CHAPTER 2
LITERATURE REVIEW

2.1 GENERAL

This chapter begins with a discussion of general usage of copper slag and its usage in highway construction in particular. As the study is focused on the use of copper slag in bituminous mixes, the mix design volumetric and performance factors are also reviewed. Finally, this chapter summarizes the observations in the present literature along with the future directions towards the research on bituminous mixes.

2.2 MATERIAL UTILIZATION

The literature available on copper slag is very limited. The copper slag was used as an abrasive material by exploring its silicon properties (Szyrle and Wzniak 1988). It has been explored that the copper slag can be used as an abrasive material for machining non-ferrous metals, wood or plastic. Herman (1989) investigated the usage of copper slag as a ceramic raw material in ceramic binder and found that the mechanical properties were improved when the copper slag was added into ceramic abrasive tools.

The slag contains some concentration of metals in the ores from which they were produced (Collins and Miller 1979). Adding copper slag as a substitute for Portland cement has shown to have a significant influence on increasing the compressive strength of concrete mixes (Mobasher et al. 1996). Some non-ferrous slag has been used in concrete mixtures and as railroad
ballast. GCS has been used as a bridge blasting abrasive (Appleman 1992). Copper slag has been used for mine backfill and granular materials (Asokan Pappu et al. 2004). The main use for copper slag in Australia is in grit blasting due to its sizing and strength characteristics (www.asc-inc.org.au).

From the above observations, it is inferred that copper slag has been used worldwide for various applications, including as an abrasive material, ballast and concrete aggregates.

2.2.1 Use of Copper Slag in Highway Construction

Copper slag is classified as a mineral waste and non-ferrous slag. Only four states (California, Florida, Michigan and Texas) in USA indicate any use of non-ferrous slag (Collins and Cielieski 1994). California has made limited use of a copper oxide blasting slag in bituminous mixes to improve the stability. Copper reverbaratory slag from the Upper Peninsula of Michigan has been approved by the Department of Transportation for all aggregate uses, except as a fine aggregate in cement concrete (Gallup 1974). It has been identified by Organization for Economic Cooperation and Development (1977) to use copper slag as a granular road base material or embankment fill applications.

Waste materials can be used for construction of low-volunteer roads with twofold benefits; (a) it will help clear valuable land of huge dumps of slag, and (b) it will also help preserve the natural reserves of aggregates, thus protecting environment (Sudhir et.al 1999). In India, the usage of copper slag in pavements is very limited and unexplored. The laboratory investigation for replacing the fine aggregates with GCS has been carried out by Central Road Research Institute (CRRI) and recommended about 20-30% usage in dense bituminous mixes (Nanda 2006). The use of copper slag in the subbase as a
component in the mixture of fly ash and soil has been tested and reported (Vasant et al. 2007).

From the above literature, it is observed that the copper slag has been used in construction of embankments and pavements. The material has been analyzed for its geotechnical characteristics along with other material combinations. Copper slag has been suggested as a fine aggregate in limited quantities for bituminous mixes but not as an alternative to the aggregate in bituminous mixes.

2.3 BITUMINOUS MIX DESIGN CONSIDERATIONS

2.3.1 General

Bituminous mixtures are used as surface or base layers in pavement structure to distribute the stress caused by loading and to transfer the same to the underlying unbound layers. To adequately perform both of these functions, the bituminous mixture shall be designed to withstand the effects of traffic loading, to resist permanent deformation and to resist the cracking caused by loading and environment. Many factors affect the ability of a bituminous paving mixture to meet these structural requirements. Properties of component materials, the use of additives, the design methods and the construction practices, all play an important role in deciding the structural characteristics of the Hot Mix Asphalt (HMA) mix design. It is also important to recognize the interaction between mixture components in the design in order to derive the most effective design solution. It starts from selecting to the proportioning the materials in order to obtain the desired qualities and properties in the finished mix. The overall objective is to determine an economical blend (MS2 2002) of aggregates and the corresponding bitumen content to satisfy the following requirements:
• Sufficient asphalt to ensure a durable pavement by thoroughly coating the aggregate particles, waterproofing, and bonding them together under suitable compaction.

• Sufficient mix stability to satisfy service requirements and demands of traffic without distortion or displacement.

• Sufficient voids in the total compacted mix to avoid flushing, bleeding and loss of stability. Since asphalt is a temperature susceptible material, there is very high possibility that bitumen may yield at high temperatures at which it should have space to move around the aggregates.

• Sufficient workability to permit an efficient construction operation in the placement of paving mix.

To achieve the above requirements, the material characteristics play an important role in the mix design. The bituminous mix contains three material components, namely,

• Air voids
• Mineral aggregates
• Bituminous binder

The mineral aggregates and binder are blended together to get optimum mixture properties. Since the mineral aggregate has to be replaced with the copper slag, the influence of aggregate gradation, Voids in Mineral Aggregates (VMA), Voids Filled with Asphalt (VFA) and air Voids in the Mix (VIM), Dust Asphalt ratio (D/A), Bituminous Film Thickness (BFT), aggregate blending characteristics are studied in detail.
2.3.2 Effects of Grading of Aggregate

The mix considerably includes aggregate characteristics. The aggregates are the major components of bituminous mix. Only about 5-7% (in major base and surface course mix bituminous mix designs) bitumen is used in bituminous mixes. The balance of about 90-95% of the mix is of aggregate portion. Current literature on the development of aggregate grading for open graded and dense graded mixes show that the design considerations are to produce blends that give maximum density in case of dense graded mixes and voids considerations in case of open graded mixes.

As early as 1905, Richardson (1905) recognized the importance of performance of the relative proportions of the mixture components by volume in the bituminous mix. The grading of aggregates for bituminous mixes was generally considered for maximum density and minimum voids. This concept led to development of Fuller’s curve (Fuller and Thompson 1907) and its modification by Brown et al. (1991). As per the findings by Furnace (1931) when two aggregates of different sizes were blended together, there was an optimum proportioning of the two components, which resulted in minimum voids, and this minimum voids was always smaller than any of the voids in individual groupings of the aggregate. The character of the aggregate has far greater consequence in adhesion than the bituminous binder, which however exerted an appreciable influence. The individual relation between the binder and the aggregate was such that it is very difficult to designate definitely either the bitumen or the aggregate as “good” or “bad”. Each problem necessitated a separate local study (Hubbard 1938, Endersby 1947).

In the observations made on the Gradation of Mineral aggregates for dense graded Bituminous Mixes, Hveem (1954) opined that some explanations given as liabilities or hazards in the grading chart of bituminous mixtures as shown in Figure 2.1 are over simplified. He also observed that
due to avoidance of certain difficulties more or less inherent in other properties of gradation, these liabilities were shown in the form of brief notes indicating possible or probable results, if the grading curve goes beyond the tolerance limits in the area of the chart covered by the notation. However, these notations are over simplification (Hveem 1954). It is also pointed out that the best gradation of any particular mixture can only be that which utilizes the available aggregates to give as many of the desired properties as possible.

For this reason, standardization of aggregate grading can easily be carried too far and as utilization of aggregate is primarily a local problem due care should be exercised in the adoption of national standards for materials which are strictly speaking not manufactured and which vary throughout the country (Hveem 1954). Commercial aggregates are not often shipped for long distances and there seems to be no good reason for requiring that crushed stone, sand or gravel in one region should meet gradations found satisfactory for materials at some distant point. Hveem observed that the best gradation is that which best suits the particular purpose and material available. It is also insisted that a vide variety of grading can be used but much variation in the same grading cannot be tolerated.

Sterling and Kazamiari (1997) emphasized that in asphalt, where a binder can also act as an excellent lubricant, “We must have enough air voids to prevent the mix going plastic. We have to stay as far away from the Fuller curve as we realistically can”. According to Lees (1970), by considering the porosity of aggregate in a single size, and to minimize the same the two-component analysis of aggregate grading has been transformed in to multi component system. These factors elaborate the design considerations before choosing the aggregate blends of different sizes in the mix design. The aggregate gradation (Finn et al. 1976) has no significant influence on the
Figure 2.1 Grading Chart of Bituminous Mixtures (Hveem 1954)
elastic behavior of asphalt concrete. The effect of mixture gradation is confounded by the interrelationship (Haddock et al. 1999) between aggregate properties and volumetric as well as the selection of grading. The design of asphalt mixture is largely a matter of selecting and proportioning constituent materials to obtain the desired properties in the finished pavement structures (The Asphalt Institute). However, when conventional mix design methods are fundamentally (www.asphaltexpertsystm.com/index-files/LOGIC) experience-based, it is difficult to obtain desirable mix properties from these methods. It provides neither universal guidance nor safe procedures/physically validated criteria for the quality of bituminous mixes. The aggregate grading design and evaluation are part of this core issue for any bituminous mix.

About the superpave gradations in USA, (http://www.nssga.org/pdf/whitepaper.pdf) the restricted zone in aggregate gradations is intended to eliminate poor performing humped grading that contains too much round natural sand in relation to total sand. Unfortunately, the restricted zone also eliminates many successful heavy duty, rut resistant mixtures produced with 100 % crushed aggregate from sources that have been used successfully for many years. Many States in the USA prohibit grading that pass through the restricted zone. At least ten states do not have a restricted zone requirement in their superpave specifications. States that have granite sources typically have very high performance mixtures that pass through the restricted zone. Best economy is also achieved with these mixes due to the balanced aggregate skeleton that these blends create. Use of all particle sizes produced at an aggregate source is important for keeping extra production and inventory costs at a minimum.

Research currently being performed at National Centre for Asphalt Technology (NCAT) and elsewhere, suggests that asphalt mixtures made with aggregate gradings that pass through the restricted zone exhibit performance
qualities equal to and even better than mixtures made with other grading of aggregate. The restricted zone is not needed, and its use in superpave specifications should be eliminated. Differences often exist for certain geologic types of aggregates between the grading of aggregate used in laboratory design and the grading of the same aggregate products and proportions used in the field mixture. Consequently, aggregates obtained from the aggregate production plant may need to be 5 to 10 % coarser (to account for grading changes that occur during transport, handling, and mixing) in order to meet mixture design volumetric requirements in the field. However, an aggregate product that is 5 to 10 % coarser at the aggregate production facility may not meet Superpave-grading requirements (and may be rejected from consideration). Mixture designers, specifiers, and field plant personnel should become knowledgeable in the properties of aggregates obtained from specific sources, and should permit adjustment of aggregate grading during laboratory mixture design to be representative of the aggregate grading that will be received and used at the asphalt plant.

From the above literature, it is observed that

- The mixture properties are independent of aggregate gradations. For the same gradation limits, the mixture properties may vary according to the aggregate interrelationship, including its volumetric properties.

- Best gradation of aggregate for bituminous mix should utilize the available aggregates to give as many of the desirable properties as possible.

- Use of all particle size produced at an aggregate source in the aggregate gradations is important to keep extra production and inventory cost at minimum.
• Conventional aggregate gradations are fundamentally experience-based and provide neither universal guidance nor safe procedure/physically validated criteria.

• The restricted zone in any aggregate grading shall be avoided to get better performance grading and greater flexibility.

2.3.3 Effects of VMA on Performance of HMA

In early mix design, methods proposed by Marshall (Jackson 1949) did not have any VMA requirement and explained that no limits can be established for VMA for universal applications because of the versatile application of bituminous materials to many types of aggregate gradations. McLeod (1955) presented the analysis on voids properties of compacted paving mixtures in which he laid out the principle of minimum VMA requirement. The findings on the design of asphalt mixture for greater durability by McLeod (1971) clearly illustrates that the VMA cannot be determined from the aggregate properties alone, it should be determined through the measurements of the compacted paving mixtures. The same view was expressed by Sennov (1987) in the theory of granulated materials in road construction, that theoretically it is difficult to predict the aggregate volumetric parameters, even the resultant voids ratio, when the gradation curve is known. The VMA is considered most important mix design parameter, which affects the durability of the bituminous mix. High VMA values allow enough asphalt to be incorporated in the design to obtain maximum durability without the mix flushing. The high VMA mixes are advantageous in minimizing the severity of thermal, reflection cracking, and lower susceptibility to variations in asphalt and fines content during production compared to the low VMA mixes. (Kandhal and Koechler 1985).
The minimum VMA requirement in any mix is difficult to achieve. The Minimum VMA can “fail” mixes that have acceptable performance records and they may require higher asphalt content to achieve such requirements, leading to higher project cost. Hence, the Minimum VMA requirements for bituminous mix design should be discouraged (Coree and Hislop 1999). Lefebvre (1957) found that the fine aggregates were the most critical component, controlling the VMA and contributing to stability. His recommendations included using a moderately high percentage of fine aggregate containing a small percentage of fine sand. The coarse aggregates, although good for stability, are bad for VMA, particularly if mineral filler is present. Campen et al. (1959) stressed that a satisfactory mixture is one in which the aggregate contains enough voids to permit addition of sufficient asphalt in the mix.

Minimum VMA values are required so that a durable binder film thickness may be achieved. Increasing the density of the HMA by changing the gradation of the aggregate may result in minimum VMA values with thin films of binder and a dry looking, low durability HMA. Therefore, economizing in binder content by lowering VMA is actually counter productive and detrimental to pavement performance. Low VMA mixes are also very sensitive to slight changes in binder. If binder content varies even slightly during production, the air voids may fill with binder resulting in a pavement that flushes and ruts. VMA is most affected by the fine aggregate fractions, which pass the No.200 sieve. The reason for this is that these particles tend to be absorbed by the binder film. Because they take up volume, there is a tendency to bulk (extend) the binder resulting in a lower VMA (IDOT 2006).

A maximum value for VMA is recommended to prevent high rut potential mixes (NCAT Report 0204). Based upon the critical rut depth of 9.5
mm, a maximum VMA value would be 18 percent. This, in essence, would be a 2 percent range for VMA, if 16 percent were the minimum value. Limiting VMA to no more than 2 percent above the minimum value has also been recommended by the WesTrack Forensic Team (PCGM 1998).

From this, it is observed that, the volumetric properties of bituminous mixes cannot be ascertained from the aggregate properties alone, but the same can be ascertained only by performing the mix design. High VMA mixes are advantageous in minimizing the severity of thermal and reflective cracking. Economizing in binder content by lowering VMA is actually counter productive and detrimental to pavement performance. Maximum value for VMA is recommended to prevent high rut potential mixes.

2.3.4 Effects of VFA on Performance of HMA

Voids filled with asphalt (VFA) are also an important factor to be considered in HMA mix design. VFA are the void spaces that exist between the aggregate particles in the compacted paving HMA that are filled with binder. VFA is expressed as a percentage of the VMA that contains binder. The main effect of VFA is to limit VMA and subsequently maximize the binder content (IDOT 2006). VFA also restricts the voids in the mix (VIM). In the volumetric requirements for superpave mix design, it has been recommended that in the design of HMA mixes, the maximum VMA values can be recommended and the VFA requirements may be eliminated in order to make the superpave system simpler and more direct and reduce the chances of designing HMA with poor rut resistance (NCHRP 2006). In the effect of different types of filler materials on characteristics of HMA concrete by Mehari (2007), it has been observed that, the effective asphalt cement in terms of VFA that provides the required BFT around the aggregate particles
determine the durability of the mixture. In order to achieve durable mixture, the minimum BFT shall be mentioned in the design.

From this it is observed that the VFA is an indirect method of controlling the VMA and BFT in the design of HMA mixes. The VFA requirements may be eliminated if the VMA requirements and BFT are considered in the HMA design.

2.3.5 Effects of VIM on Performance of HMA

In the early mix design methods proposed by McLeod (1956) the air voids are considered for volumetric calculations. He illustrated the volumetric relationship between total asphalt binders VIM and VMA in a compacted mixture. He proposed the volumetric criteria such as minimum VMA of 15%, VIM of 5% along with minimum asphalt content of 5%. Though the Indian specification (MORTH 2001) specifies the VIM in the range of 3-6 % for dense graded bituminous bases, there is no specific guideline for VIM in the open graded bituminous macadam base courses. Henley and Langlois (1980) opined that the open graded mix shows less number of reflective cracks than that of dense grade mix. The percentage of air voids in that open graded mix was designed to have at least 20% VIM depending upon the addition of crushed stone. In the analysis of resistance to permanent deformation by Gibb and Brown (1994), the VIM in the mix has well correlated with the resistance to deformation of the mix. It is evident that the material with lower VIM has the poorer resistance to deformation.

Summarizing the research on Open Graded Asphalt Concrete (OGAC) for mitigation of reflection cracking on asphalt concrete overlays, Bhosale and Mandal (2008) observed that the conventional overlay of dense bituminous macadam having VIM of 4 to 5% shows faster rate of crack
propagation than the OGAC overlay. The OGAC overlay has VIM of 26% and VMA of 33%, which effectively serves the purpose of crack relief layer.

Vallerga et al. (1980) observed that the open graded plant mixes have shown resistance to reflection cracking. They also opined that the open graded combinations may offer even greater effectiveness in the control of reflection cracking.

The reduction in VIM in bituminous mixes proportionately reduces the flow characteristics of the bitumen, which directly affects its vulnerability to cracking (Jha 2005). The dense graded mixtures develop reduction in VIM (Shanmugasundaram et al. 2005) after secondary compaction due to traffic, which results in plastic deformation of base courses. The minimum VIM level shall be specified to allow for further densification of the mixture under traffic without creating instability in the mixture. It has been observed that the open graded asphalt mixtures used in Oregon (Huddleson et al. 1993) the mixes with 9.5-15.6% VIM perform better than the BC mixes having VIM of 2-6% and the rutting resistance is generally improved over the dense graded BC mixes.

From the above literature, it is observed that

- The VIM in the bituminous mixture is an important component to be considered for the performance of bituminous mixes.

- The VIM in the mix is specified in order to allow further densification of the mixture under traffic without creating instability in the mixture.

- The open graded mixes, invariably having VIM of more than 10% are found to be useful to tackle the problem of
premature failure and other problems related to mixture densification.

2.3.6 Effects of the D/A Ratio on Rutting of HMA

The volume of fine or mineral aggregates (smaller than 0.075 mm) will also have an effect on the performance of the bituminous mix. Too much fine material makes a mix brittle and requires more asphalt (Anderson et al. 1997). On the other hand, little fine material can make the mix deformed permanently. The only way to determine the actual effect is to perform mix testing. The Standard Specification (SSMD 2004) for construction Michigan Department of Transportation recommends a D/A ratio of minimum of 0.6 to maximum of 1.2. The Standard Specifications of Highway construction, Oregon Department of Transportation Salem (SSOD 1984) also recommends a D/A ratio of 0.6 to 1.2 for all dense graded mixes. In current practice in India, the design requirement for bituminous mixes does not specify any requirement for D/A ratio (MORTH, 2001). The Maryland Design Specification (DSMD 1999) for 4.75mm mixtures specifies a maximum D/A ratio of 2, whereas the Georgia Specifications (SSGD 1993) specify a maximum of 2.4. When there is an increase in D/A ratio, it is established that the durability of the mix is also increased. The decreasing P0.075 contents lead to higher rut-depths. In the design of 4.75mm mixtures (NCAT 2002), it is observed that the rut-depth is generally increased as the percentage of dust is decreased. The coarse gradation provided higher rut-depths than the fine or medium gradation. The dust percentage is higher in fine and medium gradation than in the coarse gradation. The D/A ratio of 1.6 is recommended (FHWA 1998) in the study on effects of fine aggregate angularity on mixture performance.
The D/A ratio is an important factor to be considered in the design of bituminous mixes. Especially the rut depth and performance of the mixes are generally dependant on the D/A ratio of the mixes.

### 2.3.7 Effects of BFT on Durability of HMA

BFT is a factor, which was well recognized in the early mix designs. In the factors involved in the Design of Asphaltic pavement (NCHRPR, 1967), it has been opined that the optimum film thickness for maximum tensile strength is something more of 10 microns. Mixes produced with dense grading result in BFT ranging from 7 to 10 microns (Campen et al. 1959).

With all the design factors considered, a balance must be achieved. Therefore, care must be taken to satisfy all the design requirements and at the same time achieving optimum film thickness (NCHRPR 1967). It is generally agreed that high permeability, high VIM and thin asphalt coatings on the aggregate particles are the primary causes of excessive aging of the asphalt binder, which contributes to the lack of durability of the Hot Mix Asphalt (HMA) mixes often encountered in the field. Fine aggregate particles may have a much thicker coating as compared to the coarse aggregate particles. Each mix design (Mee 2006) has its own optimum bitumen content. The bitumen content at a fixed percent of air voids varies according to the nominal maximum aggregate size and gradation. The bitumen content plays a significant role in calculating VMA and Voids Filled with Bitumen (VFB). However, Kandhal, et al. (1998) argued that VMA should be calculated based on the surface area in order to have an average asphalt thickness.

Campen et al. (1959) presented the relationship between voids, surface area, BFT and stability for dense graded HMA. The authors recognized that thicker asphalt binder films produced mixes, which were
flexible and durable. Also observed that the thinner film mixes were brittle, tended to crack and ravel excessively. On the basis of the data they analyzed, average film thickness ranging from 6 to 8 microns were found to have provided the most desirable pavement mixtures. They also concluded that the film thickness decreases as the surface area of the aggregate is increased. However, the asphalt binder requirement was found to increase as the surface area was increased. But at a rate much lower than that guided by a relationship of direct proportionality. Goode and Lufsey (1965) have concluded that a minimum BFT value of 6 microns could be included as a criterion in all mix design procedures.

Kandhal and Chakroborthy (1996) quantified the relationship between various asphalt film thickness and the aging characteristics of a dense-graded HMA mix so that an optimum average BFT desirable for satisfactory mix durability could be established. They used the aging procedures to simulate both short and long-term aging of HMA mixtures. An optimum film thickness of 9 -10 microns was recommended as the desirable range of BFT in which the HMA mix (compacted to 8% air voids content to facilitate aging) aged at an accelerated rate. Obviously, the optimum BFT for HMA compacted to 4 – 5% VIM content in service should be somewhat lower than 9 – 10 microns because the rate of aging would be considerably lower at 4 – 5 % VIM compared to 8% VIM.

From the above literature, it has been observed that

- The BFT is an important factor to be considered at the design of bituminous mixes. The optimum range of BFT shall be in excess of 10 microns.
- The BFT value is lower than 9 to 10 microns in the mix, aged at an accelerated rate.
The design procedures are directed towards the volumetrics of the mix rather than considering this factor in the mix design procedures.

### 2.3.8 Effects of Aggregate Ratios

The aggregate combination of the compacted mix is a determining factor for the performance of Hot Mix Asphalt (HMA). The grading curves define the combination of aggregates. The structure of the aggregate skeleton is closely related to the rutting and fatigue characteristics as well as the durability of mix (Rouque et al. 2006).

The Bailey method was originally developed to improve the design of HMA mixes. It is not meant to complement the design method for HMA mixes, rather it allows the designer to assess whether coarse aggregate interlock will exist in the mix or not (Vavrik et al. 2002).

The interim guidelines for the design of hot mix asphalt in South Africa (Taute et al. 2001) define the coarse aggregate fraction so as to constitute the coarse and fine fraction in a blend. Bailey aggregate ratios, although only intended as a guideline, provide a useful addition to the mix design procedure (Denneman et al. 2007).

The coarse aggregate and fine aggregate ratios and their ranges provide a starting point where no prior experience exists for a given set of aggregates. If the designer has acceptable existing designs, they should be evaluated to determine a narrower range so as to target for further designs. (TRCE 2002)

From the above literature, it is observed that the Bailey method for a gradation selection specifying coarse aggregate and fine aggregate ratios for the bituminous mixture gives a starting point for the designer to select the
gradation. The ratios are calculated for each aggregate gradation and the same has been checked with the recommended range.

2.4 REVIEW OF FINE AGGREGATE BITUMINOUS BASE

The use of Fine Aggregate Bituminous Base (FABB) in the form of SA in India has been practiced as early as in 1965. In the research carried out by the Central Road Research Institute (Mehra et al. 1965) on bituminous stabilization of sandy soils describes the SA mixture with different sand grading. The stability of such mixes ranged from 400lb to 900lb (1.77kN to 3.99kN) with VIM ranging from 19% to 32%. In another study on the performance of thin SA surfacing by Swaminathan et al. (1969), the VIM in the SA mixes were found to be 19%. The stability values ranged from 1000lb to 2000lb. The softer grade bitumen 80/100 gave better performance than 60/70 grade bitumen in these SA mixes. The review of FABB in USA revealed that the Georgia Department of Transportation had proposed SA mix designs (Walker and Hicks 1976) utilizing poorly graded local pit sands, which would contain 5 to 25 %, by weight passing the No. 200 (0.074mm) sieve. The most stable mixes were derived from mixes, which contain sizeable portion of sand, which pass through the No. 200(0.074mm) sieve. Marshall Stabilities of these mixes typically ranged from 200 to 600 lb (0.892kN to 2.66kN) or more. Clean uniformly graded local sands have produced mixes with very low Marshall Stabilities (i.e., less than 100lb (0.44kN)).

Tests performed on SA mixes using Savannah pit sands indicated that mix stability dramatically increased with the addition of more than 50 % screenings (Walker and Hicks 1976). The Marshall stabilities of such mixes ranged from 600 to 1200 lb (2.66kN to 5.93kN) and the tests performed on SA mixes resulted in a stability of 978lb (4.35kN) with 60 % screenings and 5.5 % asphalt. Specification for FABB mixes base in Delaware State (HRBSR
1971) specifies a minimum Marshall Stability requirement of 500lb (2.22kN) with 85/100 grade bitumen.

North Carolina (HRBSR, 1971) specifies a Marshall stability of 500-600lb (2.22kN to 2.6kN) with 0-8% passing No.200 (0.075mm) sieve. Oklahoma State Department of Transportation (HRBSR, 1971) specifies a minimum Hveem Stability of 200lb (1.11kN) with air voids not more than 18%. The percentage passing in No.200 (0.075mm) sieve was restricted to 5 to 20%. Virginia Transportation specifies a minimum Marshall stability of 400lb (1.77kN) with 85/100 grade bitumen as binder.

South Carolina utilizes SA construction extensively throughout their Coastal Plain (Fletcher 1974). The performance of both surface and base SA on lightly and heavily traveled roads has generally been good. Their specifications limit the percent passing the No. 200 (0.074mm) sieve to 12%. The typical base course Marshall Stability requirement is 300lb (1.33kN) except for secondary roads for which there is no stability requirement. SA used in North Florida, the percentage passing No.200 sieve was restricted to 12%. These sands (West of Tallahassee) tend to be more angular and produce satisfactory stabilities. Sands with less than 12% passing the No.200 (0.074mm) sieve and less than 7% clay sized (0.005mm) have given good performance (Walker and Hicks 1976).

It was observed that the SA pavements in Mississippi provided good service despite excessive VIM and that the poorest performing SA had mineral filler added to reduce the VIM. The majority of Mississippi’s Coastal Plain is characterized by hilly areas of sandstone and shale as well as limestone. The central and northern portions of the state contain local deposits of well-graded alluvial sands, which typically produce 600 to 800 lb (2.66 to 3.55kN) Marshall Stability SA mixes. When combined with stone screening of crushed gravel, these sands can form mixes with Marshall Stabilities well
above 1000lb (4.44kN). The percent passing the No. 200 (0.075mm) sieve shall be limited to between 3 and 11% for the surface course and between 2 and 20% for the base and binder courses.

The following observations are made in the review of FABB course applications:

- Sand asphalt has been widely used as a Fine Aggregate Base Course and the Marshall method of mix design has been followed in most of the design practices.

- The Marshall Stability requirement of such fine aggregate bases was ranged from 250lb to 1200lb (1.11kN to 5.33kN).

- The 80/100 grade bitumen has been recommended in most of the designs and the binder content requirements were between 4 to 7%.

- The voids in the mix were typically above 10% and were limited to a maximum of 20%.

2.5 SUMMARY

It has been observed that in general the copper slag has been used as an abrasive tool for cleaning the harder substances. It has been used to enhance the stability requirements with an objective to improve the quality aspects of bituminous pavements. In general, there is no literature stating the bulk utilization of the GCS. The utilization has been discussed only in primary level studies.

On the review of bituminous mix performance and volumetric design considerations, it is observed that the mixture properties are
independent of aggregate gradations. Best gradation of aggregate for bituminous mix should utilize the available aggregates to give as many of the desirable properties as possible. Use of all particle sizes produced at an aggregate source in the aggregate gradations is important to keep extra production and inventory cost at minimum.

The mixture properties are very important in the design process in order to achieve durable mixes. The effect of VMA on performance of bituminous mixes shows that high VMA is advantageous in bituminous mix design in order to avoid high rut depth potential mixes. The introduction of VFA requirements in the HMA design is to control over the VMA and BFT requirements. For simpler and effective design considerations, the requirement of VFA may be eliminated. Instead, the VMA and BFT shall be included. As far as VIM is concerned, the literature survey shows that the open graded mixes perform better than the close graded, especially in the bases. The D/A ratio of the mixes play an important role while deciding the performance of mixes. BFT is also considered as an important factor for durability of bituminous mixes. The aggregate ratios are useful tool to obtain the desired aggregate gradation and mix volumetric.