aggregate and bitumen, causing the mixture to strip.

Mixtures that are permeable to water are mostly susceptible to stripping. Therefore, the lower the air content in the mixture when compacted, the less the risk of bitumen detaching from the aggregate (Chen *et al.*, 2016). Short- term aging occurs mostly during the mixing as well as the construction stages, while long-term-aging occurs in service through chemical oxidation. When oxidized, the bitumen gradually increases in viscosity becoming harder and less flexible. Oxidation of bitumen is influenced by factors such as temperature, bitumen film thickness, loss of volatiles and time. The loss of volatile components of the mixture also depends on the exposure conditions and temperature.

CHAPTER 3: METHODOLOGY

3.1 Introduction

The procedure for field and laboratory work in this research was placed into three phases.

(a) Phase 1: Evaluation of typical cold-mix asphalts used by local contractors

This phase entailed sampling of stockpiled ready-to-use cold-mix asphalts for patching at contractor sites and subjecting the samples to a series of laboratory tests. Additionally, patch materials from failed patches were also taken from the road and subjected to laboratory testing.

(b) Phase 2: Design of dense-graded cold-mix asphalts

This phase entailed the design of a series of dense-graded cold-mix asphalts of Gradation 0/25 mm with simplified material proportioning ratios to aid formulation by contractors in the field.

(c) Phase 3: Evaluation of cold-mix asphalts containing nanoclay filler

This phase entailed the incorporation of nanoclay (pulverized clay particles with a size range of 1nm-100nm) in cold-mix asphalt as filler material to explore potential improvements in mix properties.

3.2 Evaluation of cold-mix asphalts used by local contractors

Representative stockpiled ready-to-use patching mixes were sampled from six contractor sites across three regions in Ghana to the laboratory for analysis. The samples were given alpha-numeric identities, with the alphabet tied to the name of the region as follows; A1 and A2 for samples from the Ashanti Region, B1 and B2 for samples from the Bono Region and V1 and V2 for samples from the Volta Region. In addition, one sample each was taken from failed road patches in the three regions and identified as A3 for the sample from Ashanti, B3 for the sample from Bono and V3 for the sample from Volta to the laboratory for analysis. The failure modes observed included dishing, shoving and failure by matrix disintegration.

All the samples were subjected to the following tests in the laboratory:

- Bitumen Content Test (Solvent Extraction Method) ASTM D2172
- Aggregate Gradation Test ASTM C-117
- Stability and Flow Test ASTM D1559

a) Bitumen Content Test (Solvent Extraction Method) ASTM D2172

The asphalt contents of the stockpile mixes were determined by solvent extraction per ASTM D2172; "Standard Test Methods for Quantitative Extraction of Bitumen from Bituminous Paving Mixtures". Test samples were heated in an oven to 110 ± 5^{o}C until they were soft enough to be separated with a trowel. A representative sample was quartered and oven-dried at a temperature of 110 ± 5^{o}C to constant weight. The sample was taken out and allowed to cool. A weighed quantity was placed in a centrifuge extractor and trichloroethylene (TCE) was added for the extraction of the binder, a filter paper is then placed at the mouth of the centrifuge extractor and covered. Addition of TCE was continued until the effluent was clean. The aggregates obtained from the extraction test were oven-dried to a constant mass in an oven at 110 ± 5^{o}C. The asphalt content was then calculated as follows;

Asphalt Content (%) = $\frac{W1-\ (W2+W3)}{W1}$

where,

W1 = Weight of mix taken before extraction (g)

W2 = Weight of mix after extraction (g)

W3 = Weight of filler collected in filter paper (g)

B = Weight of filter paper before extraction (g)

D = Weight of filter paper after extraction (g)
W3 = B – D (g)

b) Aggregate Gradation Test ASTM C-117

The dried aggregates from the extraction tests were used for sieve analysis. The weighed dry sample was placed on the topmost sieve and the entire nest of sieves of required sizes were placed in a mechanical sieve vibrator to allow the sample to separate through the nest of sieves. The mass of aggregates retained on each sieve was determined and the percentage by mass retained on each sieve was calculated to arrive at the percentage passing the sieve size.

c) Compaction of Samples

The sampled cold-mix asphalts were compacted following the Marshall procedure using the Marshall mechanical compactor. Representative samples of each stockpiled product was heated in an oven and maintained for some time at a temperature of 110^{o}C to make the material workable (Hassan, 2005). A weighed sample was then placed in the mould and spaded vigorously with a spatula around the perimeter and interior 15 times and 10 times, respectively, to condition the specimen before compacting. The average temperature of the mixtures before compaction was 110^{o}C while the compaction temperature ranged between 105 to 110^{o}C (Hassan, 2005). The samples were left in the mould for some hours before extrusion.

d) Stability and Flow Test ASTM D6927

The compacted specimens were conditioned for thirty minutes in a water bath maintained at a temperature of 60^{o}C before subjecting them to the stability and flow test. At the end of the conditioning period, each specimen was removed from the bath, carefully blotted dry and positioned in the testing head. A compressive load was applied at a constant rate until failure. The failure load is the stability value while the compression at that load is the flow.

3.3 Design of Dense-Graded Cold-Mix Asphalts

3.3.1 Quality of Aggregates

The aggregates for the study, which were sourced from a granite quarry in the Ashanti Region, were subjected to the following tests to establish their suitability for asphalt mixes.

a. Sieve Analysis
   Washed Sieve Analysis: ASTM C-117

b. Specific Gravity & Water Absorption: ASTM C-117
c. Abrasion Resistance/Strength
   - Los Angeles Abrasion Test: ASTM C-131
   - Aggregate Crushing Value Test ASTM D5821
   - Flakiness Index ASTM D4791
   - Aggregate Impact Value ASTM D2794

Table 3.1 is a summary of the quality indices of the aggregates used for the trial blends. The indices have been presented alongside those of Ministry of Transportation Standard Specification for Road and Bridges Table 17.10 Aggregate Requirement for Cold Mixes as captured in Table 2.2 (MOT, 2007). It is seen that all the requirements were met.

Table 3.1. Aggregate quality indices

QUALITY INDEX	TEST VALUE	MOT SPECIFICATION (2007)
Los Angeles Abrasion Value (LAAV) (%)	19	≤ 40
Aggregate Crushing Value (ACV) (%)	22	
Aggregate Impact Value (AIV) (%)	23	
Flakiness Index (%)	26	≤ 30
Water Absorption (%)	1.1	
Specific Gravity	2.676	

3.3.2 Aggregate Structure Design

For ease of formulation, four different aggregate sizes were blended by weight of total aggregate quantity and identified as CM1, CM2, CM3, CM4 and CM5 in the study (see Table 3.2). Aggregate size proportioning was guided by the five dense gradations regimes defined by the Asphalt Institute MS-19 Specification (1997).

Table 3.2. Aggregate size proportions in dense-graded blends

Aggregate Size (mm)	% by Weight of Total Aggregates				
	CM1	CM2	CM3	CM4	CM5
20	5	-	-	-	-
14	15	10	20	22	17
10	25	30	23	23	25
0 – 5	55	60	57	55	58

3.3.3 Binder Type used

Cationic Emulsion Bitumen K 1-60 (sourced from a local supplier) with the following properties was used as the binder:

- Rapid Setting Grade
- Bitumen Residue by Evaporation: 60%

3.3.4 Patching Mix Design

The Marshall mix design method was used for the design of the patching mixes. The procedure is standardized under ASTM D1559.

(a) Determination of Initial Residual Asphalt Content (IRAC)

The Centrifuge Kerosene Equivalent (CKE) method was used to establish the Initial Residual Asphalt Content, P_b, per Eq. (3.1).

$$P_b = 0.7 \times (0.05A + 0.1B + 0.5C) \tag{3.1}$$

where,

P_b = Initial Residual Asphalt Content (in %) by mass of total mixture

A = % of aggregate blend retained on Sieve No. 8 (2.36 mm)

B = % of aggregate blend passing Sieve No. 8 (2.36 mm) and retained on Sieve No. 200 (0.075 mm)

C = % of aggregate blend passing Sieve No. 200 (0.075 mm).

(b) Determination of Initial Emulsion Content (IEC)

The Initial Emulsion Content (IEC) was determined from the following expression:

$$IEC(\%) = 100\frac{P_b}{X} \tag{3.2}$$

where,

IEC = Initial Emulsion Content (% by mass of total mixture)

P_b = Initial Residual Asphalt Content (in % by mass of total mixture)

X= % bitumen residue in emulsion.

(c) Determination of Optimum Bitumen Emulsion Content

For each aggregate structure, a total of six cold-mix samples were prepared with asphalt contents that varied by increments of 0.5% on either side of the initial asphalt content and then compacted by the Marshall procedure. Specimen were compacted by the application of 50 blows on either side at a compaction temperature of 110^{o}C.

Compacted specimens were labeled and tested for the following properties in accordance with ASTM D 6927;

- Stability
- Flow
- Voids (VTM, VMA,VFA) were calculated following standard equations.
- Optimum Bitumen Content

3.4 Formulation of Cold-Mix Asphalts with Nanoclay Filler

Montmorillonite nano-clay, with the properties detailed in Table 3.3, was used in the study as filler in the cold mixes to evaluate any potential improvements (or otherwise) it could impart to the mixes.

Table 3.3: Properties of Montmorillonite nanoclay filler (ITM, 2021)

Property	**Value**
Size (nm)	< 80
Purity (%)	99
Colour	Off-White
Density (g/cm^3)	1.88
Moisture content (%)	< 2.5
Flexural Modulus (GPa)	3.78
Hardness	83
Tensile Strength (MPa)	101
Modulus of Elasticity (GPa)	4.657

For each of the five aggregate blends, i.e., CM1, CM2, CM3, CM4 & CM5, montmorillonite nanoclay filler was added at contents of 1%, 2% and 3% by weight of the graded aggregates before adding bitumen emulsion. For a given nanoclay content, the Marshall procedure was used to establish the optimum bitumen content. Stability and Flow tests for the mixes were carried out in accordance with ASTM D 6927.

CHAPTER 4: RESULTS AND DISCUSSION

4.1 Important Aspects of Reviewed Specifications

4.1.1 Aggregate Strength Requirement

The Ministry of Transportation Standard Specification for Road and Bridge Works (2007) on aggregates for road works specifies the strength requirement of the aggregate on the basis of the following quality indices: Los Angeles Abrasion (LAA), Flakiness Index (FI) and 10% Fines. These indices, for the most part, define aggregate suitability quantitatively in terms of shape characteristics as well as resistance to compressive and abrasive forces. The Asphalt Institute MS-19 (1997) specifies the minimum strength requirement only on the basis of maximum accepted Los Angeles Abrasion Value whereas the Chevron USA Incorporated Procedure (1977) is silent on this requirement.

4.1.2 Aggregate Gradation Requirement

Ghana's Ministry of Transportation Standard Specification for Road and Bridge Works (2007) does not have any aggregate gradation requirements for pothole patching mixes to guide contractors. Instead, it indulges Engineers supervising patching projects to attach special specifications that provide guidelines for the work. The Specification is also silent on the source from which supervising Engineers are to obtain such guidelines. Interestingly, no such guidelines could be found attached as Special Specification to any pothole patching contract documents available at the Ghana Highway Authority Head Office or any of the District Offices reviewed in this study. Searches conducted also in the other sister road agencies such as the Department of Urban Roads and Department of Feeder Roads were fruitless. These searches were undertaken from January 2020 to May, 2020. This is a great cause for concern as contractors executing patching projects would not have any standard to work to. Rather, a Ghana Highway Authority Specification titled Bituminous Spray Sealing Guide (2001) for chip seal works was the document found to contain aggregate gradation requirement and corresponding approximate binder content for cold mixes for pothole patching. If this aspect of the Spray Sealing Guide is deemed suitable for or applicable to cold-mixes in pothole patching, then the relevant portion should be lifted and made part of the technical specifications for pothole patching works. Alternatively, supervising Engineers on pothole patching works could be directed to the Spray Sealing Manual to obtain the necessary guidance for controlling the quality of the cold mix. In comparison, the Asphalt Institute Series M-19 (1997) clearly provides aggregate gradation requirements for both open-graded and dense-graded aggregate structures (see Tables 2.2 and 2.3) which permit better use of available aggregates. To this end, the Specification provides five dense-graded and four open-graded aggregate structure types to enable a wide range of aggregate structures to be selected. Similarly, the gradation requirement provided by the Nikolaides Specification (1994) also provides nine aggregate gradation envelopes (see Table 2.6) to allow for a broader gradation spectrum. However, with this Specification, no aggregate structure classification as either open- or dense-graded is made.

4.1.3 Binder Requirements

The Ministry of Transportation Standard Specification for Road and Bridge Works (2007) defers the determination of the emulsion binder component in the cold patching mix, referred to as the "nominal binder content" in the specification, to the Design Engineer. The Engineer is directed to establish this through laboratory tests and site trials. This could be very cumbersome considering the frequency of potholes and the swift patching response needed and could be easily overlooked resulting in contractors not receiving the correct requirement. In comparison, both the Asphalt Institute MS-19 (1997) and Chevron USA Incorporated Procedure (1997) use the Centrifuge Kerosene Equivalent Method to establish a correlation between the aggregate gradation and the residual emulsion binder content. The Asphalt Institute MS 19 goes a step further to establish the Initial Emulsion Content. This helps to avoid potential computational errors that could arise from failure to take into account the liquid phase in the determination of the nominal bitumen content required for the mix. The Nikolaides Specification, on the other hand assigns a predefined nominal binder content to each aggregate gradation type making it very easy to use.

It can be seen that the lack of specificity regarding binder requirement in the MOT Specification does not help matters given the fact that the stability of CMAs is dependent on the internal friction and cohesion within the compacted matrix, with the latter derived from the binder in the matrix. So where there is deficiency in binder content, cohesion within the compacted matrix will likely fall short of the optimum level required, resulting in a mix that easily disintegrates under traffic action.

4.1.4 Binder Compatibility with Aggregates

The establishment of a correlation between the binder content and the aggregate gradation type as seen by the Asphalt Institute MS-19 (1997), Chevron USA Incorporated Procedure (1977) and Nikolaides Specification (1994) is in order but a further classification to harmonize the bonding affinity by grouping the bitumen emulsion type by charge to correspond with aggregate types of opposite surface charge would have been in order. This recommendation was first made by Leech (1994) when he established that to facilitate breaking of emulsions, anionic emulsions should be used with positively charged aggregates such as limestone whilst cationic emulsions are more suitable for negatively charged aggregate such as granite and quartzite.

4.1.5 Evaluation of Aggregate Coating

Coating of aggregates recorded as a numerical percentage, is observed after mixing emulsion bitumen and aggregates thoroughly and drying. Significantly, factors such as mixing effort, emulsion type and quality, quality and charge of aggregate affect the degree of coating. Coating is necessary to engender inter-particle cohesion and bonding in a compacted matrix. When a substantial proportion of the surfaces of the aggregates in the mix are not coated, the mix is likely to be friable and prone to disintegration. The Chevron Method stipulates mixing either by the use of spoon and bowl or by mechanical means. In Ghana, the Ministry of Transportation Standard Specification for Road and Bridge Works (2007) is totally silent on coating requirement whereas both the Asphalt Institute MS-19 (1994) and Nikolaides Specification (1994) require a at least a minimum 85% of the aggregate surface area in the mix to be coated; the Chevron USA Incorporated Procedure (1977) accepts a minimum coating of 70% (Yeggoni *et al.*, 1994).

4.1.6 Mix Curing Protocols and Strength Evaluation

The Ministry of Transportation Standard Specification for Road and Bridge Works (2007) is silent on curing protocols and no guidelines are provided for curing, again the Specification does not provide acceptable values for soaked stability and allocates all the powers in determining the acceptable stability of the mix to the Engineer. Ironically, most Engineers engaged on how this acceptable stability is determined also pointed to the Specification in stark contrast.

Asphalt Institute MS-19 (1997) specifies a minimum acceptable soaked stability of 1.224 kN tested at $25^oC \pm 1^oC$ (50 Blows for medium traffic, compaction temperature 110^oC), whiles the Nikolaides Specification (1994) accepts soaked stability values greater than 1.335kN tested at $25^oC \pm 1^oC$ (50 Blows for medium traffic, compaction temperature 110^oC). On the other hand, the Chevron USA Incorporated Procedure (1977) utilizes the Resilient Modulus, Stabilometer S-Value, Resistance R-Value and Cohesiometer C-Value to determine compacted strength. The Resilient Modulus ($M_{R)}$ measures the elastic response of the cold patch as a means of assessing its structural contribution to pavement. The Resistance (R-Value) is a measure of the stability/bearing capacity of the mix at a test temperature of $23^oC \pm 3^oC$. Likewise the Stabilometer (S-Value) is also a measure of the stability after full curing at a temperature of $60^oC \pm 3^oC$. The Cohesiometer C-Value is also a measure the cohesive resistance or tensile strength of the compacted mix at a temperature of $60^oC \pm 3^oC$.

For open-graded emulsified mixes, the Chevron USA Incorporated Procedure (1977) specifies an S-Value greater than 30 and a C-Value greater than 100. For dense-graded emulsified mixes, the Procedure specifies an Acceptable Resistance Value ($R_{T)}$ value greater than 70 (Waller, 1980. The R_T is derived from the expression;

$$R_T = \text{R-Value} + 0.5\text{C-Value} \qquad (4.1)$$

4.2 Characteristics of Field Cold-Mix Asphalt Samples

4.2.1 Asphalt Type and Content

Table 4.1 contains a summary of the asphalt binder type as well as the residual asphalt content, of field cold-mix samples taken from contractor stockpiles, while Table 4.2 contains similar details on samples taken from failed road patches.

Table 4.1. Asphalt content and binder type of stockpiled cold-mix samples

ID of Patching Mix Sample	Asphalt Binder Type	Asphalt Content (%)
A1	Emulsion	3.60
A2	Emulsion	3.22
B1	Cutback	5.04
B2	Emulsion	3.13
V1	Emulsion	3.50
V2	Cutback	4.76

Table 4.2. Asphalt binder type and content of cold-mix samples from failed road patches

ID of Failed Patch Sample	Asphalt Binder Type	Residual Asphalt Content (%)
A3	*	4.34
B3	*	5.04
V3	Emulsion	3.94

The symbol (*) indicates the binder type could not established.

The type of binder used for patch materials A3 and B3 could not be established as the contractors who executed the project could not be reached. From the data above, it is evident that patch materials that were formulated with bitumen emulsion were generally characterized by binder contents that were lower than those of mixes formulated with cutbacks. The reason for this is not far-fetched. It is suspected that computation of the binder content might have been based on the weight of the emulsion and not the weight of the quantity of binder within the emulsion. Such an approach undoubtedly would lead to a mix with deficiency in binder after the water in the emulsion has evaporated. Deficiency in binder content means the aggregates in the matrix would not be coated sufficiently with binder. This would result in a friable mixture prone to rapid disintegration in service due to lack of adequate inter-particle adhesion.

4.2.2 Grading Characteristics of Stockpiled and Failed Patch Samples

The gradation characteristics of the aggregate of the stockpiled and failed road patch samples are shown in Fig. 4.1.

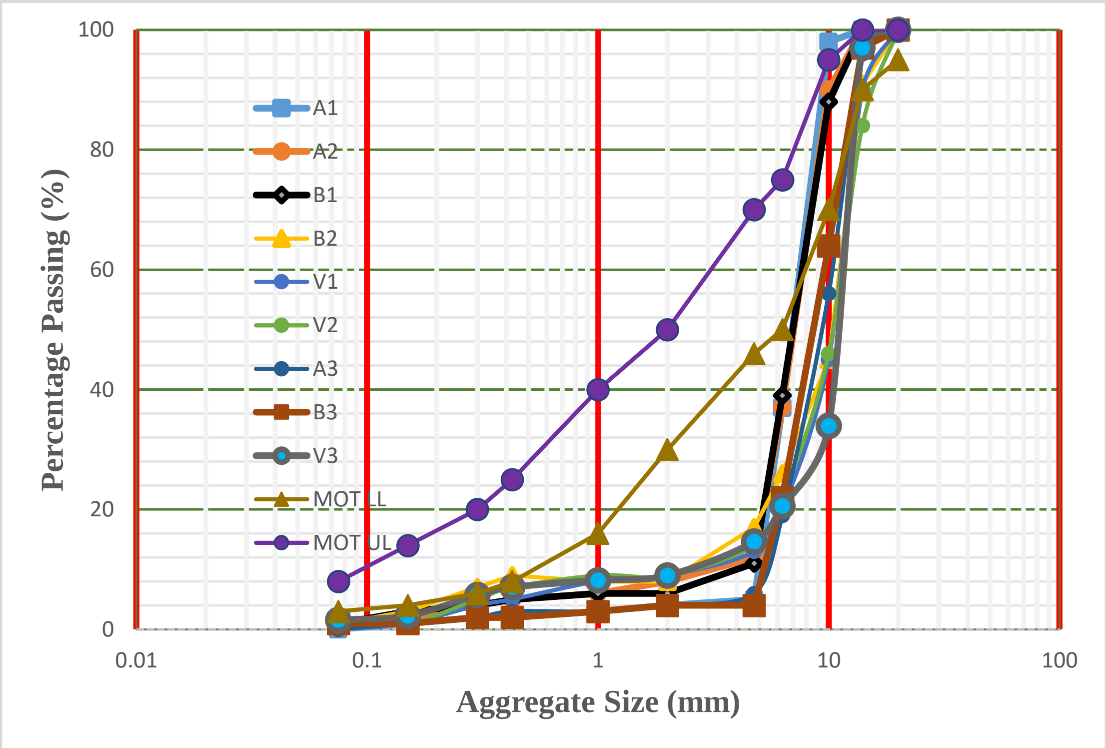


Fig. 4.1. Particle size distribution curves of sampled patching mixes

Superimposed on the chart are the gradation envelopes for the Ministry of Transportation Standard Specification for Road and Bridge Works (2007) grading requirements for Asphalt Concrete Wearing Course Type II Nominal Size 14 Table 17.3. It is seen that all the samples did not pass the grading requirements (MOT, 2007). In addition, Samples A1, A2 and B1 were formulated with 10mm maximum nominal aggregate size whereas the rest of the samples (i.e., B2, V1, V2, A3, B3 and V3) were formulated with 14mm maximum nominal size aggregates.

Based on Fig. 4.1, it may be noted that, the cold mixes lacked sufficient fines in their composition as the aggregates passing the 4.75mm Sieve size ranged from 2% - 9% only. This is against the backdrop of a fines content requirement of the order of 45%-75% per the MOT (2007) Specification. Generally, lack of fines results in low compaction densities, poor aggregate interlock and high voids content. The resulting effect is a mixture that is unable to attain high stability and rigidity. Furthermore, Fig. 4.1 shows evidence of a narrow range of particle sizes within the bulk (about 80%) of the aggregate mass of the samples recovered from the field, an indication that the blends were largely composed of single size aggregates. The uniformity coefficient (C_u) associated with each of the particle size distribution curves may provide a quantitative characterization in this regard. Arguing from the standpoint of the MOT (2007) Specification aggregate requirement for asphalt concrete Wearing Course Type II (Nominal Size 14mm), an aggregate blend that strictly meets the lower limits of the Specifications would be expected to have a C_u value of the order of 8 while that which meets the upper limits would have a C_u value of the order of 35. This suggests that aggregate blends that meet specifications are expected to have C_u values that fall between 8 and 35. This is in stark contrast to the range of 1- 4 established for the field samples (see Table 4.3).

Table 4.3. Coefficient of uniformity (C_u) of patching mix aggregates

AGGREGATE ID	D_{60} (mm)	D_{10} (mm)	C_u
A1	7.0	5.0	1.4
A2	8.0	4.5	1.1
B1	7.5	5.0	1.5
B2	12.0	3.0	4.0
V1	11.0	3.0	3.6
V2	11.0	3.0	3.6
A3	10.0	5.0	2.0
B3	10.0	5.0	2.0
V3	11.0	3.0	3.6
MOT Specs (LL)	10.0	1.2	**8.0**
MOT Specs. (UL)	7.0	0.2	**35.0**

In summary, the aggregate mass of the field cold-mix samples were deficient in fines content and composed largely of coarse fractions with a narrow particle size range, thus making them essentially single-size aggregates.

4.3 Marshall Stability and Flow of Stockpiled and Failed Patch Mixes

The samples were conditioned in a water bath at a temperature of 60^{o}C for 40 minutes (ASTM D1559, 1997). All the samples disintegrated during the conditioning and could not be tested. This can be attributed to poor gradation and inadequate binder resulting in a mixture that lacks cohesion and hence low stability. Because of the single size nature of the aggregates, the compacted sample could not develop an interlocking mosaic to provide sufficient rigidity. In addition, insufficiency of binder, as established earlier, probably led to low cohesion and poor inter-particle adhesion within the compacted matrix

4.4 Properties of Trial Dense-Graded Cold-Mix Asphalts

a) Trial Blend CM1

Fig. 4.2 provides a plot of the sieve analysis test performed on Trial Blend CM1 superimposed on the grading envelope provided by the MOT (2007). It can be seen that, the grading requirement was met. The blend also had a coefficient of uniformity value C_u=16, which falls within the range 8-35 associated with the MOT (2007).

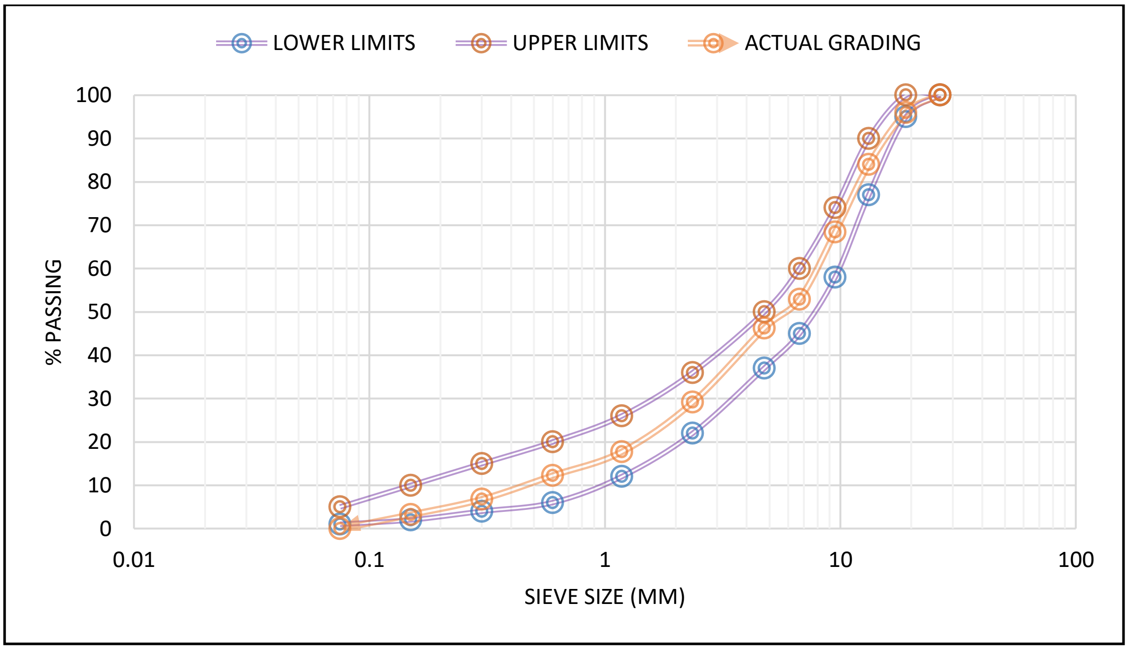


Fig. 4.2. Particle size distribution curve for Mix CM1

The actual percentage passing the various sieves have been given in Table 4.4 below.

Table 4.4 Aggregate Gradation for Trial Blend CM1

Sieve Size (mm)	**Percentage by mass passing (%)**	**MOT (2007)**
20	100	100
14	93	90 - 100
10	85	70 - 95
6.3	68	55 - 85
4	53	46 - 75
2	46	35 - 60
1	29	25 - 45
0.425	18	14 - 32
0.3	12	11 - 27
0.15	7	6 - 17
0.075	3	3 - 8

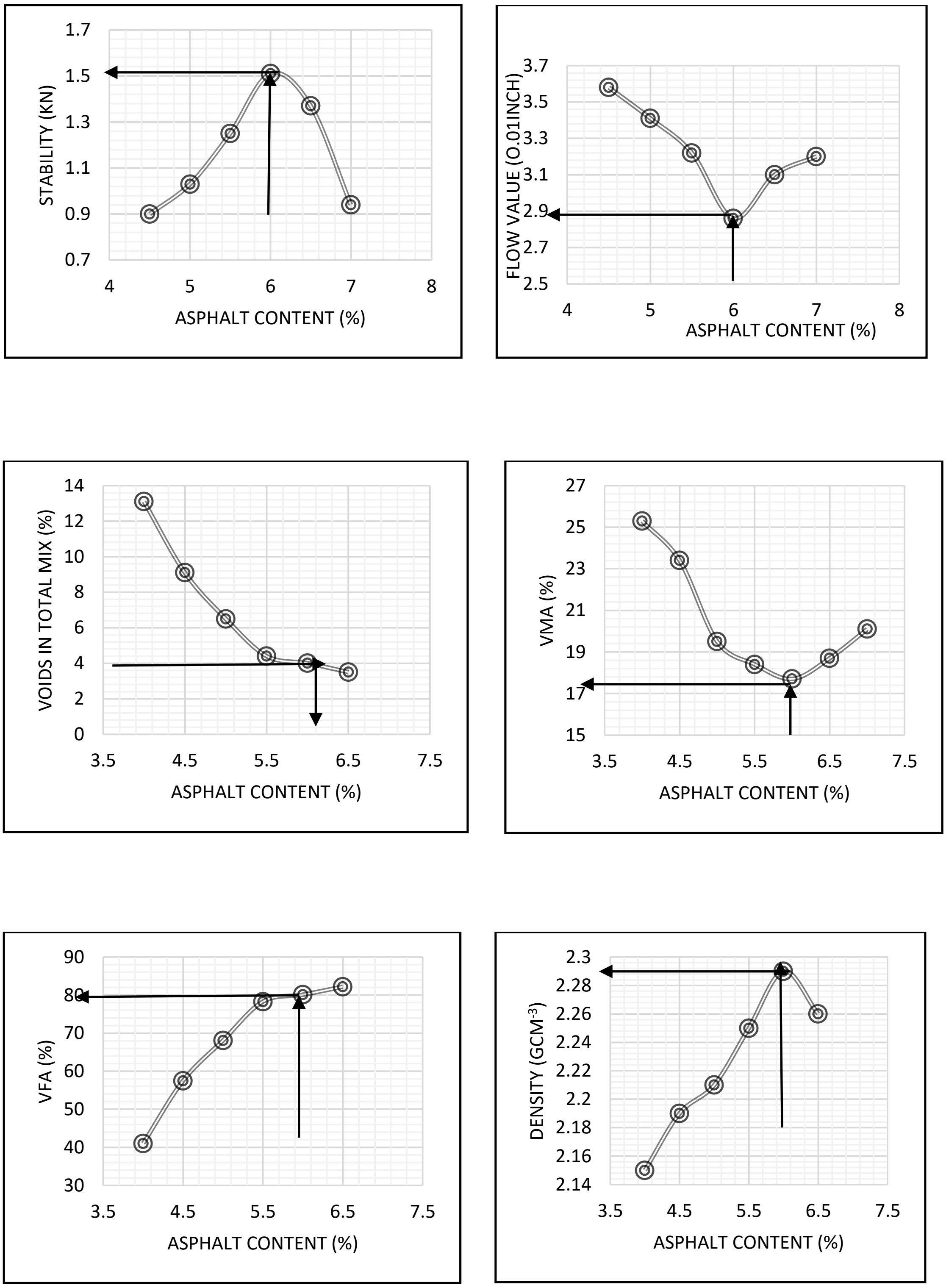


Fig. 4.3: Graphical presentation of cold-mix asphalt design data for CM1

The design established an optimum bitumen content of 6% for mix production. This is the asphalt content that produces exactly 4% air voids. This is based on the National Asphalt Pavement Association Method (QI Series, 2019). Table 4.5 contains the relevant mix properties at the optimum binder content for the CM1 aggregate structure. The Stability value was compared with the Ghana Highway Authority's Bituminous Spray Sealing Guide, Section 14 Specification for Pothole Patching and Repairs (2001) which requires a minimum Stability of 1kN. Since this specification is silent on the other parameters such as the Flow, VTM and VFA; these were compared with the Ministry of Transportation's Standard Specification for Road and Bridge Works (2007) Table 17.5.

Table 4.5: Properties of Mix CM1 at optimum binder content.

MIX PROPERTY	**MOT (2007)**	**MAX. VALUE**	**REMARK**
Marshall Stability (N)	1000 (min) *	1510	OK
Flow Value (0.01 inch)	2 – 4	2.86	OK
VTM (%)	3 – 5	4.0	OK
VFA (%)	65 – 75	75	OK

* GHA Specification (2001)

A simplified procedure to batch such a mix in the field to obtain a bitumen content of 6% would be to adopt the ratio **1:15.7** in terms of binder-to-aggregate ratio by weight. However, given that cold mixes are mostly formulated with emulsions in the field, it would appear easier and more helpful for field technical personnel to formulate the mix on the basis of emulsion quantities. If the assumption is made that the average residual bitumen content in emulsions is 60% by weight of the emulsion, then the above ratio translates approximately to **1:9 (**i.e., one part of emulsion to nine parts of aggregates by weight).

b) Trial Mix Blend CM2

As shown by the grading curve in Fig. 4.4, mix blend CM2 exhibits a broad range of particle sizes within the bulk mas to meet the MOT (2007) requirements. The Uniformity Coefficient for the aggregate structure was established to be 16.

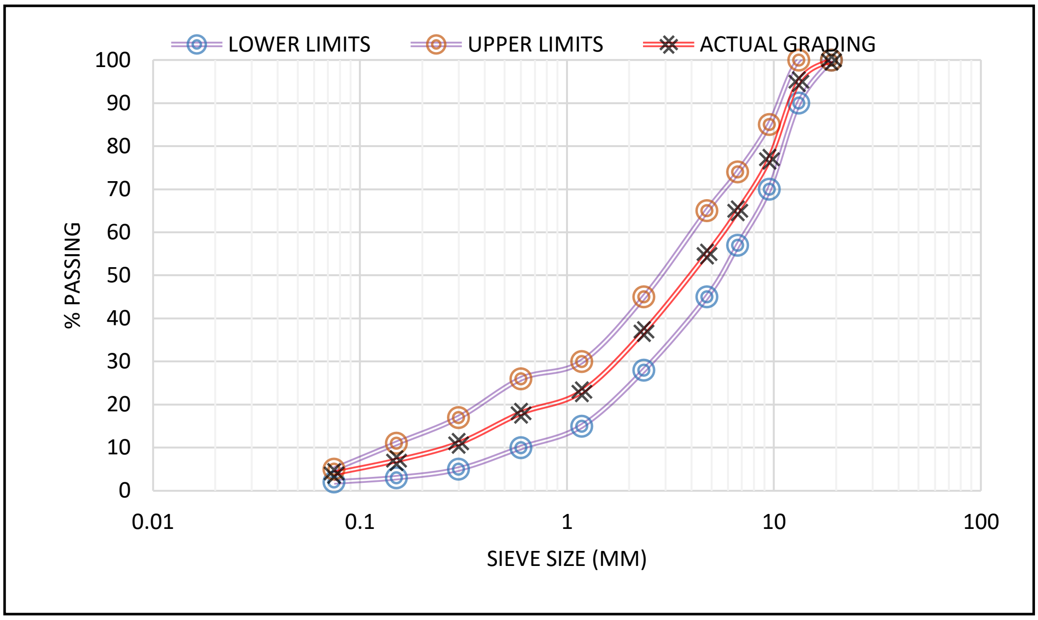


Fig. 4.4. Particle size distribution curve for Trial Mix CM2

The actual percentage passing the various sieves have been given in Table 4.6 below.

Table 4.6 Aggregate Gradation for Trial Blend CM2

Sieve Size (mm)	Percentage by mass passing (%)	MOT (2007)
20	100	100
14	95	90 - 100
10	77	70 - 95
6.3	68	55 - 85
4	55	46 - 75
2	38	35 - 60
1	25	25 - 45
0.425	18	14 - 32
0.3	13	11 - 27
0.15	8	6 - 17
0.075	4	3 - 8

Data on the trial design of the cold mix composed of the aggregate structure identified as CM2 is presented graphically by Fig. 4.5.

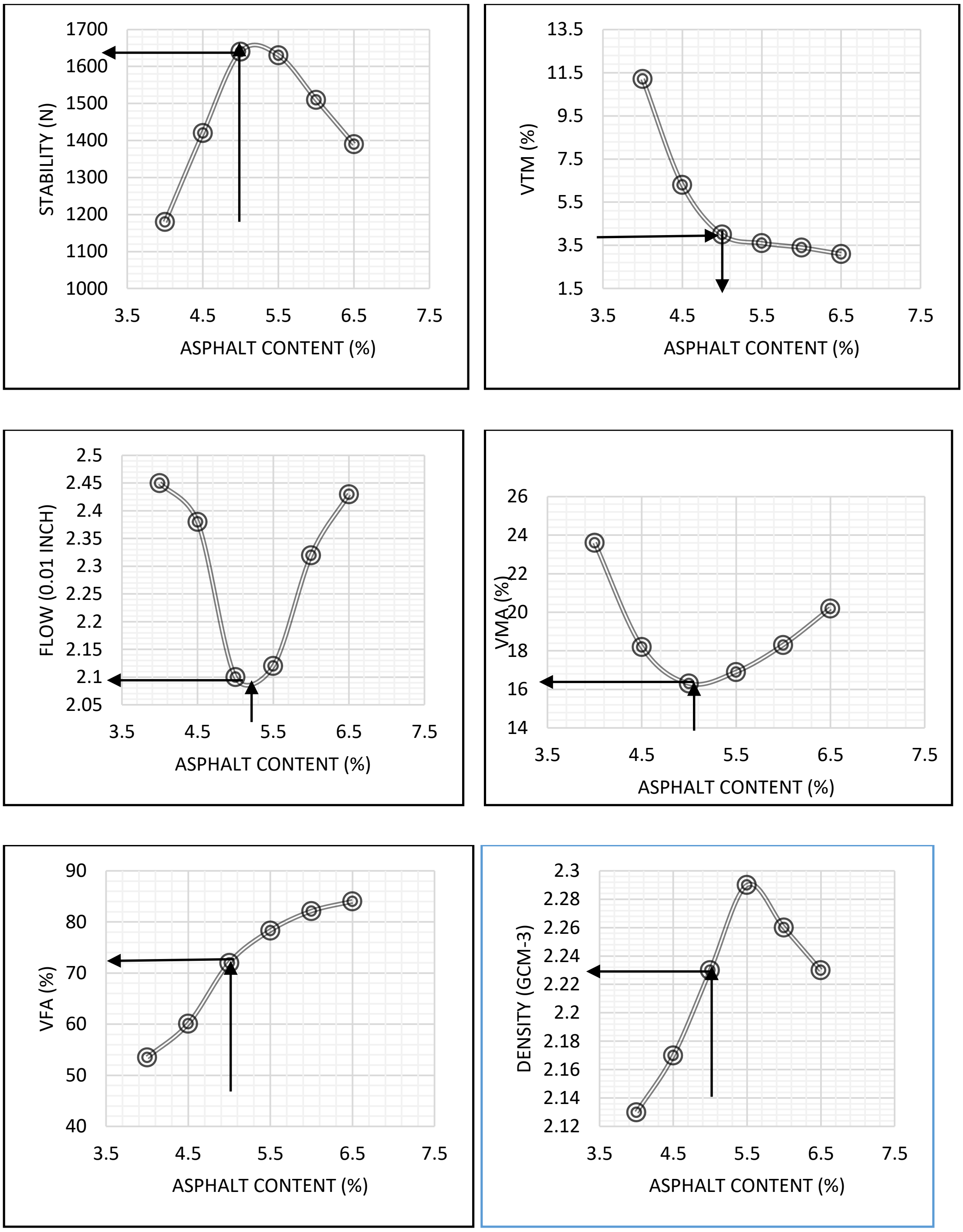


Fig. 4.5: Graphical presentation of cold-mix asphalt design data for CM2

An optimum bitumen content of 5% was established for mix formulation. This is the asphalt content that produces exactly 4% air voids. This is based on the National Asphalt Pavement Association Method (QI Series, 2019). Table 4.7 contains the relevant mix properties at the optimum binder content for the CM1 aggregate structure. The Stability value was compared with the Ghana Highway Authority's Bituminous

Spray Sealing Guide, Section 14 Specification for Pothole Patching and Repairs (2001) which requires a minimum Stability of 1kN. Since this specification is silent on the other parameters such as the Flow, VTM and VFA; these were compared with the Ministry of Transportation's Standard Specification for Road and Bridge Works (2007) Table 17.5.

Table 4.7: Properties of Trial Mix CM2 at optimum bitumen content

MIX PROPERTY	MOT (2007)	MAX. VALUE	REMARK
Marshall Stability (N)	1000(min) *	1640	OK
Flow Value (0.01 inch)	2 – 4	2.10	OK
VTM (%)	3 – 5	4.0	OK
VFA (%)	65 – 75	72	OK

*GHA Specification (2001)

To attain the optimum bitumen content, a mix ratio of 1:19 in terms of binder-to-aggregate by weight is required. Based on the previous assumption regarding the residual bitumen content of emulsions, this translates approximately to the ratio **1: 11** in terms of emulsion-to-aggregate by weight for CM2 cold-mix production in the field.

c) Trial Mix Blend CM3

Fig. 4.6 shows the particle size distribution curve for the proposed aggregate Blend CM3 alongside the grading envelope of the MOT (2007) requirements. The Uniformity Coefficient characterizing the aggregate structure is 20.

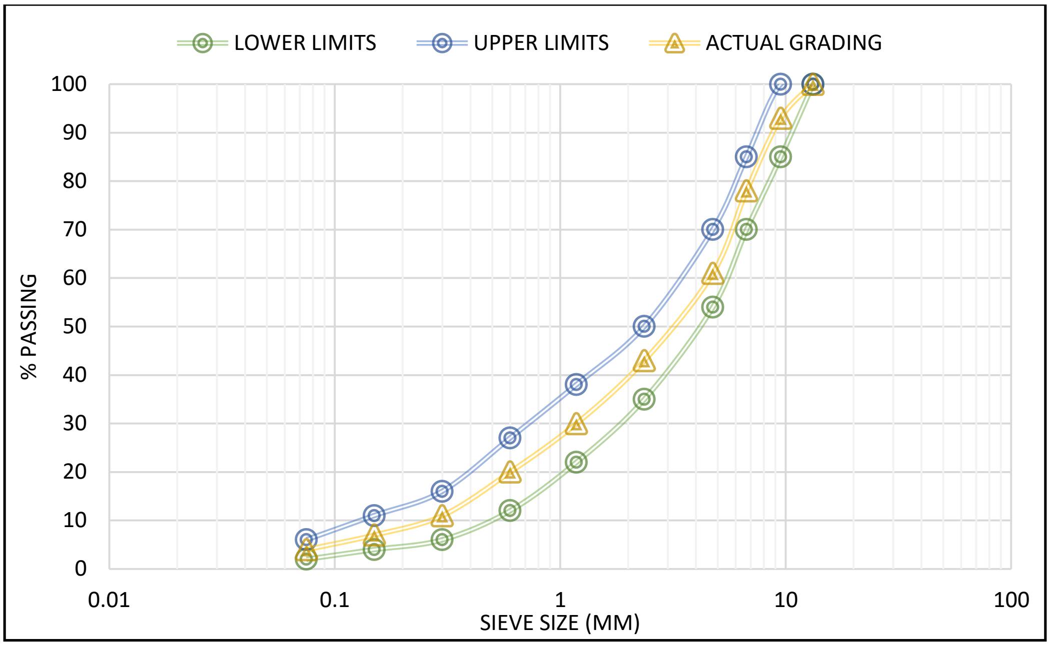


Fig. 4.6: Particle size distribution curve for Trial Mix CM3

The actual percentage passing the various sieves have been given in Table 4.8 below.

Table 4.8 Aggregate Gradation for Trial Blend CM3

Sieve Size (mm)	**Percentage by mass passing (%)**	**MOT (2007)**
20	100	100
14	91	90 - 100
10	88	70 - 95
6.3	57	55 - 85
4	49	46 - 75
2	35	35 - 60
1	27	25 - 45
0.425	19	14 - 32
0.3	15	11 - 27
0.15	9	6 - 17
0.075	5	3 - 8

A graphical presentation of the characteristics exhibited by the designed cold trial mix CM3 is given by Fig. 4.7. For this mix, a bitumen content of 5% was also obtained. Table 4.9 provides a summary of the mix properties at the optimum asphalt content.

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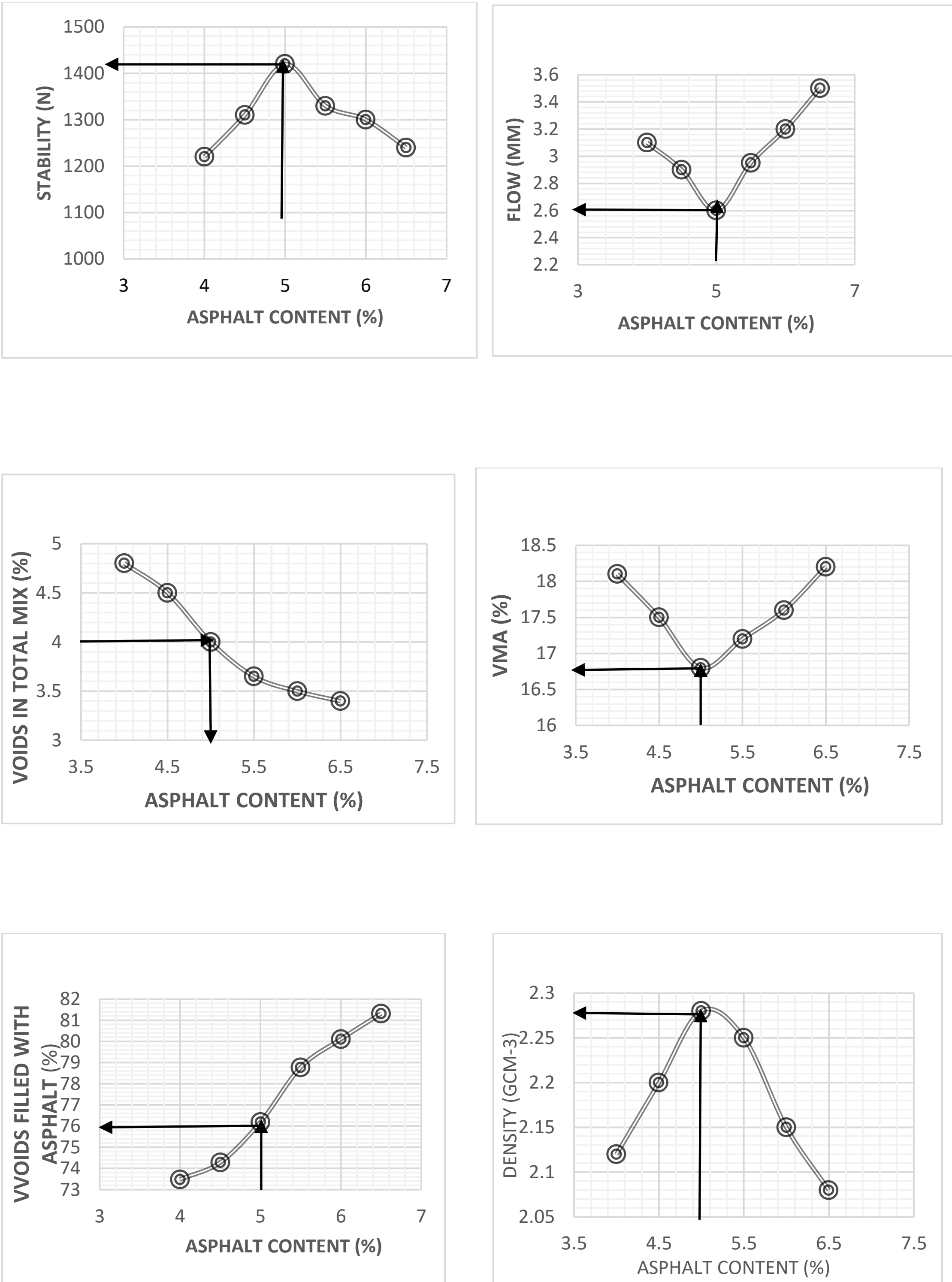


Figure 4.7: Graphical presentation of cold-mix asphalt design data for CM3

Table 4.9: Properties of Trial Mix CM3

MIX PROPERTY	MOT(2007)	MAX. VALUE	REMARK
Marshall Stability (N)	≥1000 *	1440	OK
Flow Value (0.01 inch)	2 – 4	2.60	OK
VTM (%)	3 – 5	4.0	OK
VFA (%)	65 – 75	75	OK

*GHA Specification (2001)

Similar to CM2, CM3 may also be proportioned using the ratio **1:19** in terms of bitumen-to-aggregate quantities by weight. Also in terms of emulsion use in mix formulation in the field, the ratio becomes approximately **1:11**.

d) Trial Mix Blend CM4

Fig. 4.8 provides the gradation Trial Blend CM4 superimposed on the grading envelope provided by the MOT (2007). The blend met the grading requirement. CM4 further established a Cu of 10 also within the limits provided to guarantee a broader distribution of aggregate sizes.

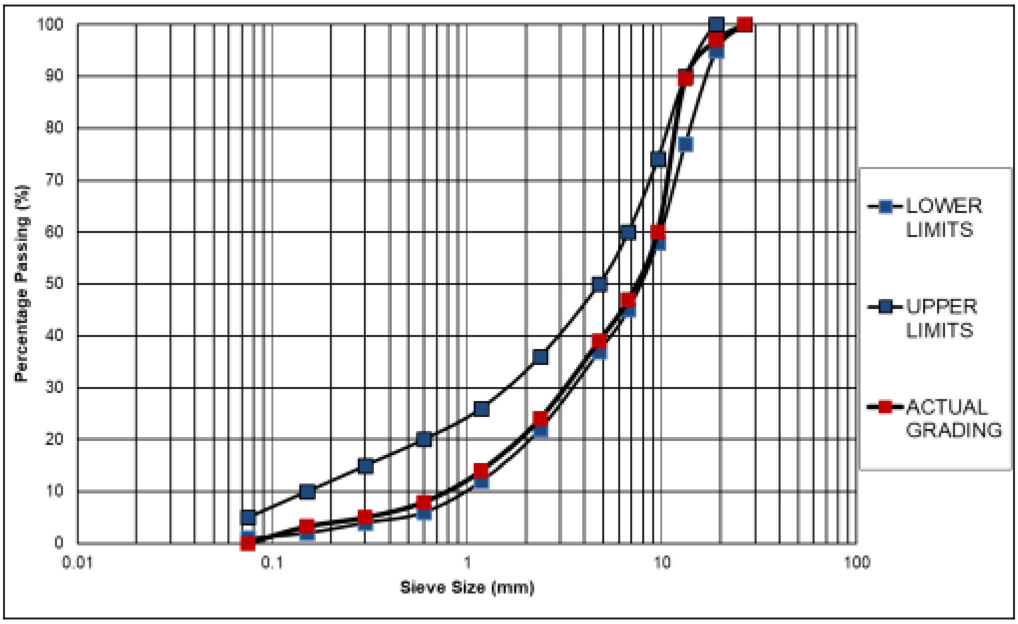


Fig. 4.8: Particle size distribution curve for Trial Mix CM4

The actual percentage passing the various sieves have been given in Table 4.10 below.

Table 4.10 Aggregate Gradation for Trial Blend CM4

Sieve Size (mm)	**Percentage by mass passing (%)**	**MOT (2007)**
20	100	100
14	93	90 - 100
10	90	70 - 95
6.3	68	55 - 85
4	46	46 - 75
2	35	35 - 60
1	26	25 - 45
0.425	15	14 - 32
0.3	12	11 - 27
0.15	7	6 - 17
0.075	3	3 - 8

Data on the trial design of CMAs containing the CM4 aggregate structure have been presented in Fig. 4.9. An optimum asphalt content of 4.5% was established for the mix. This is the asphalt content that produces exactly 4% air voids. This is based on the National Asphalt Pavement Association Method (QI Series, 2019). Mix properties at the optimum bitumen content has also been provided in Table 4.11. It is seen that all requirements are met at the optimum bitumen content. To formulate cold-mix asphalts having this aggregate structure, a simplified material batching ratio of **1:21** in terms of bitumen-to-aggregate by weight is proposed. In terms of emulsion quantities, this ratio translates approximately to **1:13.**

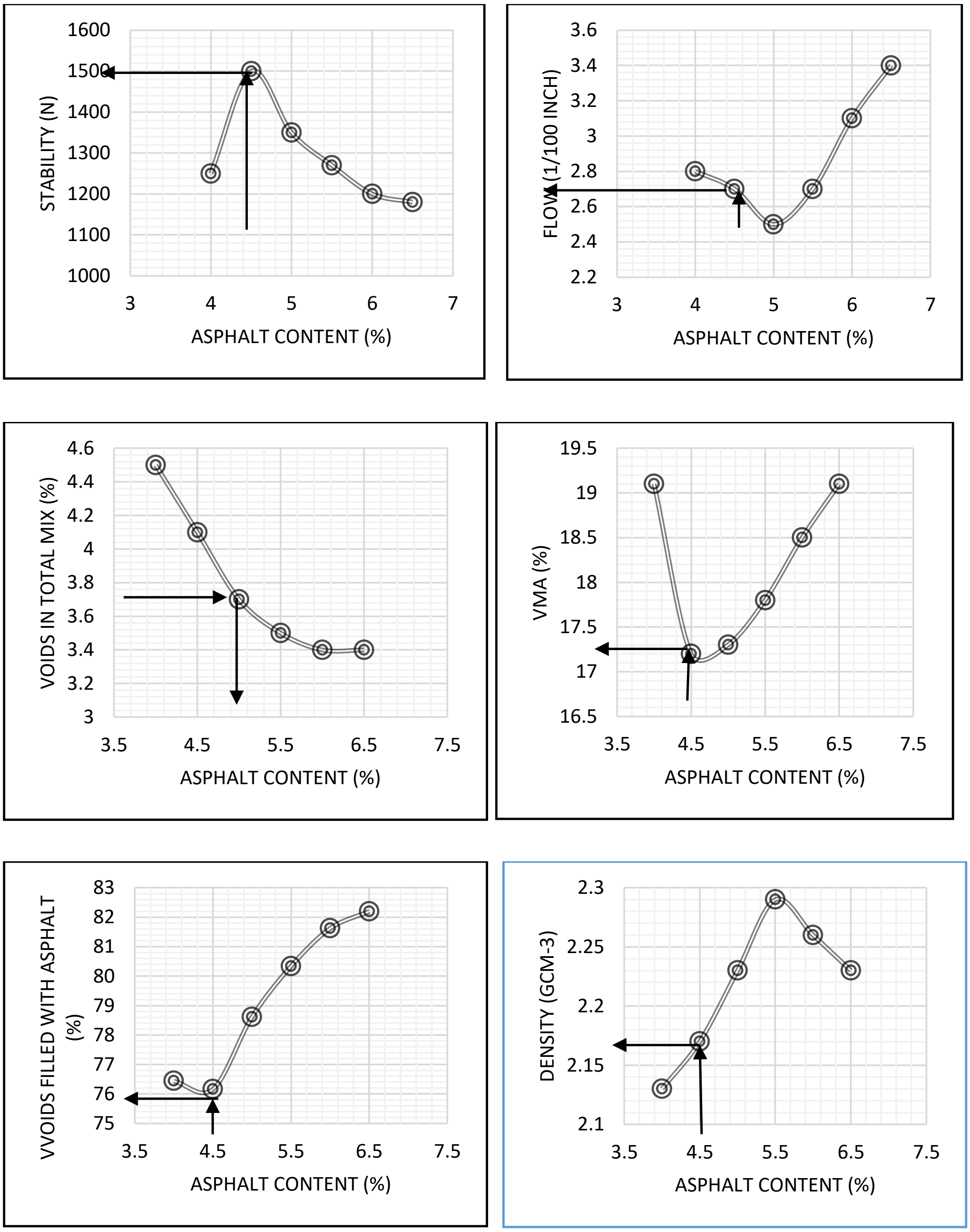


Fig. 4.9: Graphical presentation of cold-mix asphalt design data for CM4

Table 4.11: Properties of Trial Mix CM4

MIX PROPERTY	MOT (2007)	MAX. VALUE	REMARK
Marshall Stability (N)	1000(min) *	1500	OK
Flow Value (1/100 inch)	2 – 4	2.50	OK
VTM (%)	3 – 5	4.0	OK
VFA (%)	65 – 75	75	OK

*GHA Specification (2001)

e) Trial Mix Blend CM5

This trial blend is characterized by a broader range of particle sizes within the bulk aggregate mass resulting in a uniformity coefficient of C_u=17.5, and well within the range defined by MOT (2007) grading requirements (see Fig. 4.10).

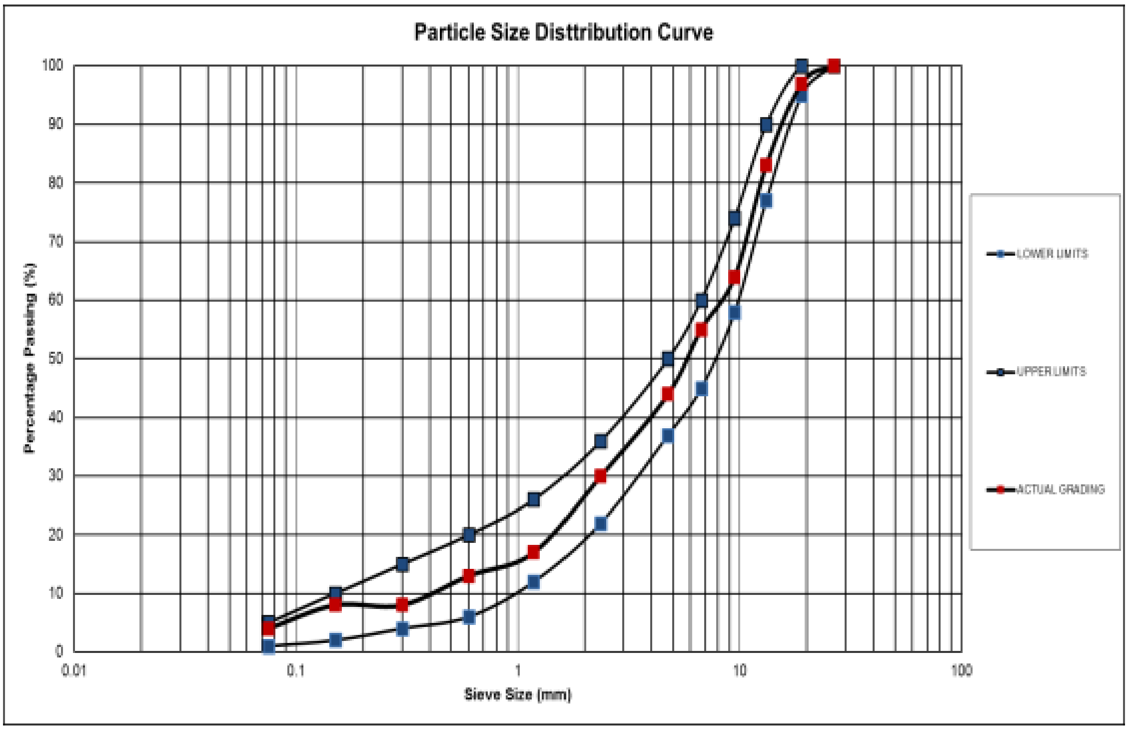


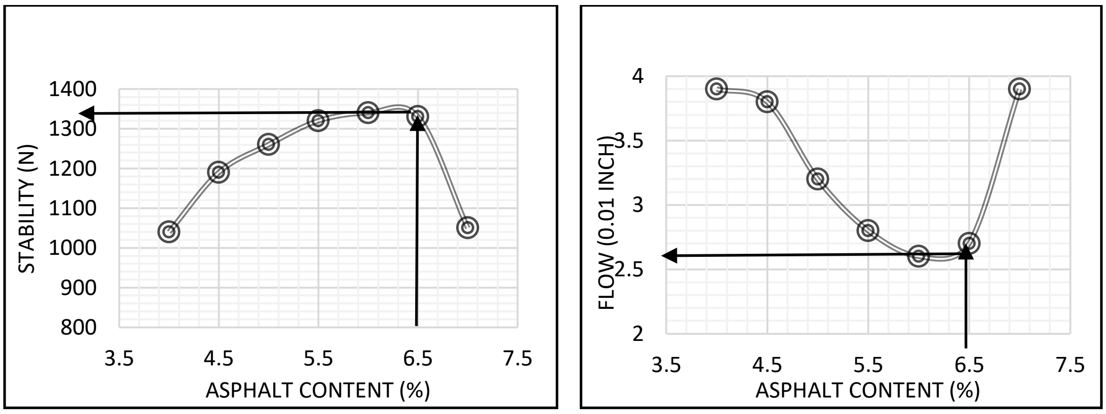
Fig. 4.10: Particle size distribution curve for trial mix CM5

The actual percentage passing the various sieves have been given in Table 4.12 below.

Table 4.12 Aggregate Gradation for Trial Blend CM5

Sieve Size (mm)	Percentage by mass passing (%)	MOT (2007)
20	100	100
14	95	90 - 100
10	72	70 - 95
6.3	59	55 - 85
4	47	46 - 75
2	37	35 - 60
1	29	25 - 45
0.425	19	14 - 32
0.3	14	11 - 27
0.15	8	6 - 17
0.075	3	3 - 8

A graphical presentation of the characteristics exhibited by trial mixes involving the CM5 aggregate structure is shown in Fig. 4.11. An optimum asphalt content of 6.5% was established for the aggregate blend. This is the asphalt content that produces exactly 4% air voids. This is based on the National Asphalt Pavement Association Method (QI Series, 2019).Table 4.13 provides a summary of the properties. A simplified binder-to-aggregate ratio of **1:14** by weight is proposed for mix formulation, and in terms of emulsion quantities to use in the field, this translates to **1:8.**



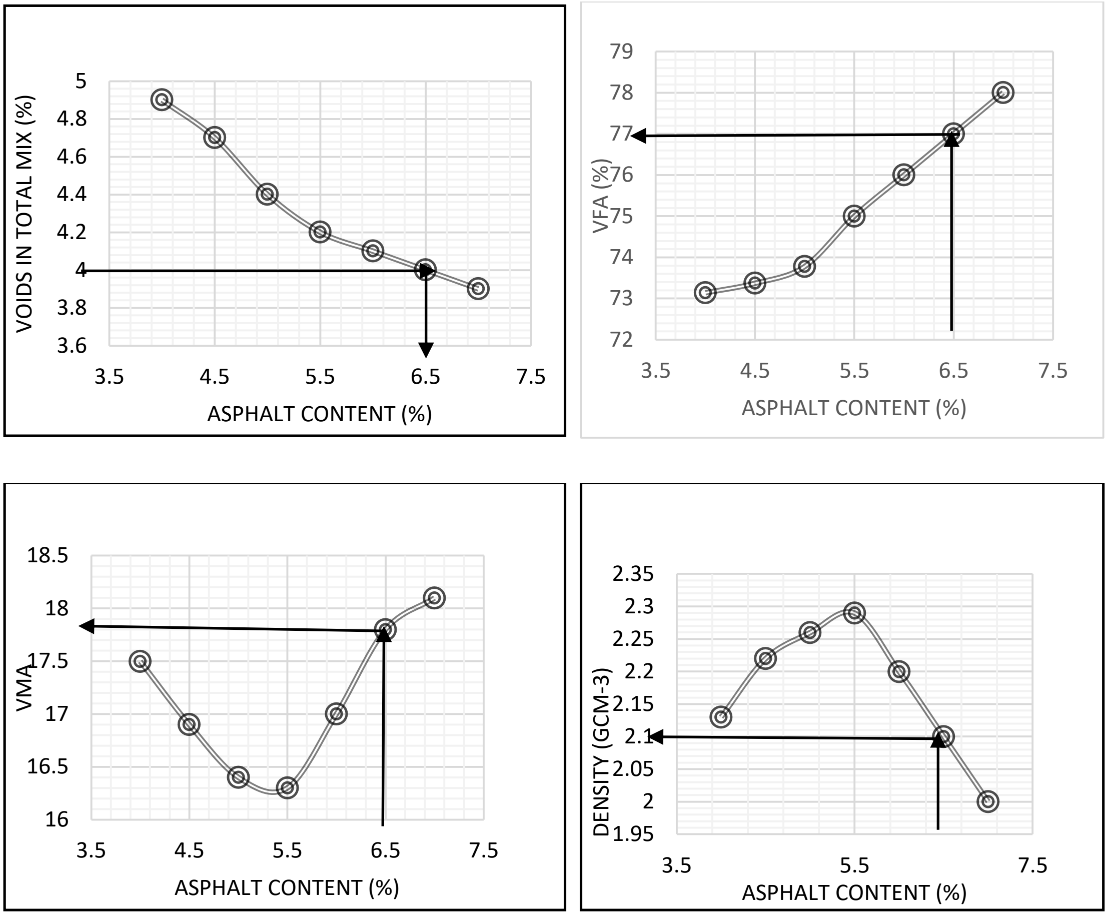


Fig. 4.11: Graphical presentation of cold-mix asphalt design data for CM5

Table 4.13: Properties of Trial Mix CM5

MIX PROPERTY	MOT(2007)	TEST VALUE	REMARK
Marshall Stability (N)	1000(min) *	1340	OK
Flow Value (1/100 inch)	2 – 4	2.7	OK
VTM (%)	3 – 5	4.0	OK
VFA (%)	65 – 75	75	OK

*GHA Specification (2001)

Table 4.14 provides a summary of the optimum asphalt contents associated with the proposed aggregate mixes and the emulsion-to-binder ratios required for their formulation in the field. In comparison, the optimum asphalt contents of the proposed aggregate blends stand in sharp contrast to the asphalt contents evaluated for the field cold-mix samples considered in this study from contractor sites (refer to Table 4.1).

Table 4.14: Summary of optimum asphalt content and emulsion-to-binder ratio for trial mixes

AGGREGATE BLEND ID	NOMINAL MAXIMUM SIZE (mm)	OPTIMUM BINDER CONTENT (%) BINDER TYPE- (K 1-160)	EMULSION-TO-AGGREGATE RATION (BY WEIGHT OF TOTAL MIX)	COEFFICIENT OF UNIFORMITY (Cu)
CM1	14	6.0	1:9	16
CM2	14	5.0	1:11	16
CM3	14	5.0	1:11	20
CM4	14	4.5	1:13	10
CM5	14	6.5	1:8	17.5

4.3.1 Compaction Levels at Site

Proposed trial blends were designed for an air voids content of 4%, a minimum of 96% relative compaction should be achieved on site.

4.3.2 Lift of Asphalt

A minimum lift of asphalt of 50 mm is recommended for the proposed designed blends.

4.5 Properties of Cold Mixes with Nanoclay Filler

a) Stability

For compacted asphalt concrete, Marshall stability is not an absolute measure of strength, however, the parameter derives largely from internal friction and cohesion within the compacted matrix. Cohesion in this regard, derives from inter-particle bonding and is affected more by the amount of bitumen within the matrix in the form of asphalt coating around the aggregates, while the internal friction measures the interlocking and frictional resistance of the aggregate. In view of this, the Marshall stability and flow test, as used in this study, reliably provided a fair basis for the performance prediction of the nanoclay-modified cold mixes. The results for Marshall Stability, Flow, Voids, and Density of the nanoclay-modified mixes, at the optimum binder content, shown in Table 4.15, satisfied the requirements. Also, it is observed from Figs. 4.12 and 4.13 that, all gradation groups recorded improvements in their stability when montmorillonite nanoclay was added as a modifier.

Table 4.15. Properties of nanoclay-modified compacted cold-mix asphalts

Nanoclay Modifier	Properties		Trial	Blends		
		CM1	CM2	CM3	CM4	CM5
0%	Optimum Bitumen Content (%)	4.6	5.4	5.5	6.0	6.0
	Stability (N)	1420	1275	1400	1330	1425
	Flow (0.01 inch)	2.6	3	3	3.4	3.55
	Air Voids (%)	4	4	4	4	4
	Voids Filled with Asphalt (%)	75.0	75.0	75.0	75.0	75.0
	Density (kg/m^3)	2210	2245	2270	2220	2300
1%	Optimum Bitumen Content	4.6	5.5	5.6	6.2	7.0
	Stability (N)	1800	1920	1710	1560	1520
	Flow (0.01 inch)	3	3.2	3.5	3.4	3.8
	Air Voids (%)	4	4	4	4	4
	Voids Filled with Asphalt (%)	76	77	74	76	75.8
2%	Optimum Bitumen Content	4.7	5.7	5.8	6.5	7.7
	Stability (N)	2520	2710	2230	2160	2210
	Flow (0.01 inch)	3.1	3	3.2	3.7	4.2
	Air Voids (%)	4	4	4	4	4
	Voids Filled with Asphalt (%)	75.5	75.8	75	75.4	74
3%	Optimum Bitumen Content	5.0	6.0	6.2	6.8	8.1
	Stability (N)	1980	1940	1750	1845	1780
	Flow (0.01 inch)	3.2	3.5	4.2	3.8	4.6
	Air Voids (%)	4	4	4	4	4
	Voids Filled with Asphalt (%)	74	75.2	75.1	75.1	72.9

MIX PROPERTY	SPECIFICATION
Marshall Stability (N)	1000(min) (GHA)
Flow Value (1/100 inch)	2 – 4 (MOT)
VTM (%)	3 – 5 (MOT)
VFA (%)	65 – 75 (MOT)

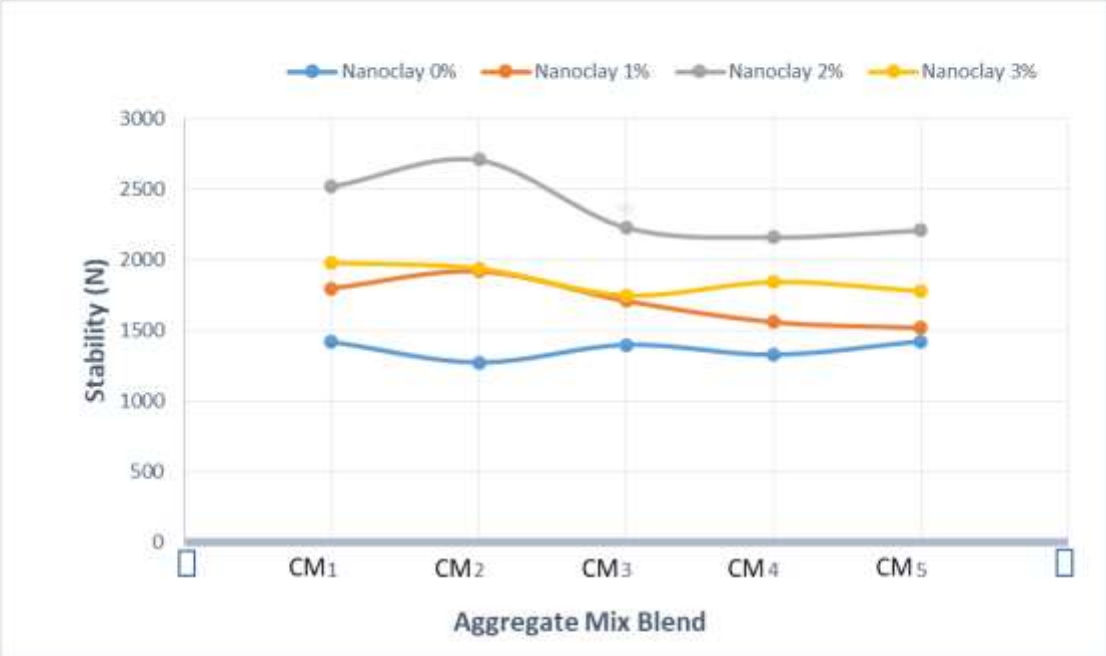


Fig. 4.12: Marshall Stability values per each gradation

It is seen that at a given nanoclay content, the stability did not vary much across the different mixes although for a given mix, the values tended to increase with increasing nanoclay content.

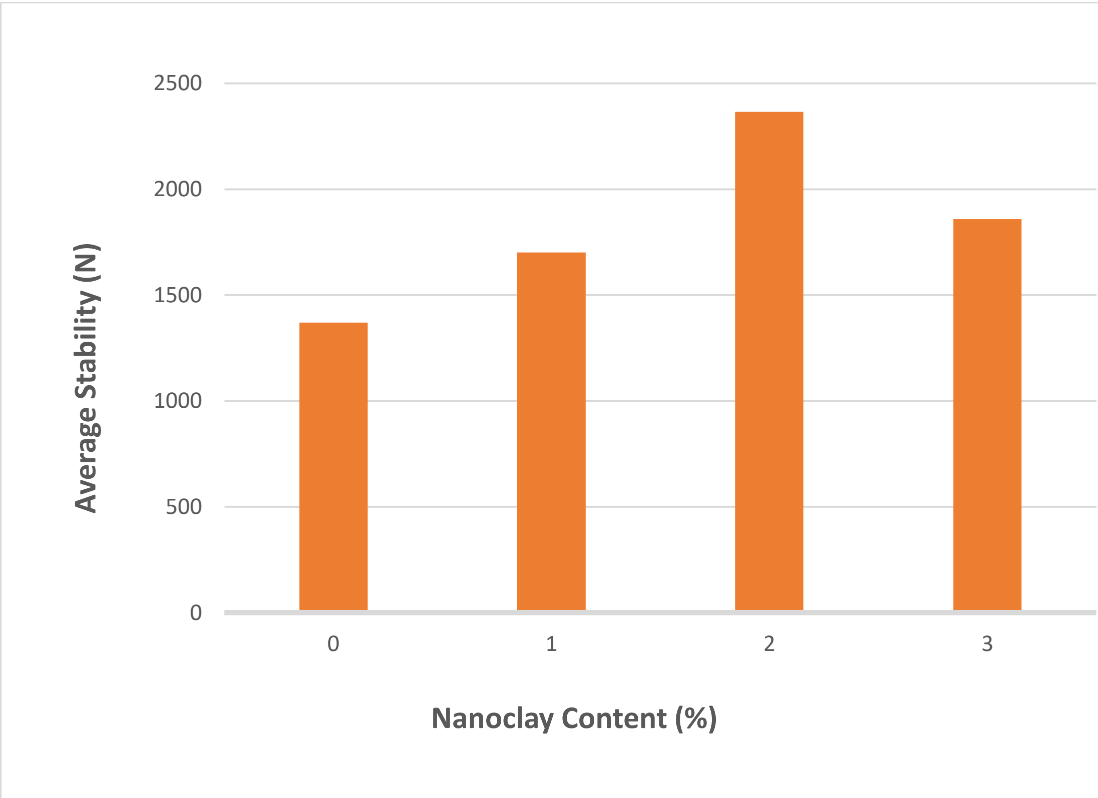


Fig. 4.13: Average stability at each nanoclay content

The average Marshall stability for the five mixes without nanoclay (control) was 1.37 kN. However, when 1% nanoclay was incorporated in the blends, the average Marshall stability increased to 1.70 kN, representing a 24% increase. When the nanoclay content was increased to 2%, the average Stability of the mixes increased to 2.36 kN, which represents a significant increment of 73% over the control value. However, at a 3% nanoclay content, the average stability of 1.86kN for the mixes fell below that for the 2% nanoclay content. This appears to suggest that the optimum nanoclay content for the mixes hoovers around 2%. It is also significant to mention that, aggregate gradation Group Two (CM2) recorded the highest improvements at all the nanoclay contents investigated.

b) Flow

Generally, it can be seen (Fig.4.14) that, all flow values of the cold asphalt mixtures mostly increased when nanoclay was incorporated. At the 3% nanoclay incremental level, the highest flow value was recorded with an average flow value of 3.86 for the five aggregate gradation groups, about 24.1% greater than the average flow for cold asphalt mixtures produced from the gradations of the unmodified mixture. An average flow of 3.28 and 3.44 were recorded at 1% and 2% nanoclay dosages, respectively; about 8% and 10.6% greater than the average flow of the control mixes.

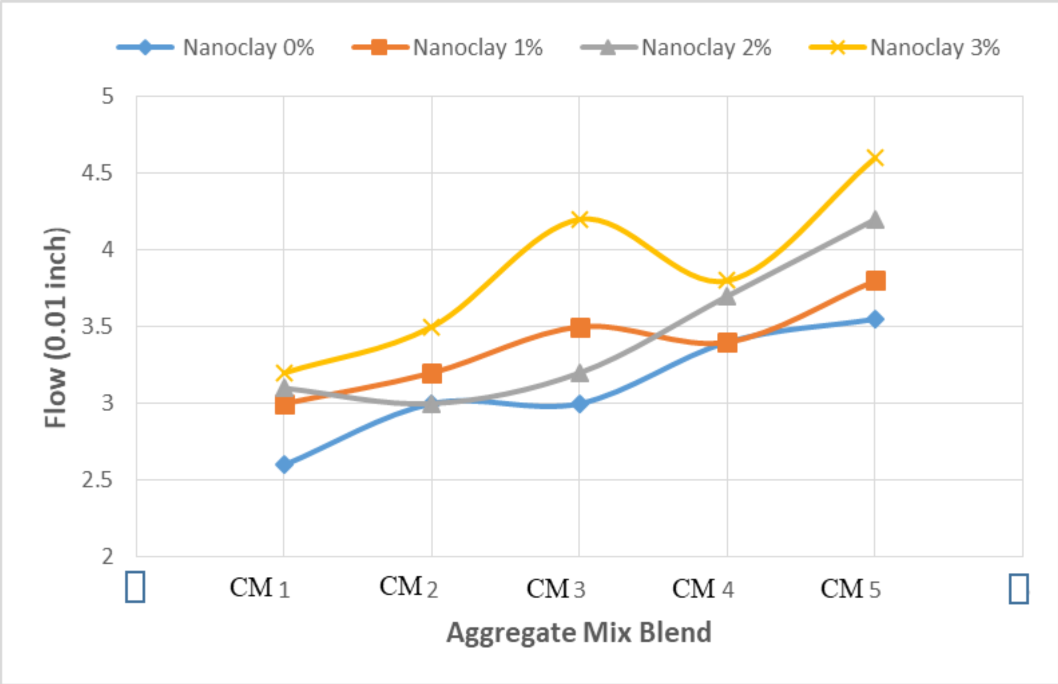


Fig. 4.14: Flow values per each gradation type

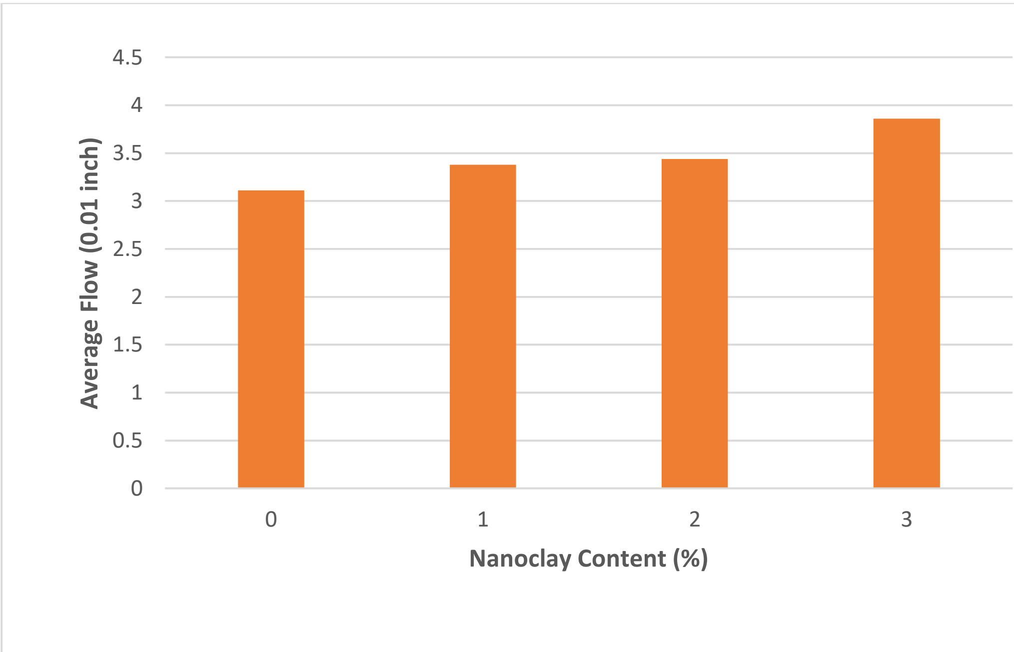


Fig. 4.15: Average flow at each nanoclay content

In between the nanoclay content levels, that is from 1% to 2% and 2% to 3%, average flow increased by 1.8% and 12%, respectively. Even though, samples with nanoclay exhibited plastic behaviour as evidenced by their high flow values and, therefore, were expected to have low stability values, they turned out to have higher stability values compared to the samples without nanoclay. Interestingly, it is again significant to note that, aggregate gradation group CM2 turned out to be the mix with the lowest average flow at all nanoclay contents investigated.

c) Optimum Bitumen Content

The results in Fig. 4.16, show that all aggregate gradation groups had an increase in Optimum Bitumen Content when nanoclay was added.

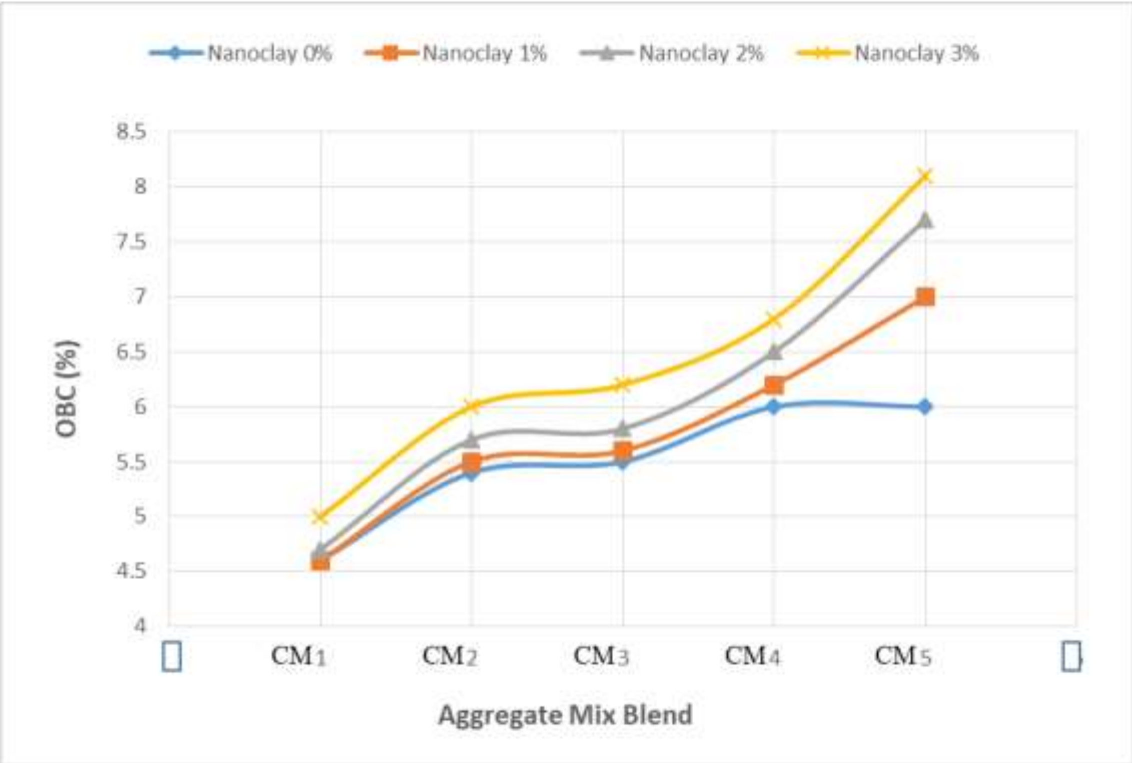


Fig. 4.16: Optimum OBC values per each gradation type

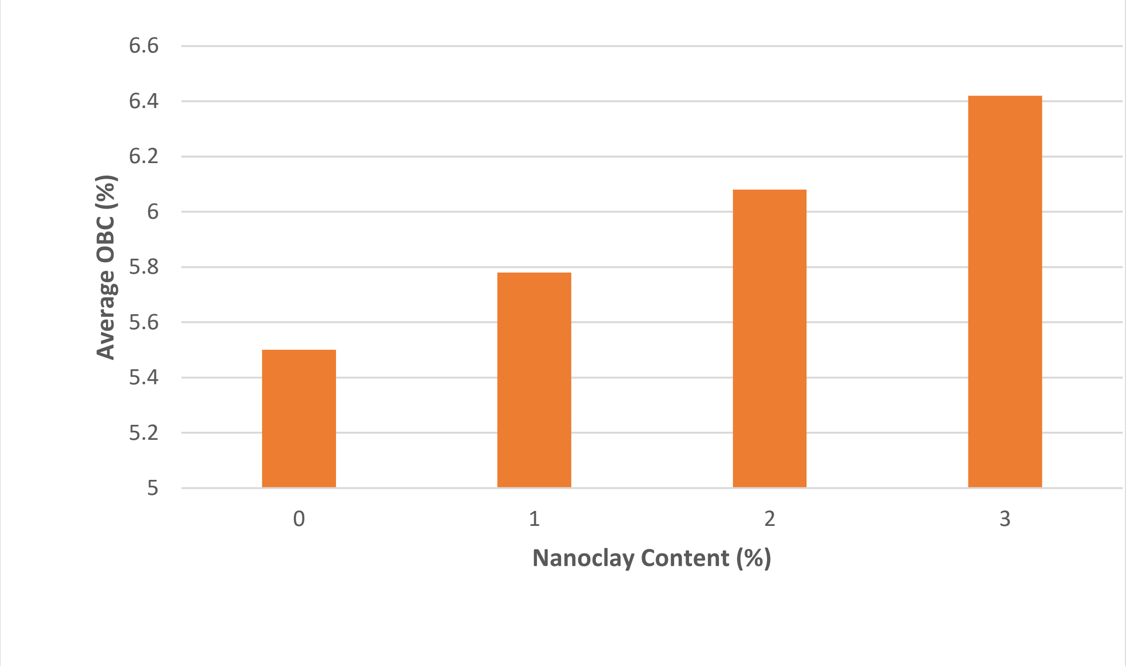


Fig. 4.17: Aggregated average mix optimum bitumen content at different nanoclay contents

As seen in Fig 4.16, the nanoclay filler increased the optimum bitumen content of the different gradation mixtures. The increase in the average optimum asphalt content aggregated for the mixes was only marginal on the addition of 1% nanoclay but substantial for nanoclay contents of 2% or more (see Fig. 4.17). The general increase in bitumen content on addition of nanoclay may be responsible for the increase in flow observed for the mixes.

d) Voids Filled with Asphalt

Mostly, the aggregate gradation groups decreased in VFA on addition of nanoclay except aggregate gradation CM3 which had a slight increase in VFA at 3% nanoclay content (see Fig. 4.18 & 4.19). Average VFA for all gradations decreased from 76.24% to 75.56%, when 1% nanoclay was added. When the nanoclay content was increased to

2% and 3%, the VFA of the mixtures decreased by 1.5% and 2.4%, respectively, compared to mixtures without nanoclay.

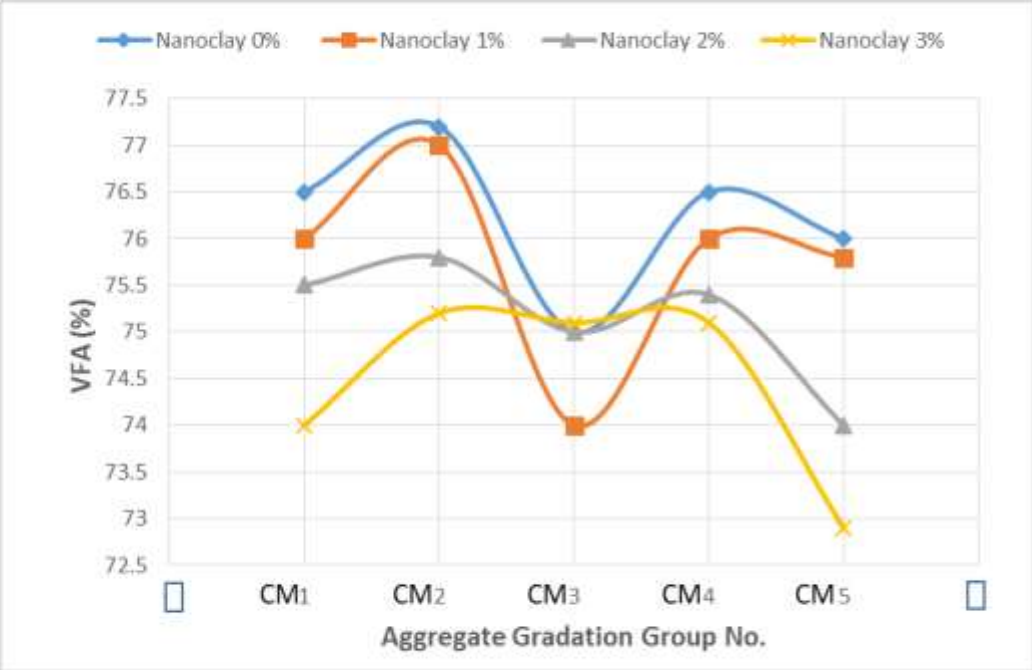


Figure 4.18: VFA values per each gradation type

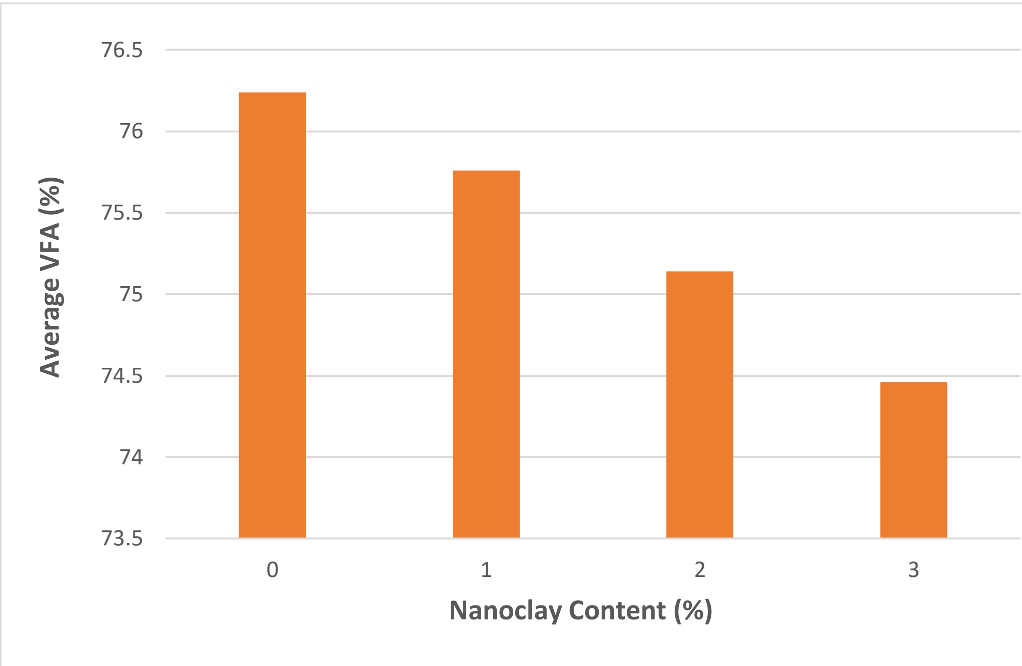


Figure 4.19: Average VFA with nanoclay content

Due to the fact that the nanoclay particles are extremely tiny, they have the advantage of being able to fill the inter-granular voids formed within the small, medium and large aggregate lattices in the compacted asphalt concrete. This way, they are able to reduce the voids within the mineral aggregates (VMA) which leaves less void spaces to accommodate asphalt coating the bitumen. This had a resultant effect in the decrease in VFA observed at each nanoclay incremental level.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Premature failure of pothole patches involving cold-mix asphalts is very common in the country and yet very little has been done to address the problem. This study was undertaken with the main aim of investigating the quality of cold-mix asphalts used in bituminous pavement maintenance works in Ghana and the specifications guiding their quality so that a more successful product with a long-lasting field performance could be formulated. The study was guided by four specific objectives based on which the following conclusions may be drawn.

Objective 1: Review of Ghana's MOT Specification alongside some foreign specifications for cold emulsified asphalt mixes for pothole patching.

- Of the specifications on cold-mix asphalts reviewed, Ghana's Ministry of Transportation Standard Specification for Road and Bridge Works (2007) was found to be the least informative in guiding and controlling the quality of cold-mix asphalts used in pothole patching as it lacks specificity regarding the formulation and field performance requirements of such mixes.

- Unlike the other specifications reviewed, the MOT Specifications tended to defer all matters and responsibilities regarding cold-mix asphalts to the Project Engineer for guidance and direction when ironically it is rather the Specifications that must provide the requirements and benchmarks as reference to aid the Engineer in their decision making regarding the quality of the mix being deployed in the field.

- In terms of procedure, the Asphalt Institute MS 19 Specification is the most user-friendly as tests involved are relatively simple, easy to perform and lead to results that are relatively easy to interpret.

Objective 2: Characteristics of typical cold-mix asphalts used by local contractors in pothole maintenance.

- Patching mixes formulated with bitumen emulsion by the surveyed contractors tended to have relatively lower residual binder contents than those formulated with cutbacks. The differences are believed to result from failure to recognize and take into account, during mix component batching, the fact that a given quantity of emulsion contains much less binder (60-70%) than it may seem due to the presence of water.

- Field patching mixes used by the surveyed contractors lacked adequate aggregate structure as they tended to have very low fines fraction and high single-size coarse fractions within their bulk.

- Lack of sufficient fines in the cold-mix aggregate structure, use of mostly single-size coarse aggregates and insufficient residual binder content (particularly for emulsion mixes), combine to result in non-rigid, low-stability friable mixes that easily disintegrate under traffic action.

Objective 3: Design of a resilient dense-graded cold-mix asphalt with simplified material proportion ratios.

- Five different aggregate structures, identified as CM1, CM2, CM3, CM4 and CM5 in this study, are proposed for cold-mix formulation as all were found to be robust and met all requirements for cold-mix asphalts. Emulsion-to-aggregate ratio, corresponding to each of the aggregate blends may be used to formulate a successful patching mix in the field.

Objective 4: Potential property improvements attainable by incorporating nanoclay filler in cold-mix asphalts.

- Addition of montmorillonite nanoclay to cold-mix asphalts as filler leads to improvement in mix stability and increase in the optimum binder content but with significant increase in flow. This suggests that the material has the potential to induce plastic behaviour in cold mixes and render them more rut-susceptible, especially at high temperatures.

5.2 Recommendations

Based on the findings of this study, the following recommendations are made;

1. The Ministry of Transportation's Specification (MOT, 2007) needs to be reviewed to include more information to beef up the section on Cold-Mix Asphalts to make it more informative and user friendly.
2. The aggregate structures proposed in this study and the corresponding emulsion-to-aggregate ratio may be used as guides for a simplified production of good quality cold-mixes in the field.
3. The proposition to match emulsions with aggregates of opposite charges, so as to facilitate early breaking and achieve stronger binder-aggregate bonds in cold-mix asphalts, needs further investigation for possible adoption and implementation. Further studies using an anionic bitumen emulsion to test properties of the proposed trial blends is recommended.

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