DOKUZ EYLÜL UNIVERSITY

GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

INVESTIGATION OF MECHANICAL PROPERTIES OF POROUS ASPHALT PAVEMENT WITH WARM MIX ASPHALT ADDITIVE

by

Ahmet Buğra İBİŞ

September, 2022

İZMİR

INVESTIGATION OF MECHANICAL PROPERTIES OF POROUS ASPHALT PAVEMENT WITH WARM MIX ASPHALT ADDITIVE

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We have read the thesis entitled **"INVESTIGATION OF MECHANICAL PROPERTIES OF POROUS ASPHALT PAVEMENT WITH WARM MIX ASPHALT ADDITIVE"** completed by **AHMET BUĞRA İBİŞ** under supervision of **PROF.DR. BURAK ŞENGÖZ** and we certify that in our opinion it is fully adequate, in scope and in quality, as a thesis for the degree of Master of Science.

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ABSTRACT

Recently, the need to overcome the flood risks has gained more attention around the world. One of the practical solutions is the implementation of pervious pavement. Porous asphalt is a type of pavement with an open grade gradation mixed with a polymer-modified binder that contains higher air voids than conventional asphalt pavements after compression, which allows precipitation waters from the pavement surface to infiltrate into the substrates. Porous asphalt, helps to reduce the noise problem by ensuring that most of the noise caused by the friction between the wheel and the pavement surface is absorbed by the surface, due to its high void. In addition, since the pavement surface consists of coarse aggregates depending on the aggregate gradation used, it improves driving safety by increasing the skid resistance between the wheel and surface, which is one of the essential problems, especially in rainy weather.

This thesis investigated the applicability of porous asphalt pavement using bitumen with 50/70 penetration, different polymers (SBS and Elvaloy), organic warm mix asphalt (WMA) additive (Sasobit), and two different aggregate types (Limestone and Basalt). Preliminary design studies determined grading types and aimed to determine the optimum bitumen ratio used in the mixture depending on the particle loss test result and Marshall design data. Compliance with the specification was checked by evaluating the properties of the designed samples, such as particle loss, Marshall stability, indirect tensile strength, permeability, and bitumen draindown.

The results showed that the mixtures prepared with base bitumen and WMA additive Sasobit did not meet the porous asphalt specification limits, while the mixtures prepared with polymer-modified bitumen complied with the current specification. In addition, mixtures prepared with a basalt-limestone aggregate combination showed better results than mixtures prepared using only limestone.

Keywords: Porous asphalt, permeable asphalt, modified porous asphalt, porous asphalt design, polymer bitumen, warm mix asphalt

ILIK KARIŞIM ASFALT KATKILI GÖZENEKLİ ASFALTIN MEKANİK ÖZELLİKLERİNİN İNCELENMESİ

ÖZ

Son zamanlarda, sel risklerinin üstesinden gelme ihtiyacı tüm dünyada daha fazla ilgi görmeye başlamıştır. Pratik çözümlerden biri de gözenekli asfalt uygulamasıdır. Gözenekli asfalt, kaplama yüzeyine gelen yağış sularının alt tabakalara sızmasına izin veren sıkıştırmadan sonra geleneksel asfalt kaplamalara nazaran yüksek hava boşlukları içeren polimer modifiye bağlayıcı ile karıştırılmış açık dereceli gradasyona sahip bir kaplama türüdür. Sadece taşkın risklerini önlemekle kalmayan gözenekli asfalt, sahip olduğu yüksek boşluk oranı sayesinde araç tekerleği ve kaplama yüzeyi arasında oluşan sürtünmeden kaynaklı gürültünün büyük kısmının yol yüzeyi tarafından emilmesini sağlayarak, gürültü probleminin azaltılmasına da yardımcı olur. Ayrıca kaplama yüzeyi, kullanılan agrega gradasyonuna bağlı iri tanecikli agregalardan oluştuğu için özellikle yağışlı havalarda önemli sorunlardan birisi olan araç tekerleği ile kaplama yüzeyi arasındaki kayma direncini artırarak sürüş güvenliğini önemli derece iyileştirmektedir.

Bu tez, 50/70 penetrasyonlu bitüm, farklı polimerler (SBS ve Elvaloy), organik ılık karışım asphalt (IKA) katkısı (Sasobit) ve iki farklı agrega türünün (Kireçtaşı ve Bazalt) kullanılarak gözenekli asfalt kaplamanın uygulanabilirliğini araştırmıştır. Ön tasarım çalışmaları sonucu gradasyon tipleri belirlenerek karışım içerisinde kullanılacak optimum bitüm oranı; parça kaybı test sonucu ve Marshall tasarım verilerine bağlı olarak belirlenmesi amaçlanmıştır. Tasarımları yapılan örneklerin parça kaybı, Marshall stabilitesi, indirekt çekme mukavemeti, permeability ve bitüm süzülme gibi özelliklerine değerlendirilerek şartnamaye uygunluğu kontrol edilmiştir.

Sonuçlar, saf bitüm ve IKA katkısı Sasobit ile hazırlanan karışımların gözenekli asfalt şartname limitlerini karşılamadığını diğer yandan polimer modifiye bitümle hazırlanan karışımların mevcut şartnameye uygun olduğunu göstermiştir. Ayrıca kireçtaşı-bazalt

agrega kombinasyonu ile hazırlanan karışımların yalnızca kireçtaşı kullanılarak hazırlanan karışımlara göre daha iyi sonuç göstermiştir.

Anahtar kelimeler: Poroz asfalt, geçirimli asfalt, modifiye poroz asfalt, poroz asfalt tasarım, polimer bitüm, ılık karışım asfalt

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CHAPTER ONE
INTRODUCTION

Road transportation, the most common transportation in the world, has a broader usage network daily with the increasing population and transportation demand. The development of road transport structures is becoming increasingly necessary with each passing day. The rise in traffic loads and density has become a big problem over time. Since road transportation is more individual and intensive than other transportation systems, the applications should be continuously developed and long-lasting.

Highway pavements are divided into two main groups, flexible and rigid depends on the type of binder used in the pavement layer. Pavement that use bitumen as a binding material are called flexible pavement. The optimal design of the pavement layers is of great importance in driving safety, comfort, and economy.

Flexible road pavements are pavements whose pavement layer consists of a bituminous mixture, the base is bituminous or granular material and the subbase is granular or crushed material. Superstructures are designed and built by considering factors such as the safety, comfort, volume, and composition of the traffic on them, as well as factors such as economy, climate, and suitability for regional conditions (Yayla, 2011).

Flexible pavements have high-performance properties as soon as they are laid and bring into use. However, pavements are exposed to various effects over time. Consequently these effects, deteriorations such as rutting, cracks of different origins, and water-induced deterioration occur in the pavement. (Qiao, 2015). Flexible pavements' deterioration becomes more critical at high summer and low winter temperatures. These deteriorations reduce the service performance of the road, the driving quality, and the service life of the pavements. For this reason, it has become a crucial to prevent the deterioration of pavements and boost road quality by developing today's highway superstructure technologies.

Failure of the wearing course to perform its function can directly affect traffic accidents. Due to the live loads directly on the wearing course, the surface friction force must be high. Precipitation water will affect the driving quality of the pavement surface as it causes a critical reduction in the friction.

Flexible pavements are generally designed to allow precipitation water to flow through the transverse and longitudinal slopes imparted to the pavement surface and into catchments and trenches along the edges of roads or car parks. Porous asphalt pavements are different types of pavements that allow precipitation waters to flow through the pavement. The purpose of the structure is to managed the precipitation water coming from or generated from the surrounding impermeable pavement surfaces, to reduce the noises caused by the contact of the vehicle wheel surface with the pavement, and to provide a safe transportation route for drivers and pedestrians by providing less splashing during rain (Thelen & Fielding Howe, 1978; Ferguson, 2005).

Porous asphalt pavement (PAP) with a high void ratio consisting of bitumen, gravel, and crushed aggregate. The mix is designed by reducing the fine aggregate ratio and using larger aggregate sizes. Owing to its high void with large gradation limits, it absorbs the precipitation water coming from the pavement surface into the pavement layers and carries it from the pavement surface with the transverse and longitudinal slopes on the road surface. Thus, it is aimed that the precipitation water coming to the pavement surface can pass into the pavement layer quickly and effectively in order to minimize the effects of rainy weather that reduce driving safety (USEPA, 1999). PAP has a 15-20% more void ratio than other asphalt pavement types upon completion of the laying and compaction process (NAPA, 2003; Bäckström& Bergström, 2000; Schaus, 2007).

Modification of bitumen improve the pavement performance and thus reduce deterioration. Currently, the addition of polymers is one of the usual methods of bitumen modification. Modified hot mix asphalt (HMA) has significantly reduced rutting and increased fatigue and thermal crack resistance (Sengoz et al., 2008; Sadeghian et al., 2019; Almusawi et al., 2020; Köfteci et al., 2020). WMA additives are another option used to improve the performance. The purpose of using WMA additive is to reduce the amount of emissions and environmental effects with less energy consuming of the prepared asphalt, reduce the viscosity, and provide suitable mixing and compaction conditions at lower temperatures, at least the same stability and higher durability than the mixtures prepared with conventional HMA modifiers (Dokandari et al., 2014).

Within the scope of the thesis, a laboratory study was carried out to design a permeable asphalt mixture prepared with combinations and gradations of basalt and limestone aggregates with three various additives for bitumen modification (elastomeric and reactive elastomeric terpolymer as well as an organic WMA additive). Asphalt concrete samples were prepared using different rates of base (unmodified) and modified bitumen and aggregate. The optimum bitumen content (OBC) was determined due to Marshall stability, particle loss, and air void ratio values of the prepared mixtures. Permeability, ITS, and bitumen drain down tests were carried out on the samples prepared with optimum bitumen content. The design values of the porous asphalt mixture(PAM) and the effects of different bitumen modifier on the mixture performance were determined extensively on the basis of the evaluation of the test results.

CHAPTER TWO
LITERATURE REVIEW

2.1 Highway and Pavement

The highway is defined as all of the structures built to bring the natural ground to the desired elevations and to ensure the movement of motor vehicles at the desired speed, safety, and comfort conditions along a route determined in accordance with predetermined geometric standards. The pavement built on top of the foundation to provide a suitable rolling surface for vehicles, resist the abrasive effects of traffic, and reduce the amount of surface water infiltrating the structure and the shear stresses transmitted to the foundation layer (Ilıcalı, 2001).

Pavements examined two groups, rigid and flexible classify by types of materials used in the layer, their properties, and construction methods.

2.2 Types of Pavement

2.2.1 Rigid Pavement

Rigid(concrete) pavements are a type of pavement that includes a Portland cement-bonded concrete slab pavement. Concrete slab pavement with high bending resistance dramatically reduces the stresses caused by traffic loads and transmits them to the subgrade uniformly. In cases where the subgrade does not have the sufficient bearing capacity, the foundation and sub-base layers are also made (Agar et al., 1998).

Concrete slabs are generally made on the sub-base layer, and in cases where sub-base pumping, frost heave, increase in water content, excessive swelling, and shrinkage are not observed, and the bearing capacity is sufficient, it can also be done by applying an impermeable material. If a sub-base layer is needed, well-draining stabilized material is laid at the specified thickness and compacted. The behavior of a consist of Portland cement pavement varies depending on the characteristics of the poured concrete layers as well as the sub-base and foundation layers laid under the pavement and the existing base soil. Therefore, during the design, the physical properties of materials such as base soil, foundation material, sand, gravel, cement, and reinforcing iron that make up the concrete, which affect the behavior of the concrete pavement, should be examined very well (Giris, 2007).

In rigid pavements, when the wheel that transmits the load to the road comes over the concrete slab, the concrete slab spreads this load in direct proportion to the magnitude of the surface area and transfers it to the layers under it. In this way, the load on the unit area transmitted to the layers under the concrete slab is reduced. This reduces the bearing capacity expected from the material under the concrete slab (Yenibogalı, 2009). Figure 2.1 shows the load distribution on the rigid pavement.

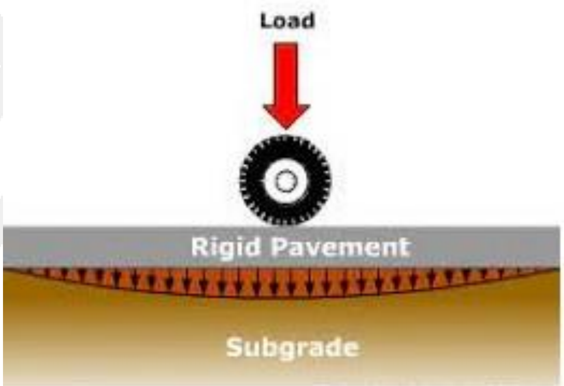


Figure 2.1 Load distribution of rigid pavement

Rigid pavements are also divided into three groups according to their construction methods.

Jointed Plain Concrete Pavement: Concrete blocks consist of sections 3 to 6 meters long. The blocks are approximately 125 to 350 mm thick. These blocks do not contain a reinforcement. Joints can be applied on the weakened surface and prepared as plug-in or non-permeable. Blocks are generally placed on cement-bonded or bitumen-binding layers. Top and side views of the jointed plain concrete pavement are given in Figure 2.2.

The short joint spacing is used to minimize mid-block cracking and keep the joint openings relatively small. This spacing includes bonded longitudinal joints for the same reason. Material clamping is used for connection load transfer in joints. Concrete bars or a stabilized foundation layer are used to improve load transfer on roads with heavy traffic loads, especially in humid areas.

Separation membrane is required in rigid pavements without joint reinforcement and in rigid pavements with joint reinforcement, in order to reduce the friction theramong the concrete slab and the sub-base, to prevent cracks in the opening in the middle area (Ecevit, 2007).

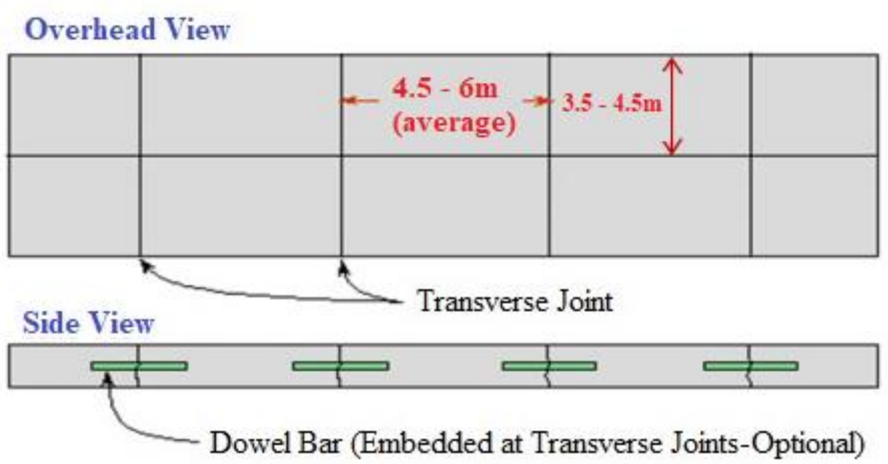


Figure 2.2 Jointed non-reinforced rigid pavements (Walubita et al., 2017)

Jointed Reinforced Concrete Pavement: It is a concrete block consisting of blocks of 8-30 m in length. The block thickness is 150-350 mm, and the reinforcement steel mesh passes in the middle. Base layer thickness is 100-200 mm. Top and side views of the jointed reinforced concrete pavement are given in Figure 2.3.

When a longer joint gap is given, cracks occur in such pavements due to shrinkage due to drying and curling due to heat. The purpose of reinforcing steel is to prevent the formation of cracks in the middle of these blocks. By holding the cracks tight, the steel transfers the load. Steel is not used to increase the flexibility capacity of the concrete block. Reinforcing steel bars are used to provide load transfer between joints. Joint and non-reinforced type rigid pavements are used extensively in regions with freezing temperatures and on intercity roads in humid regions (Ecevit, 2007).

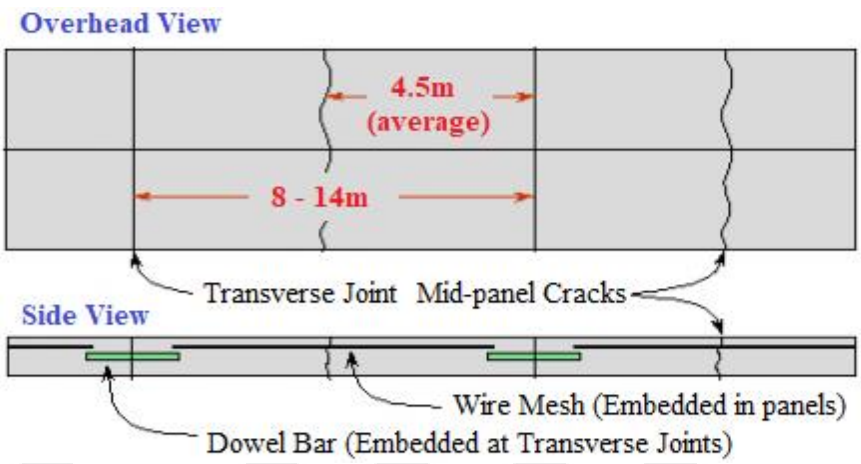


Figure 2.3 Jointed reinforced concrete pavement (Walubita et al., 2017)

Continuously Reinforced Concrete Pavement: It consists of a concrete slab laid without transverse joints. The high reinforcement content allows no joints to be placed, but as in all other reinforced concrete structures, the reinforcement does not prevent cracks but only keeps them under control. Steel reinforcement is constantly present throughout the block, and joints are placed only daily at the end of construction. Concrete block thicknesses are usually 150-250 mm. There is considerably more steel in such pavements than in rigid pavements of the joint-reinforced type. The steel ratio is generally 5% to 7% of the cross-sectional area. Top and side views of the continuously reinforced concrete pavement are given in Figure 2.4.

In this type of pavement, transverse cracks at 0.6-2.4 m intervals during the first few years of the constructed pavement occur due to the application of long joint spacings. These cracks do not grow as long as the reinforcing steel in the block is intact. If the steel is unable to withstand high tensile stresses, transverse cracks may occur, and the blocks may fail. Deteriorated crack zones combined with longitudinal cracks can lead to superficial pitting (punching effect), which indicates severe pavement deterioration. This pavement often uses a stabilized foundation to increase slab support and reduce block stresses under wheel loads.

The separation membrane, which is applied to prevent the formation of cracks on rigid pavements without joint reinforcement and on rigid pavements with joint reinforcement, is not applied on rigid pavements with continuous reinforcement in order to obtain more friction between the foundation and the subbase.

The discontinuities in the concrete block should be avoided in areas that cause cracks at close intervals, along with the risk of swelling and spilling. For this reason, manholes and waterways should be kept outside of this type of rigid pavement plates (Ecevit, 2007).

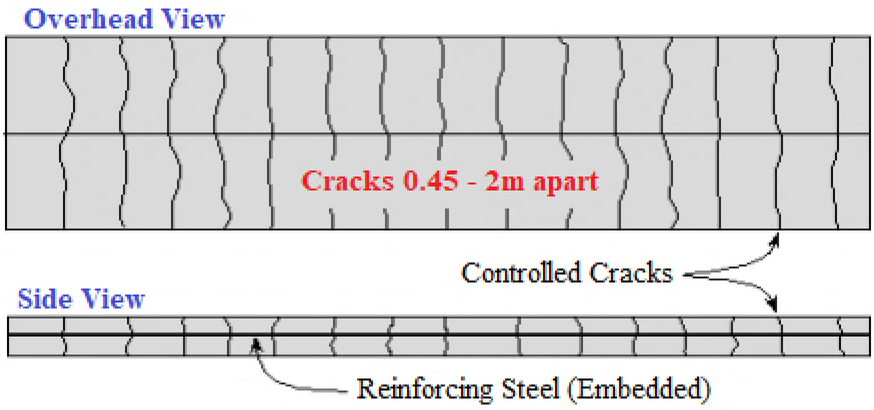


Figure 2.4 Continuously reinforced concrete pavement (Walubita et al., 2017)

2.2.2 Flexible Pavement

Flexible pavements are formed by placing hot bituminous mixture layers or surface pavement on the base layer. Bituminous hot mix is a layered structure formed by a bituminous base, binder, and wearing. Flexible pavements interact with the leveling surface created and distribute the loads on the base. Flexible pavements consist of unbounded sub-base and base materials combined with a hot bituminous mixture or bituminous surface pavements (Hunter et al., 2015).

In flexible pavements, there is a local and point load effect. The loads on the pavement are transferred directly to the foundation soil. The transferred loads affect the foundation locally and the foundation's bearing capacity. The stress value below the point where the tire touches the road is very high. The load cannot be distributed over a given surface as rigid pavements. Therefore, the bearing capacity expected from other layers under the pavement layer is high (Mutlugeldi, 2015). The distribution of the load acting on flexible pavement systems is shown in Figure 2.5.

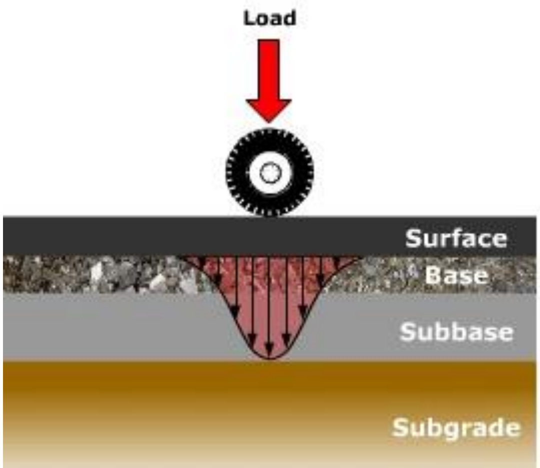


Figure 2.5 The wheel load distribution systems on flexible pavements

2.3 Types of Flexible Pavement

Flexible pavement differs from each other in terms of materials and types used in the mixture. There are four most common types of HMA pavements in terms of gradation types. Nevertheless, surface treatments are considered by most agencies to be a form of maintenance.

2.3.1 Dense Graded Asphalt Mixtures

This type of asphalt mix is used more often because it has significant impervious properties to allow water to escape from the surface. It can also be subdivided as fine-graded or coarse-graded, depending on the majority of the aggregates at end of the design.

Dense graded pavement contains all sieve sizes of aggregate. Standard specifications provide aggregate grades for densely graded mixes. Therefore, load transfer through the pavement structure. Air void contents are generally between 3 to 5%. Figure 2.6 shows the schematic drawing of dense graded HMA mixtures.

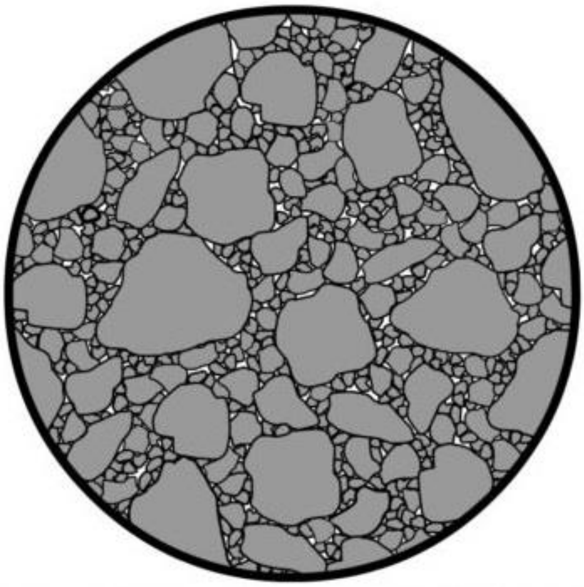


Figure 2.6 Dense graded HMA schematic drawing (Pavement interactive, 2016)

The duration of the pavements is directly affected by the traffic occupancy on the road, the climate effect and the current conditions of the pavement structrure (Hicks et al., 2000).

2.3.2 Gap Graded Asphalt Mixtures

Stone matrix asphalt (SMA), also known as stone mastic asphalt, is a gap-graded HMA. This type of pavement has emerged to reduce rutting on bituminous pavements. Mixture of a skeleton made of coarse and fine aggregate, mastic(bitumen). Coarse aggregate meets the traffic loads. Highly mastic content also fills the voids and increases durability. Fiber is put in to the SMA mixture to prevent from bitumen draindown out of the mix, as the bitumen content is high (Orhan, 2012).

The most crucial feature of stone mastic asphalt is that it contains 70-80% coarse aggregate and 6-7% bituminous binder. This feature allows the loads applied to the road surface to pass to the infrastructure without causing deformation on the road surface. Gap-graded pavement contains a few mid-sized particles. Air void contents are generally between 3 to 5% (Tasdemir, 1998).

Gap graded mixtures are used as a surface treatment on a dense grade asphalt pavement or rigid pavements. They are used to fade in the following deteriorations in the pavement; revelling, oxidation, fatigue cracks, skid problems, etc. (Guirguis, M., 2018). Figure 2.7 shows the schematic drawing of gap graded HMA mixtures.

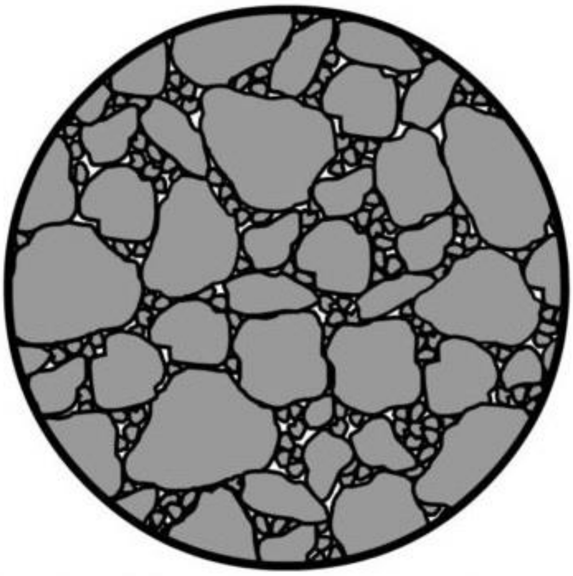


Figure 2.7 Gap graded HMA schematic drawing (Pavement interactive, 2016)

2.3.3 Open Graded Asphalt Mixtures

The permeability characteristic differs from the first two with the open-graded asphalt mixture. Open-graded mixture (OGM) is designed to be water permeable. OGM use only crushed stone and a small percentage of filler materials. In open-graded asphalt mixtures, a trace of fine aggregate in the mixture plays an essential role in obtaining the desired void ratio. Considering the high coarse aggregate ratio in the mixture, the strength of the aggregate used in these mixtures and its adherence with the bituminous binder should be high due to the contact between the aggregates. Air void contents are generally between 15 to 20% (Topaloglu, 2019). Figure 2.8 shows the schematic drawing of open graded HMA mixtures.

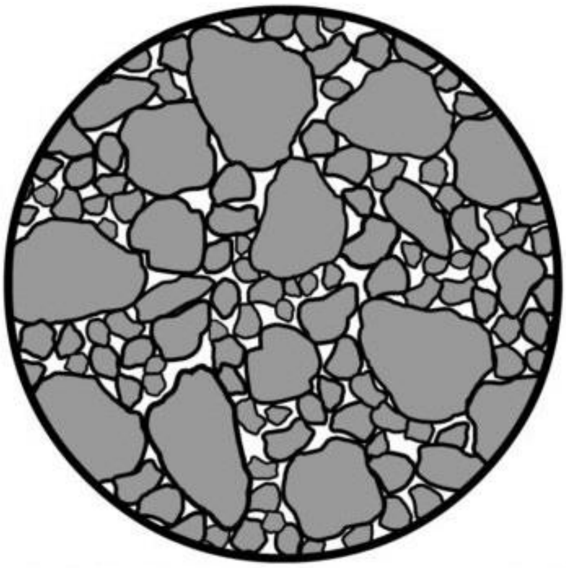


Figure 2.8 Open graded HMA schematic drawing (Pavement interactive, 2016)

Open-graded asphalt mixes have a shorter service life than other graded mixes. Functionally, the mixture fails when the gaps are clogged. Structurally, the pavement mixture will fail, raveling. When the voids are clogged, an open grade mix like as a dense grade mix with relatively low permeableness (Huber, 2000).

2.3.4 One Graded Asphalt Mixtures

Surface treatments come to mind as single or one-gradation mixtures. Surface treatments, also known as surface pavement, is a type of pavement made by laying asphalt or tar or a mixture of both as a thin film on the road surface, then covering it with aggregate in layers (Umar & Agar, 1985).

Surface treatments do not have a direct load-bearing feature. It increases the roughness(also friction) of the pavement surface. With the use of these pavements, a low-cost and temporary solution is provided by creating a smooth road surface (Hunter, 2015).

Surface treatments are the bitumen bonding sprayed on the base layer, or the bitumen and aggregate layer is laid in a single or double layer on the primer layer and compressed, providing an impermeable surface for the substrates, a smooth rolling surface with high slip resistance for vehicles, traffic and traffic jams. It is an economical asphalt pavement type that protects it from the abrasive effects of the climate. Generally, when surface treatments fail to fulfill these functions, they are considered to have completed their service life (Karasahin & Agar, 2004).

In order for the surface treatments to perform adequately, the aggregate should be in the most uniform gradation possible. It should be embedded in the bitumen by at least 30% of the average of the most diminutive dimensions with the effect of rolling after manufacturing, and then 60-70% by the traffic effect. When the amount of embedding falls below 30%, the problem of reveling occurs, and when it exceeds 70%, the problem of bleeding occurs (KGM, 2020). Figure 2.9 shows the embedding of aggregates in bitumen at a rate of 70% of the average of the smallest dimensions in the surface treatment and layer drawing.

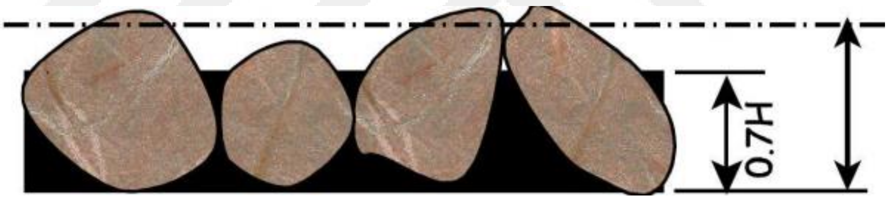


Figure 2.9 Embedding aggregates in bitumen in surface treatment and layer drawing (KGM, 2020)

CHAPTER THREE
POROUS ASPHALT PAVEMENT

3.1 Definition of Porous Asphalt Pavement

PAP is a type of open gradation pavement with a permeable characteristic by the reason of its high void structure. The PAP, which is built on an impermeable binder layer, quickly filtrates the rainwater coming to the road surface through its high void ratio. Due to the interconnected gap channels in the pavement and the longitudinal and transverse slopes given to the road, precipitation waters are quickly removed from the pavement (Khalid & Walsh, 1996). The removal of precipitation water from the pavement using the transverse slope is shown schematically in Figure 3.1.

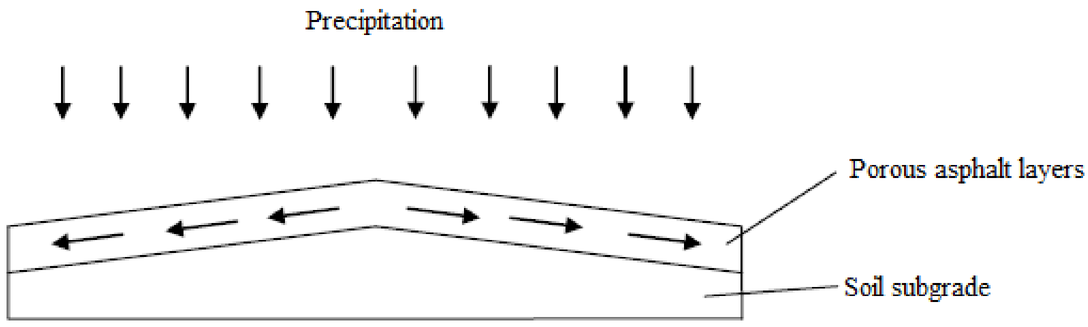


Figure 3.1 Removal of precipitation water from porous asphalt pavement

The difference between the permeable asphalt layer from other pavements is that it has 20% more voids (porosity) and larger aggregates after it has been laid and compacted. In conventional pavements, the void ratio is usually lower (like 3-5%), and the pavement should be remarkably impermeable. In permeable pavement, the aim is the opposite (Uluçaylı, nd).

3.2 History of Porous Asphalt Pavement

Since the 1960s, PAP was proposed to "transfer precipitation waters from the road surface to the lower layers, reduce sewage loads, reduce floods, raise groundwater levels and renew aquifers", this concept has been tried to be developed by various studies.

England started the first studies on permeable asphalt pavement in the mid-1960s. Although the high percentage of coarse size aggregate and the insufficient bituminous material decreased the demand for permeable asphalt in the 1980s, the use of PAP has become widespread with the development of technology in modified bitumen (Kandhal & Mallick, 1998). In the same years, permeable asphalt coating works were started in Europe and it developed the "Porous European Mix-PEM" with a higher void ratio (EAPA, 2016).

Porous asphalt was developed and tested at the Pennsylvania Franklin Institute during the 1980s. However, the idea of porous asphalt was discovered as a pathway by the California Highway Department as early as 1944 (Ahmad et al., 2017).

The type of pavement that was of interest for its purpose and application was used in 1977 by the Walden Pond State Reservation in Massachusetts for visitor parking lots. It has since raised groundwater levels while reducing erosion, pollution, sewer system load during storms, and the need for road salt during winter seasons (Ferguson, 2005).

Its primary uses increase surface friction in rainy weather and reduce drainage infrastructure costs. Porous asphalt is also a preferred type of pavement to prevent aquaplaning caused by the effect of water in highways and airports in many European countries. In countries with high traffic density, such as France and the USA, applications have been made in many cities to solve the noise problem (Rogge & Jackson, 1999).

In recent years, in parallel with the developments in binder technologies and mixture design methods, the popularity of PAM has been increasing worldwide. The use of porous asphalt has accelerated in European countries such as Spain, Switzerland, Belgium, the Netherlands, France, some states of the USA, and Far East countries such as Japan and Singapore (Khalid & Jimenez, 1995).

Research on porous asphalt has just gained interest in our country day by day. Regulations on PAP continue. It is emphasized that permeable asphalt pavement can be applied in places with heavy rainfall, and the necessity of using modified binders in regions with high seasonal temperatures. (Ozturk, 2008).

3.3 Applicability Conditions of Porous Asphalt Pavement

PAP also known as permeable paving system, is appropriate for pedestrian and low-volume, low-speed areas. This type of pavement allows for runoff volume, rate control, and pollutant reductions. Using PAP can reduce additional expenditures and land consumption for conventional stormwater managed system.

PAP can be applied on slightly sloping sites, permeable soils, and deep groundwater or bedrock levels. In addition, it should be suitable for the design in terms of the drainage feature of the applied floor. Because the base floor will differ in water permeability in terms of base type. PAP constructed where the soil under the asphalt pavement has a permeability of at least 0.43 cm per hour (Barton & Argue, 2009).

Since porous pavement has a lower load-carrying capacity than conventional pavement due to its porous structure, it is not suitable for areas with high traffic volume and high traffic flow rate. Porous pavement is not preferred in areas where the clogging of the pavement surface is higher due to environmental effects because the precipitation water that will come to the pavement surface will not be able to leak through these pores, so it will not show permeable asphalt. Porous pavements can be shown not only to remove precipitation waters from the pavement surface but also to reduce the noise and aquaplaning effect on the wet pavement surface (Oberts, 2003).

3.4 Design Criteria of Porous Asphalt Pavement

PAP, generally used for drainage purposes, differs from traditional pavement types due to its porous structure. In order to build a PAP that serves the purpose, this void structure must first be formed solidly and well protected. Designing a structural element with a load-bearing function as porous and ensuring its long-term use without deteriorating its stability are among the primary conditions. For this reason, some criteria must be met to design and apply a PAP.

Design criteria to be considered while designing porous asphalt,

- In the area where porous asphalt will be applied, the land should have a gentle slope, even if it is not flat land, the slope should not exceed 5%. As a result of the pavement being made by considering the existing slope, the water can be filtered faster into the stone bed thanks to the slope difference (Wei, 1986).

- The subfloor must meet the permeability conditions. The permeability properties of the soil on which the porous asphalt will be applied should be determined by conducting field tests, and the soil with high permeability should be preferred for rapid drainage. In cases where the Darcy coefficient of the ground is lower than the specification limits, the construction cannot be carried out because the drainage conditions are not at the desired level (Lee et al., 1990)

- The average temperature for the summer season should not be above 30 ºC. Because above this temperature value, the penetration value increased due to the bitumen's softening; accordingly, the deformations increased, and the strength decreased. These problems that occur on the pavement surface will give negative results in terms of both drainage and road safety (Wei, 1986).

- The porous asphalt, used to provide efficient drainage of the water coming to the pavement surface, has the risk of freezing the precipitation water drained from its porous structure at low temperatures. As the water that freezes in the pavement will close the voids, the pavement will lose its porosity and behave like a conventional asphalt pavement. For this reason, the design should be made considering the frost depth in the application area, and additional measures should be taken (Guide, 2003).

- It is desirable not to use PAP in areas where the ground is eroded by wind and excessive debris, oil, and fuel deposits. Because the drifting thin soil particles with the effect of the wind clog the PAP gaps, reducing the permeability property to almost non-existent (Brown & Caraco, 2001).

- The clay and silt content ratio of the natural soil should not be more than 30%. Permeability is low in silty and clayey soils, so drainage is slow. For this reason, as the clay silt ratio in the ground increases, the efficiency of the PAP will decrease (Lee et al., 1990).

- High traffic volume and heavy vehicle traffic should be avoided (Lee et al., 1990).

- In areas with high flexibility, such as bridges, the application of porous asphalt should be avoided (Malaysia, 2008).

- Laying sand, sprinkling salt, or using chemical additives should be avoided to prevent snow water freezing on the pavement surface in areas with snowfall (Lee et al., 1990).

- The weather temperature not less than 10°C while laying the PAM (Wei, 1986).

- The laid mixture should be compacted once or twice with a 10-ton compacting roller (Hein & Eng, 2016).

- The laid mixture should be opened to traffic 1-2 days after it is compacted (Hein & Eng, 2016).

In general, the design criteria are the conditions mentioned above. However, countries or organizations create different design criteria for porous asphalt design. The main reasons for the emergence of different design criteria are the mechanical and physical properties of the materials to be used in the preparation of the mixtures (bitumen, aggregate, etc.) or the area where porous asphalt will be applied (ground structure, annual precipitation, etc.).

3.4.1 Turkish (KGM) Specification

The compaction number of the porous asphalt samples was determined as 50 blows. Then, the compacted mixtures are analyzed to the Marshall stability test to find stability and air void values. The air void ratio of the PAMs to be prepared and designed should be between 18% and 22%, the Marshall stability value should be a minimum of 300 kg, and the indirect tensile strength ratio(ITSR) should be at least 80%, according to the specification. Particularly developed for porous asphalt, the particle loss test is an application to control the suitability of the prepared mixtures for the design. According to the Turkish specification, the maximum particle loss value is 20%. The permeability value of the pavement is an essential standard for the PAP to serve their purpose. These values were determined as 0.5-3.5 (m/sec), x10^{-3} in both horizontally and vertical direction. The bitumen draindown test is also a crucial evaluation for porous PAP. When a bitumen ratio above the optimum amount is used in the mixture, excess bitumen will drain from the aggregate surfaces and cause the pores of the porous asphalt to close. Therefore, the permeability will decrease. Bitumen draindown value was determined as 0.3% maximum. The fiber will be used when deemed necessary by the administration according to the changing designs. The standardized design criteria and aggregate gradation limits are shown in Tables 3.1 and 3.2, respectively (KGM, 2013).

Various bitumen modification applications are carried out to provide the above-listed requirements. These applications include sulfur addition, rubber addition, even organic or inorganic sulfides, and bitumen modification by adding polymer. The most commonly used of these additives is polymers. Polymers are made up of chemically connected long molecular chains. Based on the physical properties, polymers are classified into plastomers (plastics) and elastomers (elastic). Under load, plastomers will stretch, but will not return to their original form. While elastomers, when the load released will return to the original shape (Asphalt Institute, 2007).

The behavior of the bitumen, when the polymer added, generally can be described in two ways. If the bitumen becomes thicker and the viscosity increased, then the polymers form discrete particles in the bitumen material. In this case, the low-temperature degree properties of the obtained polymer bitumen samples will not significantly have influenced (Hunter et al., 2015). Table 3.1 shows the polymer types used in bitumen modification, their common names, and some usage areas.

Table 3.1 Porous Asphalt Design Criteria Based on The Turkish Standard

Properties		**Specification Limits**
Number of Blows		50
Air Voids, (%)	Type-1, Type-3, Type-4	24 - 28
	Type-2	18-22
Particle Loss (Cantabro), (%), max.		20
Marshall Stability, (kg), min.		300
Permeability Value, (m/sn), $x10^{-3}$	Vertical	0.5 - 3.5
	Horizontal	
Fiber Quantity, %		0.3 - 1.0
Schellenberger Bitumen Drain down, (%), max.		0.3
Indirect Tensile Strength (ITS) Ratio, (%), min.		80

Table 3.2 Porous Asphalt Gradation Types

Sieve Size		**Type-1**	**Type-2**	**Type-3**	**Type-4**
in, No	mm	Passing	Passing	Passing	Passing
3/4"	19.0	100	100	100	-
1/2"	12.5	5-15	90-100	85-95	100
3/8"	9.5	-	63-77	5-15	85-95
No.4	4.75	5-12	11-35	5-15	5-12
No.10	2.00	5-10	10-20	5-10	5-10
No.80	0.18	-	5-10	-	-
No.200	0.075	3-5	3-7	3-5	3-5

3.4.2 ASTM D7064 – Standard Practice for Open-Graded Friction Course (OGFC)

While designing porous asphalt according to this specification, the mixtures to be prepared are compacted by the gyrator compaction method. The air void ratio should be at least 18%. Particle loss is made in two different ways, aged and unaged. The main reason for this distinction is that the aged samples represent field conditions (the effect of moisture-induced deterioration) and give more accurate results. In this case, the aged particle loss is evaluated as a maximum of 30%, and the unaged particle loss value is calculated as a maximum of 20% in the specification. Bitumen draindown and indirect tensile strength ratio were determined as maximum 0.3% and minimum 80%, respectively. The amount of fiber used while designing is limited to between 0.2% and 0.5% (ASTM D7064, 2013). The standardized limits and aggregate gradation are depicted in Tables 3.3 and 3.4, respectively.

Table 3.3 Porous Asphalt Design Criteria Based on ASTM D7064 (ASTM D7064, 2013)

Properties	**Specification Limits**
Gyration	50
Air Voids, (%), min.	18
Particle Loss (Cantabro), (%), max. ***Cantabro Abrasion Loss on Unaged/dry Specimens** **** Cantabro Abrasion Loss on Aged/moisture Specimens**	20^{*} - 30^{**}
Bitumen Drain down, (%), max.	0.3
Fiber Quantity, %	0.2 – 0.5
Indirect Tensile Strength (ITS) Ratio, (%), min.	80

Table 3.4 Porous Asphalt Gradation Types based on ASTM D7064

Sieve Size		**Option-1**	**Option-2**	**Option-3**
in, No	mm	Passing	Passing	Passing
3/4"	19.0	100	100	100
1/2"	12.5	85	92.5	100
3/8"	9.5	35	47.5	60
No.4	4.75	10	14.5	25
No.10	2.38	5	7.5	10
No.200	0.075	2	3	4

3.4.3 Chinese Specification

The blow number of the PAMs to be prepared as in other specifications was determined as 50. According to this specification, the particle loss and bitumen draindown rates of the PAM samples are designed to be a maximum of 20% and 0.3%, respectively. Results below these values are considered suitable for design, and there are no lower limit values. Marshall stability and ITSR were determined as a minimum of 3.5 kN and 75%, respectively. Mixture designs are required to be made following these lower limit values. Unlike other specifications, Marshall flow value is also added as a porous asphalt design criterion. This value is required to be between 20 and 40. The amount of fiber to be used will be between 0.3% and 0.5% in designs where it is required (Ma et al., 2020). Related specification limits and aggregate gradation types are illustrated in Table 3.5 and Table 3.6, respectively.

Table 3.5 Porous Asphalt Design Criteria Based on Chinese Specifications

Properties	**Specification Limits**
Number of Blows	50
Air void content, (%)	18-25
Cantabro loss, (%) Max.	20
Marshall stability, (kN) Min.	3.5
Marshall flow (0.1 mm)	20-40
Draindown ratio, (%) Max.	0.3
Fiber Quantity, %	0.3 – 0.5
Indirect Tensile Strength (ITS) Ratio, (%), min.	75

Table 3.6 Porous Asphalt Gradation Types based on Chinese specifications

Sieve Size		**PAC5**	**PAC10**	**PAC13**	**PAC16**
in, No	mm	Passing	Passing	Passing	Passing
3/4"	19	-	-	100	100
5/8"	16	-	-	100	94
0.53"	13.2	-	100	91	82.8
3/8"	9.5	100	99.1	62.1	53.3
No.4	4.75	90	27.1	21.4	19.9
No.10	2.36	21	13.4	13.3	11.2
No.16	1.18	20	10.1	10.1	8.2
No.30	0.6	15.5	7.5	7.5	6.5
No.50	0.3	11.9	5.8	5.8	5.5
No.100	0.15	9.1	4.8	4.8	4.9
No.200	0.075	7	3.9	3.9	4.3

3.4.4 Malaysian Specification

Mixtures prepared according to the specification should be compacted with 50 blows. The particle loss value and the air void ratio are intended to be a maximum of 15% and between 18% to 25%, respectively. Schellenberger Bitumen Drain down value should be a maximum of 0.3% (Malaysia, 2008). Specification limits and gradations of porous asphalt illustrated in Table 3.7 and Table 3.8, respectively.

Table 3.7 Porous Asphalt Design Criteria Based on Malaysian Specification

Properties	**Specification Limits**
Number of Blows	50
Air Voids, (%)	18-25
Particle Loss (Cantabro), (%), max.	15
Schellenberger Bitumen Drain down, (%), max.	0.3

Table 3.8 Porous Asphalt Gradation Types based on Malaysian specification

Sieve Size		**Grading-A**	**Grading-B**
in, No	mm	Passing	Passing
7/8"	20,0	-	100
5/8"	14	100	85-100
7/16"	10	95-100	55-75
No.3	5	30-50	10-25
No.10	2.36	5-15	5-10
No.200	0.075	2-5	2-4

3.4.5 Indonesian Specification

According to the specification, the mixtures will be compacted by hitting 50 blows. It is necessary to design in such a way that the particle loss value is a maximum of 20 percent and the Marshall stability value is a minimum of 350 kg. To have an effective permeability, the air void ratio of the mixtures is determined between 17 percent and 23 percent. The minimum permeability value will be determined as 0.001 (cm/sec), and the minimum limit will be considered while designing. Schellenberger Bitumen Drain down value should be a maximum of 0.3% (Pradoto et al., 2020). Specification limits and porous asphalt gradation types as shown in table 3.9 and Table 3.10, respectively.

Table 3.9 Porous Asphalt Design Criteria Based on Indonesian Specification

Properties	**Specification Limits**
Number of Blows	50
Air Voids, (%)	17-23
Particle Loss (Cantabro), (%), max.	20
Marshall Stability, (kg), min.	350
Schellenberger Bitumen Drain down, (%), max.	0.3
Permeability, (cm/sn), min.	0.001

Table 3.10 Porous Asphalt Gradation Types Indonesian Specification

Sieve Size		**Gradation-A**	**Gradation-B**
in, No	mm	Passing	Passing
1"	25	-	100
3/4"	19.0	100	95-100
1/2"	12.5	93	64-84
3/8"	9.5	65	-
No.4	4.75	18	10-31
No.10	2.36	8	10-20
No.200	0.075	3	3-7

3.5 Mechanism of Porous Asphalt Pavement

The permeable asphalt pavement, consisting of open-graded coarse aggregate, ensures that the precipitation water on the wear layer immediately enters the road body and infiltrates. This region is called the filter layer. Under this layer, there is an accumulation(reservoir) layer, and the water coming from the pavement surface is collected here. It works as a kind of water tank. The water coming to this layer is either infiltrated into the lower layers or is discharged with the help of a drainage system. The porous asphalt layers are shown in Figure 3.2.

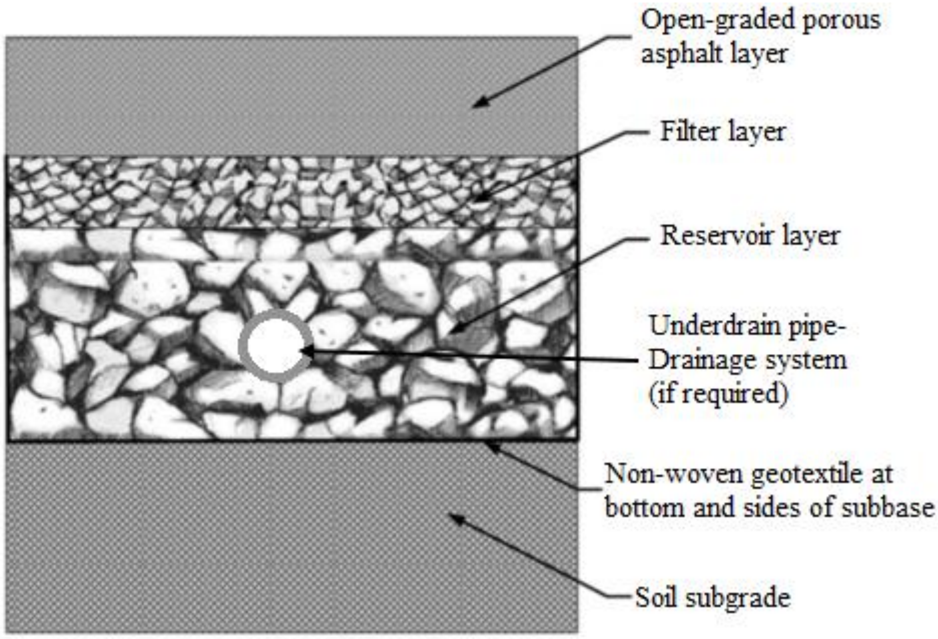


Figure 3.2 Typical cross-section for porous asphalt pavement (Lowa Stormwater Management Manual , 2009).

Pavement Layer: As indicated in Figure 3.2, the pavement in the top layer is the permeable asphalt with open gradation. This layer consists of permeable asphalt pavement with a little part of fine granular substance and a porosity of approximately 20% by volume. With this feature, it differs from conventional asphalt. The thickness of the permeable asphalt thickness ranges between 6.5 and 10 cm depending on many criteria such as environmental factors, traffic density, the number of vehicles with high tonnage transition over the pavement surface, and the amount of tonnage (Cahill et al., 2003).

Filter Layer: Filter layer 2.5 – 5.0 cm. thick, crushed aggregate (12.5mm grain diameter). Massive aggregates are not included in the gradation content. In addition to the in-built filtration (large pore spaces are relatively limited), the filter layer support stability for the deposition layer through the application of the asphalt mixture. This layer filters the solid particles from the pavement surface. It ensures that the water coming from the permeable layer is filtered and transmitted to the accumulation layer. When choosing the design and materials to be used, deposition and permeable layer properties are taken into account (Cahill et al., 2003).

Reservoir Bed Layer: The reservoir layer is generally 4.0 – 7.0 cm. thick, well-graded, and washed aggregate mixture. Its thickness varies depending on the required storage volume, the need to store rainwater, the frost depth, and the ground leveling. In addition to transfer the loads to the ground, the reservoir bed layer collect rainwater until it drains into the ground. This layer also serves as temporary storage and has a drainage system. The drainage system in the accumulation layer transmits the surface waters to a specially designed water tank. Thanks to this reservoir, the waters coming to the pavement surface are directly added to the underground waters (Cahill et al., 2003).

3.6 Advantage and Disadvantage of Porous Asphalt Pavement

3.6.1 Advantages

PAP is preferred to prevent floods that may occur as a result of heavy rainfall in the city, reduce high traffic noise, make the pavement surface safer, and feed groundwater directly.

Road Safety: Increasing road safety depends on developing pavements suitable for all weather conditions. One of the essential parameters in road safety is the friction force that occurs in the contact area between the wheels of the vehicles and the pavement surface. Porous asphalt is a type of mixture developed to prevent rainwater from accumulating on the road surface. It is an open-graded mixture of PAPs with a high coarse aggregate ratio and a low fine aggregate and filler ratio. Since porous asphalt has a permeable structure, water accumulation on the road surface is prevented. Aquaplaning that occurs due to water accumulation does not occur in PAPs. Porous asphalt prevents the formation of puddles on the road surface and prevents the vehicle wheels from splashing water, thus increasing the drivers' visibility in rainy weather (Aman, 2013). Figure 3.3 shows the change in visibility of permeable and conventional pavements due to the spray effect caused by vehicles in rainy weather.

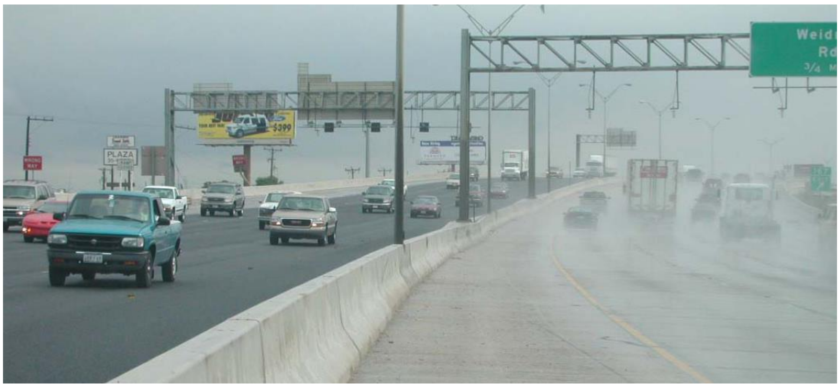


Figure 3.3 Visibility of permeable asphalt and conventional asphalt pavements in rainy weather (Topaloglu, 2019)

The decrease in the friction coefficient caused by the wetting of the road surface, which is observed in traditional wear layers, is to a lesser extent in PAPs. This significantly reduces traffic accidents and provides safe travel for high speed (Ozay, 2011).

Noise Reduction: Noise is a combination of car, bus etc. engines, wind, and vehicle wheels noise. Wheel noise is caused by the pressure of air trapped between the road surface and the wheels. With its porous asphalt structural feature, this pressure on the road surface is reduced and these unwanted noises are reduced on the pavement surface. Studies to improve the noise level, which becomes more and more critical with each passing day, have pioneered the design of double-layer porous asphalt. It is seen that it is possible to reduce the noise level of sections using double-layered porous asphalt by 6dB(A). In the double-layer porous asphalt design, there is an underlying layer of coarse aggregate particle similar to conventional porous asphalt. Above that, porous asphalt wear layer of finer aggregates (Hwee & Guwe, 2004).

Ground Water Feeding: It is possible to remove surface waters caused by precipitation from the pavement surface in an environmentally friendly manner with PAPs. Instead of removing the precipitation water coming from the pavement surface with the sewage system, the porous asphalt is transferred to the base ground thanks to its void structure, and the underground water is fed. In this way, it contributes to the ecological cycle (Cahill et al., 2003).

3.6.2 Disadvantages

3.6.2.1 Aging

The change in the chemical structure of the bitumen due to aging, the removal of the volatile substances in the bitumen from the structure and the oxidation effect of the air cause the bitumen to harden. Low temperatures cause the hardened bitumen to become brittle and the fracture energy to decrease. Bituminous mixtures have low temperature cracking properties on aggregate type, bitumen type, and air void ratio. The decrease in the void ratio observed in PAPs over time reduces the pavement's exposure to atmospheric conditions, which delays aging effects. The addition of soft bitumen and rough aggregates to the asphalt mix increases the asphalt mix's resistance to cracking at low seasonal temperatures. It is stated that there is a relationship between the structural stiffness of bitumen at low air temperatures and the cracks that occur (Al-Qadi & Wang, 2009).

Modified bitumen is more resistant to the effects of aging and modified bitumen is more difficult to reach critical hardening. The modification increases the strength of the bitumen against cracking. The initial performance of the modified bitumen to be used in the mixtures gave better results than the unmodified bitumen (Sengoz et al., 2009).

PAPs are sensitive to the effects of aging due to their high void ratio. The large binder surface area is the main factor in the aging of the bitumen. In PAPs, the oxidation of the binder material occurs faster due to the porous nature of the pavement structure. This adversely affects the durability of PAPs. Therefore, when PAPs are prepared using modified bitumen, mechanical and structural deterioration caused by aging is prevented (Ozay, 2011).

3.6.2.2 Stripping and Ravelling

The use of open gradation aggregates in porous asphalt and the lack of fine aggregate reduce the contact surface area between aggregates and increase the mechanical loading on the binder. Shear forces due to traffic loads increase the risk of breaking off aggregates from the road surface. In PAPs, the breaking off aggregate from the road surface is a more common problem. Because this type of pavement has a low rate of fine aggregate, the material that holds the aggregates together is mostly a bituminous binder. In addition, it is suggested that aggregate losses in the pavement structure may occur due to the decrease in the adhesion of the bituminous binder to the aggregate surface at high temperatures, weakening of the adhesion with the effect of water or embrittlement of the binder due to fatigue at low temperatures (Ozay, 2011).

The raveling and deformations occur due to aggregate loss in the pavement structure. These ravelings usually occur 6 to 8 years after the pavements are made. (Huber, 2000).

These problems can improve performance by modifying the binder material or by increasing the range of material content. However, increasing the amount of binder may cause the porous structure of the pavement layer to be filled with bitumen, thus reducing its permeability.

3.6.2.3 Skid Resistance

The skid resistance of the newly paved PAP surface, the results of the tests and driving experiments showed that it is lower than the conventional asphalt types. The reason for this is thought to be the high bitumen film thickness on the aggregates. (Dell'Acqua et al., 2011).

This decrease in skip resistance is increased again by decreasing the bitumen thickness of aggregates on the pavement surface.

3.6.2.4 Decreasing of Void Ratio

Mechanically induced permanent deformation in PAPs can cause a decrease in the void ratio. It is seen that the main reason for the permanent deformations in the pavement is the lack of sufficient physical strength of the aggregate and the loss of integrity of the aggregates by breaking under traffic loads. For this reason, it is recommended that these pavement designs be made using especially high-strength aggregates (Liu & Cao, 2009).

Another reason for the decrease in the void ratio over time is that the muddy material coming to the road surface enters the pavement and fills the voids. This causes the drainage ability of the pavement to decrease, and the precipitation waters cannot be efficiently removed from the road surface (Ozay, 2011).

As a result of solid particles entering the pores of the pavement structure and the pavement surface loses its permeability properties. A system that has lost its function may require frequent replacement and maintenance. Therefore, it causes an increase in costs. In order to reopen the clogged gaps, the pavement is washed with high-pressure water spray vehicles.

3.6.2.5 The Binder Draindown

The gradation type of the porous asphalt aggregate and the thick bitumen film it has may cause the binder to draindown during the transport of the mixture to the construction site.

The optimum bitumen content significantly impacts the properties of PAPs, such as stability, permeability. Low bitumen ratio results in the inadequate pavement of aggregate surfaces. Due to the thinner bitumen film, rapid oxidation can occur and eventually cause raveling. Conversely, excessive bitumen content leads to the leaching of the bitumen used as the binder (Whiteoak, 1990).

3.7 Maintenance of Porous Asphalt Pavement

The road pavement needs periodic maintenance and repair. The low maintenance and repair costs and the long periods ensure that the overall cost of the pavement is reduced. In case of increased maintenance and repair costs, the pavement can be considered to have completed its service life.

In permeable asphalt layers, the pavement behaves like typical pavements due to the clogged voids over time. The clogged permeable layer can be opened with pressurized water. However, if solid materials cause clogging, it is not possible to open the pores and the only solution is to remove the clogged layer and manufacture a new layer. For this reason, the approximate service life of permeable asphalt is eight years, whereas traditional asphalt layers are 20 years (Ozay, 2011).

Permeable asphalt pavements should be checked frequently for the first few weeks after construction and then at least annually after that. Especially after heavy rains, it is crucial to clean the pavement surface to prevent ponding so that the design can work according to its purpose, taking into account possible clogging situations. In solid clogging cases, the pavement must be replaced as it is impossible to open the pores. The surfaces of these coatings need to be cleaned regularly 4 or 5 times a year with high-pressure water or vacuum cleaner. Thus, the pores of the asphalt can be kept open (Briggs, 2006).

When repairing porous asphalt, the surface should never be sealed. Annual rutting/raveling value checks should be made. When these pavements are deformed in any way, if the damaged area is less than 5 square meters, it can be repaired by patching with a similar mixture design. However, in cases where the deformed area is larger than 5 square meters, all parameters of the area to be repaired should be reviewed and patched. (Cahill et al., 2005).

CHAPTER FOUR
MODIFICATION

Modification is defined as mixing various additives in specific ratios and conditions into the base bitumen to increase the performance of the mixture used in road pavements (Malkoç, 2002).

4.1 Purpose of Modification

The road pavements, due to adverse climate conditions, increased and repetitive traffic loads, and the nature of the raw bitumen, cannot complete the expected service life and maintain their sustainability at all times. Bitumen is modified and improved with various additives to increase the performance of the bitumen used against deformations caused by environmental effects and traffic loads on road pavements. Since bitumen is a viscoelastic and thermoplastic material, its consistency changes when heated. Due to these healing properties, the bituminous hot mixture is widely used as the material that binds aggregates together in road pavements. In other words, these mixtures show a flexible structure. Research has shown that the facility focuses on modification studies with various additives such as polymers, valuable waste, industrial waste and nanomaterials.

The primary purpose of adding to polymers or different additive bitumen is to change the viscoelastic behavior of bitumen and, in particular, to reduce its sensitivity to temperature without harming its behavior at low temperatures (Zhu et al., 2014).

A modification is called mixing various additives to the binder in specific proportions and conditions to improve the mechanical and physical properties of the bituminous binder or mixture used in flexible road pavements and to prevent deformation due to environmental effects.

The modification process aims to obtain flexible pavements with superior quality in terms of stability and performance, longer life, less maintenance and repair costs, and requirements. The uses of modified bitumen and mixtures can be listed as follows;

- Enhance the bond quality between the bituminous materials and aggregate,
- Improve the pavement's durability,
- Improve workability and compaction,
- Increase the resistance against aging or oxidation,
- Contributes to reducing the thickness of layer,
- Enhance the fatigue resistance pavement layer,
- To increase the mechanical properties of the mixture (Hunter et al., 2015).

4.2 Polymer Types and Applications

Various bitumen modification applications are carried out to provide the above-listed requirements. Different modifiers are used to increase the chemical and physical properties of bitumen. Fiber and polymer additives are common types used as bitumen modifiers (Liu & Wu, 2011).

Polymer modification is the most widely used method. In bitumen modification, two groups of polymers are used, namely thermoplastic elastomers such as /butylene-styrene (SEBS) (Sengoz et al., 2009; Thakre et al., 2016).

Complementing the properties of the polymer additive and bitumen to be used is an essential criterion in terms of giving maximum performance when combined. Cohesion can be defined as the tendency of modified bitumen not to degrade during storage. Compatibility with each other increases or decreases depending on the bitumen's chemical structure and the polymer's amount and structure. For example, the production temperature and processes in obtaining bitumen are essential criteria in terms of changing the properties of the bitumen. (Lu & Redelius, 2010).

Bitumen and polymer properties, polymer ratio and modified bitumen production process significantly affect the final product properties. In some polymer modified bitumens (PMB), bitumen is dominant in cases where the additive ratio is increased, and in some cases, polymer can be dominant (Zhu et al., 2014).

Table 4.1 shows the polymer types used in bitumen modification, their common names, and some usage areas.

Table 4.1 Example of different polymer types (Hunter et al., 2015)

Type	Example	Usage purpose
Rubber	Natural latex Synthetic latex Reclaimed rubber Polyethylene	Decrease HMA stiffness at low temperatures to resist thermal cracking, Rutting resistance, Improving cohesion of mixture
Polymer	Elastomer (SBS)	High temperature performance, Surface treatment and isolation, Improving cohesion of mixture, HMA rutting resistance, Crack filling
Plastomers (EVA EBA)		
Reactive polymers	Elvaloy 4170	Increase fatigue resistance, resulting polymer modified asphalt is very stable, and can be stored for extended periods.
Fiber	*Natural:* Asbestos Rock wool *Manufactured:* Polypropylene Polyester Fiberglass	Increase stiffness
Chemical modification	Polyphosphoric acid (PPA)	Increase high temperature stiffness.

4.2.1 Elastomer

Elastomers are large-molecule materials whose shape and dimensions change significantly with the application of a small force and can regain their original shape and dimensions in a short time with the removal of the force. The main elastomers used in bitumen modification are natural rubber (NR), polybutadiene (BR), isobutadien isoprene rubber (IIR), and ethylene-propylene di-monomer (EPDM), styrene-ethylene butylene styrene (SEBS), styrene-butadiene rubber (SBR), and styrene butadiene styrene block copolymer (SBS) (Lu & Isacsson, 2000).

The most common molecule to form a styrene polymer is butadiene. During the polymerization process, if the butadiene and styrene form a random arrangement, this polymer is referred to as Styrene-Butadiene Rubber (SBR). This kind of polymer may also be referred to as synthetic latex. On the other hand, if butadiene and styrene are formed discretely, it would be called Styrene-Butadiene-Styrene (SBS) (Asphalt Institute, 2007).

Although the quality of the elastomer is affected by many factors, the most important of these factors is the amount of shear caused by the mixer. When polymers are added to hot bitumen, the bitumen starts to mix with the polymer particles rapidly. The amount of shear force on the particles is essential to achieving a satisfactory level of dispersion within a realistic mixing period. Therefore, high-performance mixers must be used to properly disperse the elastomers into the bitumen (Almusawi et al., 2021).

4.2.2 Plastomers

When these polymers are combined with bitumen at ambient temperature and added to the prepared asphalt mixture, it is seen that the viscosity of the mixture increases. However, it has been observed that when the plastomers are heated, they do not increase the elasticity of the bitumen as desired, as it causes dispersion by decomposition and cooling. The most used in engineering are polyethylene, polypropylene, and polyvinyl chloride. These thermoplastics are generally resins with good electrical and chemical properties, low friction coefficients, deficient water absorption, and easy to process. Some plastomers used to modify bitumen include polyethylene (PE), polyvinyl chloride (PVC), and ethylene vinyl acetate (EVA). The viscosity of bitumen increases as a result of mixing this and similar polymers with bitumen (Lu & Isacsson, 2000).

EVA copolymer is a thermoplastic modifier with irregular chemical structures, produced by co-polymerization of ethylene and vinyl acetate. The properties of EVA copolymers are often controlled by their molecular weight and proportions of vinyl acetate. EVA disperses homogeneously and rapidly in bitumen and creates a solution compatible with bitumen. During the post-production storage, some precipitations etc. occur. Therefore, the modified bitumen must be carefully mixed before use. Addition of EVA to bituminous mixtures is not only used to enhance pavement performance, but also in cold weather applications, a significant amount of EVA is used. EVA increases the workability of the mixture due to its sensitivity to shear force and its use with softer bitumen (Asphalt Institute, 2007).

4.2.3 Thermosetting

Polymers with high cross-links do not melt and soften like thermoplastics when heated, on the contrary, they have a rigid structure. If the temperature is further increased, they undergo direct thermal decomposition and chemically decompose. For this reason, polymers that harden when heated are called thermoset polymers, which means heat-curing.

In thermoset polymers with high cross-links, since the leading chains are connected to each other with strong bonds, the chains cannot move independently of each other and become fluid. Therefore, thermoset polymers do not soften at high temperatures, they always remain hard, but at a certain high temperature they decompose giving gaseous products. The most critical thermosets are alkyds, phenolics, thermoset polyesters, epoxies, and polyurethanes (Nic et al., 2005).

CHAPTER FIVE
WARM MIX ASPHALT

5.1 Warm Mix Asphalt Technology

The main reason for the use of WMA is to stay in a lower range than the production temperature of conventional asphalt mixtures during the production of the asphalt mixture. In addition, this low production and asphalt paving temperature provide comfortable working in the environment. Considering all these data, producing various WMA additives continues at full speed to reduce the production temperature. WMA technology is a type of asphalt pavement with a potential solution in terms of advantages such as energy savings and less environmental damage. Due to the production technology, WMA additives can serve the viscosity value of bitumen at lower values than HMA. The mixtures produced using this innovative technology reduce the energy consumption, the gases caused by the fuels used in the field, and the workers' health, without reaching the high temperature required to produce HMA. In addition, it ensures that the asphalt paving section can be serviced in a shorter time (d'Angelo et al., 2008).

WMA technology is an environmentally friendly production that allows an average of 20°C to 55°C reductions in production and laying temperature compared to conventional HMA. With this technology, different approaches are seen to reduce the production temperature. Some of these are using organic or chemical additives, water-based or water-containing foaming techniques. Although these production techniques may seem different from each other, they are all developments for the same purpose. Examples include lowering the viscosity of bitumen, increasing workability, and reducing the emission data generated during production (Zaumanis, 2010).

The first modern WMA studies started in the 90s and the research results were presented at the German Bitumen Forum (GBF) held in 1997. It is aimed to introduce this innovative WMA technology to the bitumen producers and academicians in the organization. Companies are leading new developments every day by working in line with the needs of the projects. In the following years, Shell Bitumen company made trial cuts in Norway, England, and the Netherlands. They aimed to check the versatility of the test data obtained from the field with the designs made in the laboratory environment of this technology (Koenders et al., 2000).

Bitumen modifiers used in WMA technology are generally known as Sasobit, Rediset, and Advera. These modifiers are applied by mixing directly with bitumen at lower production temperatures. Modifiers are entirely dissolved in the bituminous binder and the viscosity value of the bitumen is reduced by this method (Kanitpong et al., 2008; Rubio et al., 2012; Xiao, et al., 2012).

Studies have focused on foamy WMA, in particular, to produce material at a low process temperature than the traditional HMA temperature and to create a WMA material that reduces energy consumption, odor, and smoke (Larsen et al., 2009).

Gandhi and Amirkhanian used two types of step for the incorporation of WMA modifiers add the binder and completed the sample preparation process. In the first step, the hand-blended Aspha-minR additive was investigated. In the second stage, the Sasobit® additive was examined with mechanical mixing. As a result of their usability, they presented a comparison (Gandhi & Amirkhanian 2007).

The mixture design prepared using Evotem J1 additive was subjected to the Marshall stability test and it was investigated that the used modifier allowed to reduce of the mixture temperature up to 120°C during production. At the same time, its effect on humidity was examined and it was observed that it decreased according to the mixing temperature. It has been seen that HMA's performance has not changed, on the contrary, it has performed better (Mahida et al., 2015).

It has been observed that recycled road pavement material with natural zeolite additive can be used, however, with the temperature-lowering effect of natural zeolite, it both increases the workability of asphalt and provides high performance by creating a sustainable road pavement (Valdes-Vidal et al., 2018).

When WMA additives are used in asphalt mixtures, they increase the resistance against permanent deformations such as low temperature cracks by decreasing the temperature sensitivity and increasing the softening point (Şengöz and Işıkyakar, 2008).

The HMA taken from the plant cools down during transportation and when they come to the paving area, they cannot obtain the necessary space because they are far below the application temperature. However, the low compression temperatures of WMAs minimize this problem (Croteau & Tessier, 2008).

5.2 Classification of WMA

WMA technologies, whose primary purpose is to reduce viscosity, can be categorized as various additives and techniques. Some classifications are made according to the production temperature. According to these classifications, Figure 5.1 shows mixtures classified about to production line temperatures (D'Angelo et al., 2008).

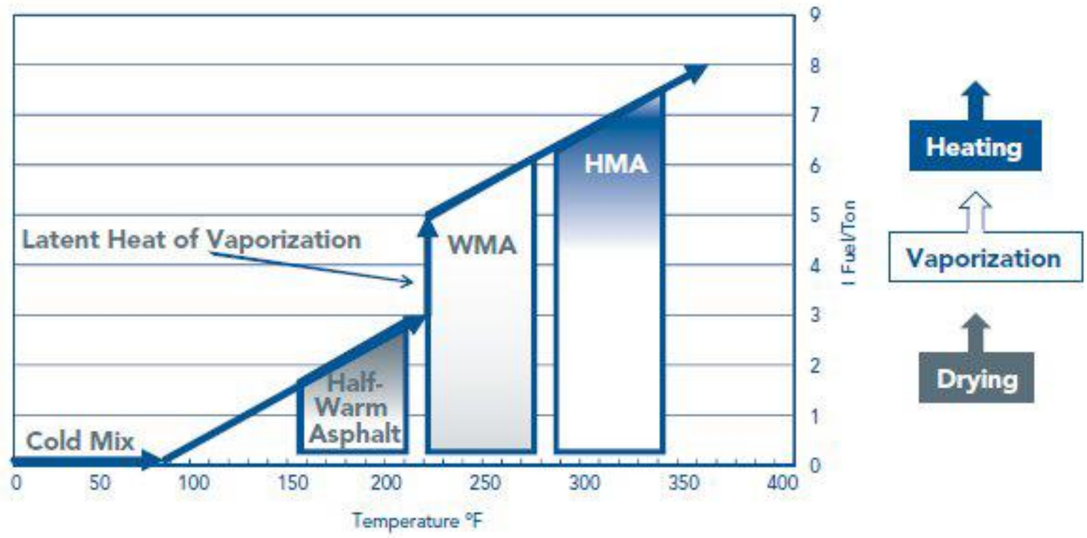


Figure 5.1 Classification of WMA asphalt technology by temperature ranges (D’Angelo et al., 2008)

In WMA techniques prepared at temperatures above 100°C, the amount of water remaining in the mixture is very low as it is lost due to evaporation. Reducing the viscosity of the binder is necessary to ensure that the aggregate is wholly covered with a bituminous binder, albeit at low temperature, and that the mixture can be effectively compressed. This reduction can also be achieved by using various modifier additives. Basically, WMA additives are classified according to different production and application methods and are generally divided into three groups:

- Organic additives
- Chemical additives
- Foaming methods (additives containing water and direct water injection).

5.2.1 Chemical Additives

Various chemicals can be used in combination with different additives in order to increase the bitumen aggregate adhesion rate and workability in the mixtures and to prevent the bitumen from peeling off the aggregate surface. (Zaumanis, 2010). WMA systems involving the use of chemical additives or surfactants appear to have insufficient effect on reducing the viscosity of the binder. Instead of such approaches, it is thought that the binder covers the aggregate surface better at a lower mixing temperature (Croteau & Tessier, 2008).

The purpose of these additive types is to improve bitumen chemically and improve bitumen performance. Surfactants, such as anti-stripping agents, consist of some combinations of chemical additives. Chemical WMA additives are added into the base bitumen, and then this mixture is mixed with the all aggregate particles. With chemical additives, the temperature drop is achieved without adding water (Kaya & Topal, 2016).

Chemical WMA modifiers do not change the viscosity of the binder material. Typically, in the range of 85°C to 140°C, they regulate and reduce frictional forces at the interfaces. Thus, it is possible to mix bitumen and aggregate at low temperatures and to compress the prepared mixture. Chemical WMA additives are generally between 1.5% and 3.0% by weight of the bitumen in the mixture, according to the manufacturer's recommendations and experimental studies. Chemical WMA modifiers can decrease the mixing and compaction temperature degree about 20 - 40°C (EAPA, 2016).

Evotherm, which is best known in this additive class, is a type of additive developed in America. In the process of production additive, an modifier is mixed together hot aggregate and produced at an average temperature of 85-115°C. This emulsion has been produced with chemicals to improve workability, adhesion, and pavement structure. 50% of the content of this chemical additive is obtained from renewable resources. Compared to HMA, Evotherm lowers the laying temperature of asphalt mixes in the production line and from the paver between 50°C and 75°C (d'Angelo et al., 2008).

Rediset® produced by Akzo Nobel is an example of these additives. Rediset® consists of a combination of organic additives and cationic surface additives (Chowdhury and Button, 2008).

Table 5.1 presents a summary of the chemical additives along with their descriptions.

Table 5.1 Summary of some chemical products and technologies available (Zaumanis, 2010)

Product	**Company**	**Description**	**Use Reported In**	**Additive**	**Production Temperature [Or Reduction Ranges]**
Evotherm® ET	Mead-Westvaco	Chemical bitumen emulsion	US, France, worldwide	Delivered in form of bitumen emulsion	85-115°C, [varies]

Table 5.1 Continues

Evotherm ® DAT	Mead-Westvaco	Chemical package plus water	US, France, worldwide	30% by weight of binder	85-115°C, [varies]
Rediset® WMX	Akzo Nobel	Cationic surfactants and organic additive	US, Norway	1.5-2% of bitumen weight	126°C, [varies]
REVIX®	Mathy-Ergon	Surface-active agents, waxes, processing aids, polymers	US	Not specified	[15-26°C]

5.2.2 Organic or Wax Additives

When organic or wax additives are added to bitumen, it reduces the viscosity of the mixture and lowers the temperature of the mixture.

Different organic additives are also used to reduce the viscosity of the bitumen in a temperature range above 90°C. While specify the additive types, it should be taken into account that the melting points are higher than the predicted temperatures and that reduce the brittleness of the asphalt that occurs at low temperatures. Generally, organic additives containing paraffin are added directly to the bitumen, they are entirely melted at melting temperatures and distributed homogeneously in the bitumen. With organic additives, the mixing temperature decreases typically between 20-40°C. In addition, these additives modify the asphalt to increase its resistance to deformation (Rubio et al., 2012).

The most widely used Sasobit additive in the organic additives class in the literature is an organic or wax-based additive produced by Sasol Wax. Sasobit WMA additive is an aliphatic hydrocarbon obtained by coal gasification process, which dissolves homogeneously in the binder at temperatures above 98°C. Consequently this soluble situation can significantly decrease the viscosity of the binder material. As is known, this decrease in viscosity can reduce the workability temperature. This temperature can also decrease between 15-55°C on average (Zhang, 2010).

In general, organic additives by adding 2-4% to the unmodified bitumen, they provide a 20-40 ° C drop in mixing temperature (Rubio et al., 2012).

Table 5.2 presents a summary of organic additives available around the world along with their definitons.

Table 5.2 Summary of some organic products and technologies available (Zaumanis, 2010)

Product	Company	Description	Use Reported In	Additive	Production Temperature [Or Reduction Ranges]
Sasobit®	Sasol	Fischer-Tropsch wax	US, EU, Worldwide	2.5-3.0% of bitumen weight in Germany 1-1.5% of bitumen weight in US	130-150°C, [varies]
Asphaltan® A Romonta® N	Romonta GmbH	Montan Wax for mastic asphalt	Germany	1.5-2.0% of bitumen weight	Varies, [20°C]
Asphaltan® B	Romonta GmbH	Rafined Montan wax with fatty acide amide for rolled asphalt	Germany	2.0-4.0%, 2.5%by mixture weight	Varies, [20-30°C]

5.2.3 Foaming Technologies

The foaming method, another WMA technology, is used by adding a small amount of water to the molten bitumen. The water injected into the molten bitumen at a high temperature evaporates at this temperature, causing a large amount of foam to form while leaving the environment. Large volumes of foams formed in the binder material cause the bitumen to expand and reduce viscosity. As it is known, the decrease in viscosity improves the workability of the mixtures. However, bitumen that cannot adhere to the aggregate surface causes peeling in this method where water is used. In order to eliminate this adverse effect, the sensitivity of the mixture to moisture is reduced to an acceptable level by using anti-peeling additives and the adhesion between the binder material and the aggregate surfaces is ensured.

Different foaming techniques are used to reduce the viscosity of the binder material. The water added to the hot bitumen will evaporate due to the high temperature. Thus, the volume of the bitumen will increase for a short time and its viscosity will decrease. The volumetric growth caused by the evaporation of water will expand the bitumen and cover the aggregates at low temperatures. Thus, the remaining moisture in the mixture facilitates the operations of the asphalt on the paving site (d'Angelo et al., 2008).

Its chemical definition is stated as hydrated alumina silicate of alkali metals. It is used by mixing with bitumen beforehand. The bitumen temperature is not changed for the mixture, but the aggregate temperature is lowered by 30°C and the mixture is prepared. The amount of use is between 0.2% and 0.4% of the bitumen weight (Aktas et al., 2018). More examples of foaming technologies are depicted in Table 5.3.

Table 5.3 Example of existing WMA foaming technologies (D'Angelo et al., 2008)

Technology	Company	Production Temperature (at plant) °C	Reported Field Practice
WAM-Foam	Shell Global Solutions and Kolo Veidekke	110–120 °C	Many European Countries and Canada
Advera® (Synthetic zeolite)	PQ Corporation	20–30 °C (Drop from HMA) 130–170 °C (German guideline Recommendation)	U.S.
Aspha-min® (Synthetic zeolite)	Eurovia and MHI	20–30 C° (Drop from HMA) 130–170 °C (German guideline Recommendation)	Germany
LT Asphalt	Nynas	90 °C	Netherlands and Italy
Double- Barrel Green	Astec	116–135 °C	U.S.
ECOMAC	Screg	Placed at about 45 °C	France
LEA, EBE and EBT	LEACO, Fairco and EIFFAGE Travaux Publics	<100 °C	France, Spain, Italy and U.S.
LEAB®	BAM	90 °C	Netherlands

CHAPTER SIX
EXPERIMENTAL

6.1 Materials

6.1.1 Base Bitumen

Base bitumen supplied from İzmir Aliağa Turkish Petroleum Refinery with a penetration class of 50/70 was used in the preparation of the designs in this study. Conventional bitumen tests were performed on the unmodified bitumen supplied from the refinery, and its properties were checked for compliance with the specification. Table 6.1 shows the properties of the base bitumen according to ASTM specifications.

Table 6.1 Properties of the base bitumen

Test	Specification	Result	Specification Limits
Penetration (25 °C; 0.1 mm)	ASTM D5	65	50-70
Softening point (°C)	ASTM D36	51	46-54
Penetration index (PI)	-	0.35	-
Flash point (°C)	ASTM D92	344	230 (min)
Specific gravity	ASTM D70	1.030	-
Rolling thin film oven test (RTFOT)			
Penetration (25 °C; 0.1 mm)	ASTM D5	53	50 (min)
Retained penetration (%)	ASTM D36	82	50 (min)
Change of mass (%)	-	0.160	0.5 (max)
Softening point after RTFOT (°C)	ASTM D36	58	48 (min)

6.1.2 Polymer Modifiers

Two different polymer modifiers, SBS Kraton D 1101 and Elvaloy 4170 were used in this study to prepare polymer-modified bitumen. In general, bitumen modifiers are frequently preferred due to their improving properties of asphalt mixtures. The solution of problems such as aging, sensitivity to temperature changes, rutting, and fatigue cracks can be given as examples.

6.1.2.1 Elastomeric Polymer

Kraton® D 1101 Styrene-Butadiene-Styrene (SBS) belonging to the elastomer class, was used as a polymer additive in the thesis study (Figure 6.1). The SBS polymer was Kraton D-1101 (powder form) supplied from Shell Chemicals Company. Kraton® D 1101 is a linear SBS polymer composed of 31% styrene and high polybutadiene blocks (Shell Tech. Bulletin, 1995). The physical properties of SBS are given in Table 6.2. Figure 6.1 shows the Kraton® D1101 additive in powder form.

Table 6.2 The properties of SBS polymer

Composition	**Specification**	**Kraton D 1101**
Molecular structure	-	Linear
Physical properties		
Specific gravity	ASTM D792	0.94
Tensile strength at break (MPa)	ASTM D 412	31.8
Shore hardness (A)	ASTM D 2240	71
Physical form	-	Powder, pellet
Melt flow rate	ASTM D-1238	<1
Processing temperature (°C)	-	150–170
Elongation at break (%)	ASTM D 412	875
Density	-	-

Figure 6.1 SBS Kraton® D-1101 (Powder Form) (Personal archive, 2022)

6.1.2.2 Plastomeric Polymer

Elvaloy® 4170 plastomer modification additive was obtained from DuPont. Elvaloy is a reactive plastomeric terpolymer whose chemical content is defined as EGA (ethylene/glycide/acrylate). Elvaloy is added to the base bitumen in solid balls while the asphalt is prepared to be passed into a hot mix. The balls melt at high temperatures and form a homogeneous, stable binder material. Its composition and physical properties are given in Table 6.3 Physical view of Elvaloy® 4170 is presented in Figure 6.2.

Table 6.3 Properties of the base bitumen

Composition	**Specification**	**Elvaloy 4170**
Molecular structure	-	Linear
Physical properties		
Tensile strength (MPa)	ASTM D 412	31.8
Physical form	-	Powder, pellet
Melt flow rate	ASTM D-1238	8
Density	-	0.557

Figure 6.2 Elvaloy® 4170 (Personal archive, 2022)

6.1.3 Organic Warm Mix Additives

Warm mix additives are categorized into organic(Sasobit) and chemical (Rediset) additives. Sasobit® have been used to produce warm mix modified samples in this study. Sasobit is a long-chain aliphatic hydrocarbon wax that can melt at 120°C. It dissolves homogeneously in bitumen under conditions where the heating temperature reaches 140°C (Sasol wax, 2014). These long chains help the wax penetrate the asphalt mix, which reduces the viscosity of the bitumen in WMA production. This additive is produced both in pellets and flakes. Figure 6.3 shows the form of the Sasobit additive used in the study.

Figure 6.3 Sasobit (Personal Archive 2022)

6.1.4 Aggregates

Two aggregate sets were used to prepare the PAM. The first set (1# Set) has consisted of only limestone aggregate, while the second set (2# Set) was a mix of basalt (the coarse aggregates) and limestone (the fine particle).

6.1.4.1 Limestone

The asphalt mixtures were prepared with limestone aggregate obtained from the Aliağa/İzmir quarry of Dere Beton. The physical and mechanical properties of the aggregate are given in Table 6.4. The aggregate gradation has been chosen as a Type 2 based on the Turkish Specifications for porous asphalt. Figure 6.4 shows the limestone aggregate weighing used in the experimental study.

Table 6.4 Gradation and properties of the limestone aggregate

Test	**Specification**	**Selected Gradation**	**Specification Limit**
Sieve analysis	ASTM C 136		
Sieve no			
¾ in		100	100
½ in		90	90-100
3/8 in		63	63-77
No.4		11	11-35
No.10		10	10-20
No.80		5	5-10
No.200		3	3-7
Bulk Specific Gravity (Coarse Agg.)		2.649	
Bulk Specific Gravity (Fine Agg.)		2.603	
Specific Gravity (Filler)		2.732	
Los Angeles Abrasion (%)	TS EN 1097-2	22.3	25 (max)
Flakiness Index	TS EN 933-3	6.6	20 (max)
$MgSO_4$ Loss (%)	TS EN 1367-2	9.2	10 (max)

Figure 6.4 Aggregate weighing samples with limestone (Personal archive, 2022)

6.1.4.2 Basalt

Basalt aggregate used in the thesis was obtained from the Dere Beton Pınarbaşı-Bornova/İzmir quarry. The physical and mechanical properties of the aggregate are shown in Table 6.5. The aggregate gradation has been chosen as a Type 2 (combination with limestone's fine particles) based on the Turkish Specifications for porous asphalt. Figure 6.5 shows the basalt and limestone aggregate combination weighing to be used in the experimental study.

Table 6.5 Gradation and properties of the basalt aggregates

Test	**Specification**	**Selected Gradation**	**Specification Limit**
Sieve analysis	ASTM C 136		
Sieve no			
¾ in		100	100
½ in		95	90-100
3/8 in		70	63-77
No.4		25	11-35
No.10		15	10-20
No.80		7	5-10
No.200		4	3-7
Bulk Specific Gravity (Coarse Agg.)		2.538	
Bulk Specific Gravity (Fine Agg.)		2.603	
Specific Gravity (Filler)		2.732	
Los Angeles Abrasion (%)	TS EN 1097-2	13.7	25 (max)
Flakiness Index	TS EN 933-3	4.8	20 (max)
$MgSO_4$ Loss (%)	TS EN 1367-2	7.7	10 (max)

Figure 6.5 Aggregate weighing samples with basalt-limestone aggregate combination (Personal archive, 2022)

6.2 Preparation of Bitumen Samples Involving Additives

6.2.1 Preparation of PMB Samples

SBS Kraton D1101 product was used as a polymer-modified bitumen additive. It is a product that allows us to obtain PMB by mixing it with bitumen under suitable conditions to improve the properties of highway pavements. Based on past research, the optimum concentration of SBS to be added to the base (unmodified) bitumen to be modified is 5% (Sengoz & Isikyakar, 2008). To prepare SBS polymer modified bitumen, a high shear laboratory mixer is used as previously mentioned. The mixer is shown in figure 6.6.

Figure 6.6 High shear laboratory mixer, heater and thermometer (Personal archive, 2022)

Base bitumen is heated to 170-180°C, the temperature at which it melts, until it becomes completely liquid. According to some studies, it is aimed that the base bitumen and polymers to be used are above their softening points by heating them at these temperatures. Also, for the polymer particles to completely dissolve in the bitumen during production, the base bitumen must be heated at high temperatures that cannot cause aging (Larsen et al., 2009; Kok et al., 2013; Munera & Ossa, 2014; Schaur et al., 2017). The molten bitumen placed in a 500 ml beaker is placed in the heater. Then, the mixer head is immersed in the beaker, and the polymer-modified bitumen is produced. At this stage, while the SBS additive is gradually added to the bitumen, the rotation speed of the mixer is adjusted to be 2000 rpm. After the entire additive is added, production continues for 1 hour at a constant temperature(175±5°C) and mixing speed. At the end of 1 hour, the production process is completed.

The concentration of Elvaloy to be used in the preparation of asphalt mixtures is recommended as 1.5% of the bitumen weight by the supplier company and under the leadership of previous studies (Topal, 2010). After the base bitumen is heated in an oven at 185 °C until it becomes fluid and transferred to the glass beaker, polymer-modified Elvaloy particles are gradually added to the bitumen, as shown in Figure 6.7. The production process was completed by mixing the pure bitumen and the modifier using a high shear laboratory grade mixer at 200 rpm for 2 hours. During this process, care was taken to keep the production temperature constant at 180±5°C. The viscosity value of the produced polymer-modified bitumen is measured immediately after the production is finished and it must be cured in an oven at 190°C for 24 hours for the additive to complete the chemical reactions (Bulatović et al., 2014).

Figure 6.7 Addition of bitumen modifiers to base bitumen (Personal archive, 2022)

6.2.2 Preparation of Modified Bitumen with WMA Additive

The ratio and production time of organic and chemical WMA additives are different from each other. In the experiments, bringing the bitumen to the required temperatures for mixing with WMA additives was provided by using a heater, and temperature control was made with a thermometer during the production (Figure 6.8).

Figure 6.8 Production of WMA added bitumen using a heater (Personal archive, 2022)

After the base bitumen is heated until it becomes fluid and transferred to the 500 ml glass beaker, Sasobit WMA additives are gradually added to the bitumen. During the production process, the rotation speed of the mixer is fixed at 1000 rpm and homogeneously mixed at a constant temperature (120±5 °C) for 10 minutes. In this study, sasobit will be added as 3% of the bitumen weight. It is thought that keeping the required production time to a minimum can prevent the problems that may arise from the aging of the bitumen (Kanitpong et al., 2008; Sengoz et al., 2013). The manufacturer recommends adding 3% of the mixture weight to the bitumen to achieve the desired decrease in viscosity and not adding more than 4% due to the possible effect of low-temperature properties (Kanitpong et al., 2008). The summary of production conditions of PMB and WMA samples is given in Table 6.6.

Table 6.6 Production process the additives and modifications

Modifier Type		Percentage (%)	Production Conditions		
			Mixing Temperature (ºC)	Mixing Duration(min)	Shearing Rate (rpm)
Polymer	SBS Kraton® D1101	5	180±5	120	2000
	Elvaloy® 4170	1.5	190±5	120	200
WMA	Sasobit	3	120±5	10	1000

6.3 Conventional Properties of Bitumen Involving Additives

Conventional bitumen tests were applied to bitumen produced with both polymers modified and WMA additives. Table 6.7 represent the properties of bitumen obtained by using WMA and PMB additives.

Table 6.7 Properties of the 50/70 penetration bitumen grade involving different additives

Test	Polymers		WMA additive
	%5 SBS	%1.5 Elvaloy	%3 Sasobit
Penetration (25 ºC; 0.1 mm)	48	49	37
Softening point (ºC)	49	65	69
Flash Point (° (°C)	260+	260+	260+
Ductility (25°C, cm)	100+	100+	100+
Rolling thin film oven test (RTFOT)			
Change of mass (%)	0.07	0.03	0.07
Retained penetration (%)	21	25	13
Softening Point Difference(°C)	3	0	4
Penetration index (PI)	-2.18	1	1.95
Storage Stability (°C)	3	1.92	1.6

6.4 Method of Determination of the Optimum Mixing-Compaction Temperature

The Equiviscous method (ASTM D 2493) is used while the mixing compression temperature is found for dense gradations. However, a particular method has not been propounded for porous asphalt. In the porous asphalt specification used in Turkey, the compaction temperature is determined as 145±5 °C since modified bitumen is used. Since the mixing and compaction temperature is an essential criterion in terms of pavement performance, it has been tried to suggest the ideal mixing and compaction temperature for porous asphalt designs, in addition to the specification, by making trials.

6.4.1 Equiviscous Method (ASTM D 2493)

A detailed explanation of this test method is described in the ASTM D2493 standard method. The basic principle of the experiment is to determine the viscosity of bitumen samples at two different temperatures. This measurement should be made using a Brookfield viscometer device. The obtained viscosity values are plotted on a graph together with the temperature values, and the mixing and compression temperature is found (ASTM, 2016).

6.4.2 Proposed Method for Porous Asphalt Pavement

The porous asphalt Turkish specification determines the compaction temperature as 145±5 °C when prepared with PMB. However, determining an optimum mixing compaction temperature is important regarding cantabro loss of PAP. It is thought that the cantabro loss will give variable values according to the type of bitumen to be used, the bitumen modifying additive, and the type of aggregate. Therefore, a study was carried out within the compression temperature limits allowed by the specification. Porous asphalt samples were prepared using base bitumen and SBS, Elvaloy, Sasobit modified bitumen, and two different aggregate groups of limestone and basalt-limestone combination mixture. Mixtures prepared with polymer-modified bitumen are compaction at 140°C, 145°C, and 150°C, and mixtures prepared with WMAmodified at three different temperatures, 120°C, 125°C, and 130°C, and the particle loss values of the mixtures are examined. The optimum mixing and compaction temperature was tried to be found.

6.5 Marshall Mix Design

Test specimens were prepared through a range of bitumen contents varying from 3% to 5%. For each bitumen content, three samples were prepared so it would provide adequate data. The aggregate weight in each sample is 1150 grams. Aggregate and bitumen are taken in a preheated form and poured into a tared mixing container on the precision weight measuring device. The aggregates are mixed with the bitumen removed from the oven in the specified proportions in the mixer at a certain speed for 2 minutes. Heat loss of the sample should be prevented by using a heater during mixing. The samples are then mixed and placed in the preheated mold, as shown in Figure 6.9.

Figure 6.9 Mixing and placing materials (personal archive, 2022)

The molds are lubricated to prevent the materials in the mixture from sticking to the compression molds. Oiled papers aim to prevent material loss from the mixture by placing it on the lower surface of the molds and the parts that will contact the Marshall mallet. Mixtures are placed in cylindrical molds without loss of material. Prepared samples are placed in the Marshall compactor test device. The mixture placed in compression molds is aimed to simulate heavy traffic loading conditions by hitting 50 blows number, as shown in Figure 6.10.

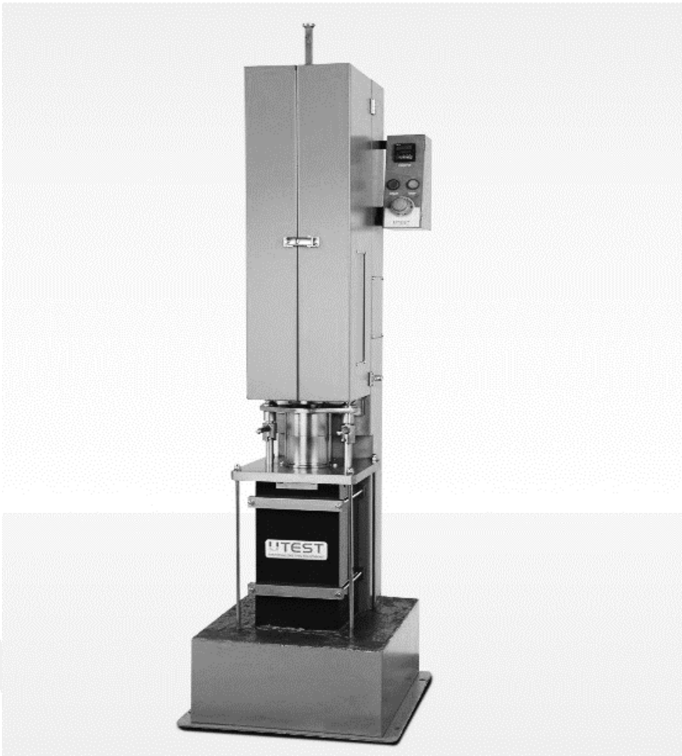


Figure 6.10 Marshall compactor (Personal archive, 2022)

After the compression process, the samples, which are ensured to cool at room temperature, are removed from the molds with the help of a jack. The average height and radius, bulk specific gravity, air void percentages, void ratio between mineral aggregates(VMA), and bitumen filled void ratios (VFA) of the cylindrical samples produced for each bitumen raito were calculated.

While calculating the bulk specific gravity values of PAMs, the weighing method used in conventional mixtures in air, water, and saturated surface air cannot be used. This is because PAMs have a high void ratio, so weighing cannot be performed in saturated surface air. The bulk specific gravity calculation is made by determining the specific gravity from the sample size mentioned in TS EN 12697-6 Procedure-D.

After calculations the prepared asphalt mixture are kept in a 60 ° C preheated water bath for 2 hours in order to bring them to the desired measurement temperature. After the curing process is completed, the mixtures are taken out of the water bath, and the excess water is removed with the help of a towel and placed in your Marshall test device without wasting time. The stability of compacted samples was measured as Marshall stability tester shown in Figure 6.11.

Figure 6.11 Marshall machine (Personal archive, 2022)

Finally, the values of stability, flow, VMA, VFA, air void ratio, and bulk specific gravity has been determined for each bitumen content.

6.6 Particle (Cantabro) Loss Test

Disintegration (particle loss) and segregation can be seen in PAPs under repeated loads. With this experiment, the amount of particle loss and segregation in the samples can be determined. The amount of particle loss determined according to the Turkish Highways Technical Specification should not be more than 20%. This experiment is done by removing the metal balls from the Los Angeles Abrasion Device (Figure 6.12).

Figure 6.12 Los Angeles Abrasion Device

At least 5 cylindrical samples with a diameter of 100±3 mm and a height of 63.5±5 mm are taken. Marshall hammer or gyratory compactor can be used to prepare the samples. The practical specific gravity (Dp) and void ratios of the compressed samples are checked.

Figure 6.13 Sample prepared for particle (Cantabro) loss test

The test room temperatures at which the test takes place has a significant effect on the test results. It is recommended to use a heater in the test room when necessary so that the room temperature is between 15°C-25°C during the test. In addition, in order for the prepared samples to be tested, they must be kept in a place where the ambient temperature does not exceed 25°C for 2 days after production.

The weight of each sample before the experiment is recorded as W_1. A single cylindrical sample is placed inside the particle loss test device(Los Angeles Abrasion Device) without balls. The Los Angeles Abrasion Device is made 300 revolutions at 3.1 rad/sec to 3.5 rad/sec (30 rpm - 33 rpm). When the cycle is complete, the sample is removed from the device, and the loose material is cleaned and weighed as W_2. The experiment is repeated in the same way for each sample (TS EN 12697-17, 2017).

Figure 6.14 Sample after particle (Cantabro) loss test

The particle loss was calculated using the following equation:

$$\mathrm{PL} = \frac{W_1 - W_2}{W_1} * 100 \tag{6.1}$$

Where;

PL = Particle loss (%)

W_1 = Weight of the sample before test, g.

W_2 = Weight of the sample after test, g.

6.7 Method of Determination the Optimum Bitumen Content

For dense-graded mixtures, the optimum bitumen content (OBC) is determined as the bitumen content corresponding to the median of designed limits of percent air voids in the total mix (i.e., 4 %). While for porous asphalt design, the OBC determination has no specific standard, and each agency or department specifies the OBC by considering different parameters such as the air void content, Cantabro loss, stability, permeability, bitumen draindown, and indirect tensile strength.

6.7.1 Turkish (KGM) Specification

Based on Turkish Specification, the OBC is defined by considering the air void content, Cantabro abrasion value, and stability value. These values must be within the specified limits that air voids range is between 18% to 22%, the maximum accepted particle loss is 20%, and the minimum stability value is 300 kg as illustrated in Table 6.8. Furthermore, in order to verify whether the obtained bitumen content is applicable, other tests are required by the standard. These tests include vertical and horizontal permeability, which should be within 0.5 to 3.5 (m/sn), x10^{-3}, maximum bitumen draindown is 0.3 %, and finally, the minimum ITSR is 80%.

Table 6.8 Porous Asphalt Design Criteria Based on The Turkish Standard

Properties		**Specification Limits**
Number of Blows		50
Air Voids, (%)		18-22
Particle Loss (Cantabro), (%), max.		20
Stability, (kg), min.		300
Permeability Value, (m/sn), x10^{-3}	Vertical/Horizontal	0.5 - 3.5
Fiber Quantity, (%)		0.3 - 1.0
Schellenberger Bitumen Drain down, (%), max.		0.3
Indirect Tensile Strength (ITS) Ratio, (%), min.		80

As the final investigation, the permeability value, bitumen drain down, and the ITSR of the PAP mixture samples should be verified based on the obtained OBC.

6.7.2 ASTM D7064

In this method, the OBC is defined based on the air void ratio (min. 18%), bitumen draindown (max. 0,3), and the un-aged abrasion value (max. 20%) standardized limits as shown in Table 6.9.

Table 6.9 Porous Asphalt Design Criteria Based on ASTM D7064

Properties	**Specification Limits**
Air Voids, (%), min.	18
Particle Loss (Cantabro), (%), max.	20
Bitumen Drain down, (%), max.	0.3

6.7.3 Chinese Specification

The determination of OBC in Chinese specification differs from other mentioned methods. In this method, a minimum and a maximum optimum bitumen contents are determined instead of calculating a single value. The minimum OBC is determined as the content matching to the inflection point of the particle loss curve. The maximum OBC is determined as the bitumen content at the inflection point of the bitumen draindown, as depicted in Figure 6.15.

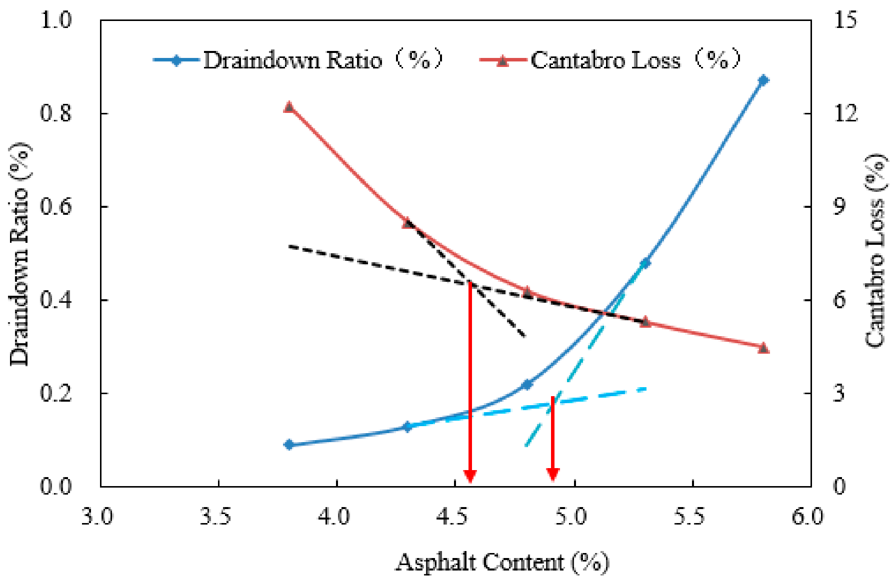


Figure 6.15 Minimum and maximum bitumen content determination based on draindown ratio and Cantabro loss (Ma et al., 2020)

6.7.4 Malaysian Specification

Regarding Malaysian Specification, the OBC is determined utilizing bitumen draindown and Cantabro tests. The rate of mass loss in the Cantabro test must be less than 15% and the bitumen drain-down shall be not more than 0.3%. Also, the obtained OBC must provide between 18 to 25% air voids. The standardized limits are depicted in Table 6.10.

Table 6.10 Porous Asphalt Design Criteria Based on Malaysian Specification

Properties	**Specification Limits**
Air Voids, (%)	18-25
Particle Loss (Cantabro), (%), max.	15
Bitumen Draindown, (%), max.	0.3

6.7.5 Indonesian Specification

According to the Indonesian specification, the OBC for porous asphalt is determined by performing several tests such as the bitumen draindown test, particle loss test, stability test, as well as the average air voids in the mixtures. To select the appropiate values for the design, the mentioned test results must be within the limit values of the specification as; the air void is between 17% - 23%, maximum particle loss is 20%, minimum stability value is 350 kg, maximum bitumen draindown is 0.3%. (Table 6.11).

Table 6.11 Porous Asphalt Design Criteria Based on Indonesian Specification

Properties	**Specification Limits**
Air Voids, (%)	17-23
Particle Loss (Cantabro), (%), max.	20
Stability, (kg), min.	350
Bitumen Drain down, (%), max.	0.3

6.8 Indirect Tensile Strength Ratio Test

It is necessary to know the stress unit deformations to determine the performance properties of the compressed samples according to the design method under traffic loads. The relationship between the stress under the traffic load and the deformation after the applied load can be explained as the modulus of elasticity. The most important of experiments used to calculate the modulus of elasticity; direct tensile, beam bending, indirect tensile and triaxle compression tests. An ITS test is applied to measure the modulus of elasticity of asphalt concrete. The ITS, which must be known to find the modulus of elasticity of the samples, is measured with the ITS device (Figure 6.16).

Figure 6.16 ITS test device (personal archive, 2022)

Marshall samples are prepared in order to examine the resistance degree of bituminous mixtures against the harmful effects due to water, and the samples are divided into two groups as conditional and unconditional. This method, which is explained in TS EN 12697-23, aims to measure the tensile strength change that occurs in the diametrical plane of water-induced bituminous mixtures in the laboratory. With this method, bituminous mixtures produced in the plants and cores taken from the pavement of any age can be tested (TS EN 12697-23, 2017).

Samples in the unconditional group should be stored in sealed plastic bags until the test is carried out in accordance with the TS EN 12697-23 standard. Afterwards, it is kept in a water bath at the test temperature specified in the standard for 2 hours (Figure 6.17).

Figure 6.17 Keep in water bath for dry sets (personal archive, 2022)

All unconditional samples are then placed under the pressure plate in order. The load is applied across the sample diameter (Figure 6.18).

Figure 6.18 Loading the sample across the diameter (personal archive, 2022)

The loading strips depend on the sample diameter. For samples with a diameter of 4 inches, loading strips with a width of 13 mm were used. Applied load to the sample connected to the test machine such that the movement of the head of the test device is 2 inches or 50.8 mm per minute. As soon as a maximum load value is reached, vertical crack formation across the diameter is observed in the sample and the sample cannot carry anymore load. The load value read from the instrument's display is recorded and the ITS values are calculated.

The some steps are applied for the samples in the group to be conditioned for ITS Ratio (ITSR) with respect to TS EN 12697-12 (TS EN 12697-12, 2018);

- Apply a vacuum to obtain an absolute (residual) pressure of (6.7±0.3) kPa, within (10±1) min. Decrease the pressure slowly to avoid expansion damage of the specimens (Figure 6.19).
- Bring the specimens out from the water. Measure the dimensions of the wet specimens with respect to TS EN 12697-29 (Figure 6.20).

Figure 6.19 Apply a vacuum (personal archive, 2022)

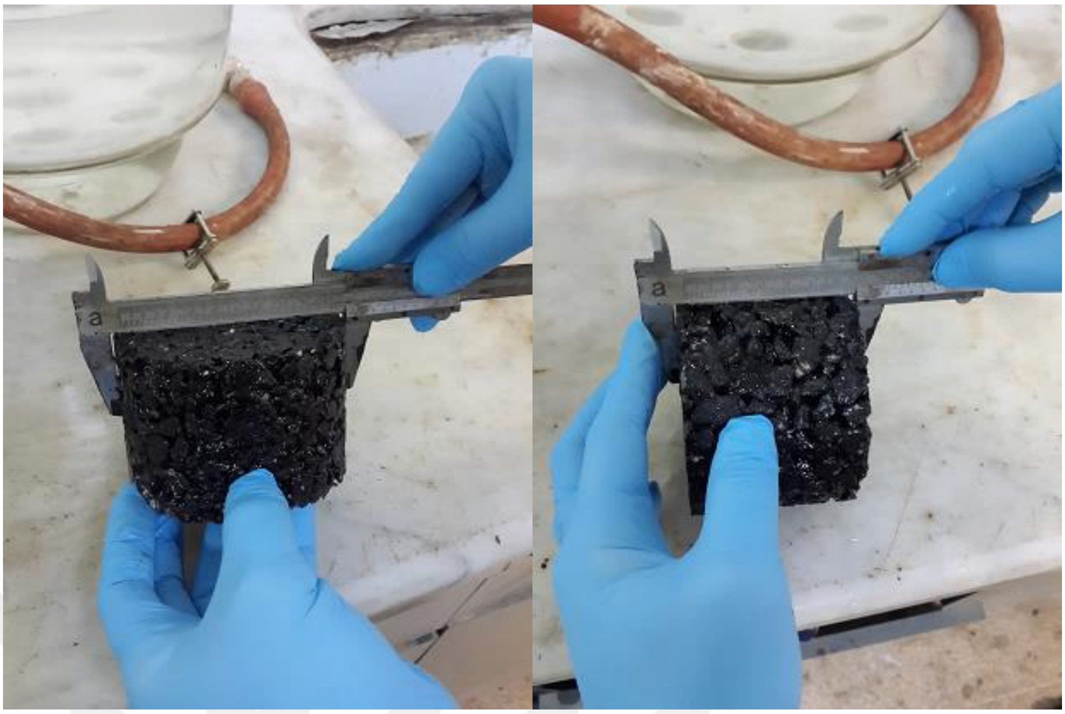


Figure 6.20 Measure the dimensions of the specimens (personal archive, 2022)

The some steps are test procedure of the conditioned samples in accordance TS EN 12697-12;

- After conditioning of the wet specimens, bring the specimens out from the water and left water and let them rest for 30 min on a flat surface (Figure 6.21).
- Store the specimen for at least 4 h in the water bath or air chamber (Figure 6.22). When using air chamber conditioning to test temperature, the temperature shall be controlled by placing the test specimens along with a dummy specimen having a built-in temperature indicator. During the last two hours, the temperature measured in the vicinity of the two set of samples shall not differ by more than 1 °C.
- Calculate the ITS of both the subsets of test samples in accordance with the procedure in TS EN 12697-23 (TS EN 12697-23, 2017).

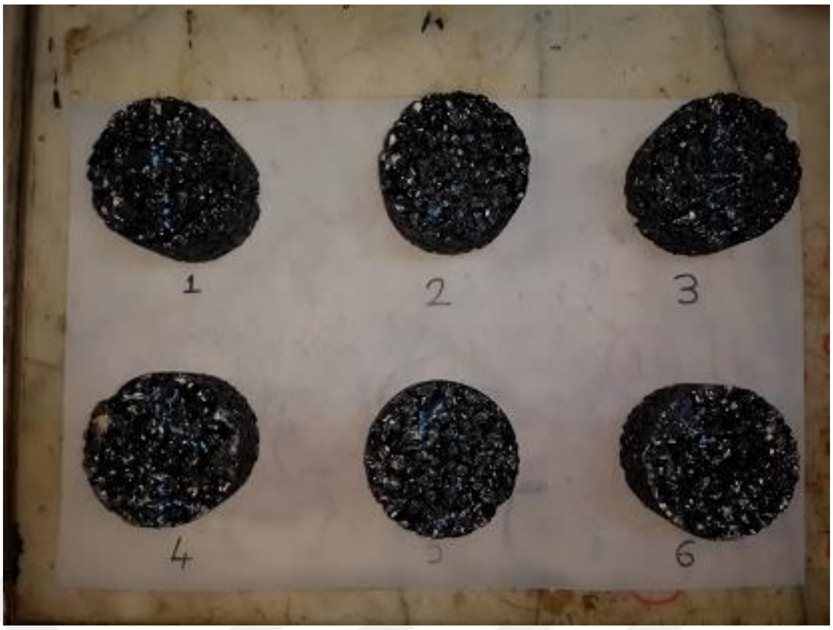


Figure 6.21 Keeping the samples on a flat surface (personal archive, 2022)

Figure 6.22 Water bath 4 hours at test temperature (personal archive, 2022)

For each test sample calculate ITS, according to the formula 6.2;

$$ITS = \frac{2P}{\pi DH} * 1000 \qquad (6.2)$$

where;

ITS = The indirect tensile strength, (kPa)

P = The peak load, (N),

D = The diameter of the specimen, (mm),

H = The height of the specimen, (mm).

The ITSR of the water conditioned set compared to that of the dry set and is calculated and stated in percent. The ITSR, is calculated according to the Equation 6.3;

$$ITSR = 100 * \frac{ITS_w}{ITS_d} \tag{6.3}$$

where;

ITSR = Indirect tensile strength ratio, (%)

ITSw = Average indirect tensile strength of the wet set, (kPa)

ITSd = Average indirect tensile strength of the dry set, (kPa).

6.9 Permeability Test

Permeability is a crucial characteristic of porous asphalt. This is as well the essential distinction between conventional and PAP mixture. The most distinctive feature of permeable asphalt mixtures is their high void ratio. Due to this high void ratio, it quickly absorbs the rainwater coming on the PAMs and removes it from the road surface with its longitudinal and transverse slope. The fact that PAMs have a high void ratio does not always mean that the rainwater on them will be removed from PAP surface very quickly. Besides the high void ratio, these voids must be in contact with each other. It is tested in two different systems, vertical and horizontal, to determine the water permeability rate of PAP mixtures.

Permeability test applied at room temperature. For the permeability test, the size of cylindrical samples has 100 mm or 150 mm diameter and a height greater than 25% of the diameter and at a minimum two times the maximum grain diameter are prepared. In both vertical and horizontal permeability tests, a water column of constant height is formed on the cylindrical sample. The test water is permitted to pass through the sample structure for the specified time interval. Then, vertical and horizontal flow rates (Qv and Qh) and vertical and horizontal permeability values (Kv and Kh) of the sample are calculated (TS EN 12697-19, 2020).

The results that flow rate of the water Qv (vertical flow) or Qh (horizontal flow) is a calculated measure of the permeability value vertical permeability (Kv) or horizontal permeability (Kh). The accepted permeability value for the porous asphalt is between 0.5 – 3.5 (m/s).

6.9.1 Vertical Permeability Test

In this test, the vertical flow of water in the sample is measured. For this purpose, the mechanism in Figure 6.23 is used. The lateral surface of the sample is coated with silicone to provide impermeability just like a membrane (Figure 6.24). The purpose of this process is to blockade of water leakage from the side surfaces of the specimen. It is ensured that the water moves in the vertical direction only in the test specimen. The sample is then fixed in a plastic test tube. The water is permitted to pass through the sample for 10 minutes and the sample is considered saturated (TS EN 12697-19, 2020).

Figure 6.23 Permeability test arrangement (personal archive, 2022)

Figure 6.24 Covering the lateral surface with silicone (personal archive, 2022)

The stages of vertical permeability test sample preparation are given in Figure 6.25.

Figure 6.25 Fixing the sample to the water column for vertical permeability (personal archive, 2022)

After the sample becomes saturated, the water column on the sample is kept constant and water flow is ensured through the sample during the determined time t. The time t must be at least 60 sec. The water passing through the sample during the time t is collected in the chamber, the empty weight of which was previously determined as M_1. At the end of time t, the weight of the reservoir filled with water is measured in M_2 (TS EN 12697-19, 2020).

The vertical flow of water is calculated as equation 6.4.

$$Q_v = \frac{(m_2 - m_1)}{t} \times 10^{-6} \quad (6.4)$$

where;

Qv = Vertical flow of water, m3/s.

m1 = Empty chamber weight, g.

m2 = Weight of the reservoir filled with test water at the end of the period, g.

t = Collection time of water in the reservoir, sec.

The vertical permeabilty of the sample can be measured with the following Equation 6.5:

$$K_v = \frac{4 * Q_v * l}{h * \pi\ D^2} \quad (6.5)$$

where,

Kv = The vertical permeability of test sample (m/s)

Qv = The vertical flow through the test sample (m3/s)

l = The thickness of the test sample (m)

h = The actual height of water column (m)

D = The diameter of the test sample (m).

6.9.2 Horizontal Permeability Test

In this test, the water flow in the sample is measured vertically and horizontally. The test result is a combination of the vertical and horizontal permeability of the sample.

To prevent the vertical movement of water through the sample, the bottom of the sample is covered with silicon (Figure 6.26). The upper part of the sample is fixed to the water column with the help of silicon to provide impermeability, and a leak-proof joint is formed between the sample and the sample. The stages of horizontal permeability test sample preparation are given in Figure 6.27. Thanks to the fiberglass tube, the height of the water column is (300±1) mm. The test water is allowed to pass through the test specimen for 10 minutes and the sample is considered saturated (TS EN 12697-19, 2020).

Figure 6.26 Covering the lower surface of the sample with silicone (personal archive, 2022)

Figure 6.27 Fixing the sample to the water column for horizontal permeability (personal archive, 2022)

After the sample becomes saturated, the water column on the sample is kept constant and water flow is ensured through the sample during the determined time t. The time t must be at least 60 sec. The water passing through the sample during the time t is collected in the chamber, the empty weight of which was previously determined as M_1. At the end of time t, the filled with water reservoir weight is measured in M_2 (TS EN 12697-19, 2020).

The vertical flow of water is calculated as equation 6.6.

$$Q_h = \frac{(m_2 - m_1)}{t} \times 10^{-6} \tag{6.6}$$

where;

Qh Horizontal flow of water, m3/s

m1= Empty chamber weight, g

m2= Weight of the reservoir filled with water at the end of the period, g

t, Collection time of water in the reservoir, sec.

The vertical permeabilty of the sample can be measured with the following Equaiton 6.7:

$$K_h = \frac{Q_h * l}{(H + P + 0.5\, l) * (\pi * D * l)} \tag{6.7}$$

where,

K_h: The horizontal permeability (m3/s)

Q_h: The horizontal flow rate through the test sample (m3/s)

l: The thickness of the test sample (m)

P: The height of the lower tube that is fixed to the test sample (m)

D: The diameter of the test sample (m).

6.10 Bitumen Draindown Test

PAMs contain more bitumen than conventional HMA pavements (Radenberg et al., 2016). It has a gradation structure where the ratio of the fine grains is low. There is more aggregate in certain sieve intervals and less aggregate in certain sieve intervals. While preparing the mixture, due to the temperature, this excess bitumen content in the mixture acquires a flowing feature and is separated from the aggregate mixture by draindown. Schellenberger bitumen draindown test; is a test method developed by the Germans. It is widely used as it is an internationally valid test method. This test is among the design criteria for PAMs.

In this test method, the empty weight of the 1000 ml glass beaker and a 1000 g PAP sample prepared at 135°C is put into the 1000 ml glass beaker and weighed with an accuracy of 0.1 g. After the beaker cap is covered, it is kept in an laboratory oven at 170°C during 1 hour. At the end of this period, it is removed from the oven and the mixture is poured without shaking the beaker. If there is any aggregate adhering to the glass beaker, it must be cleaned. Then the glass beaker is weighed with an accuracy of 0.1 g and the weight of the empty beaker is subtracted. The amount of filtered bitumen is calculated by proportioning the amount of mixture initially tested (TS EN 12697-18, 2018). The maximum value according to the specification is 0.3%. Bitumen draindown is calculated as in equation 6.8.

$$BD = 100 \times \frac{W_5 - W_3 - W_6}{W_4 - W_3} \tag{6.8}$$

Where

BD= The drained material from test mixture, (%)

W_3= The mass of the empty beaker, (g)

W_4= The mass of the beaker plus batch, (g)

W_5= The mass of the beaker plus retained material after upturning, (g)

W_6= The mass of the dried residue retained on the sieve, (g).

Details of some experimental steps are shown in Figures 6.28 and 6.29.


Figure 6.28 Keeping the mixture in an oven at 170 °C for 1 hour (personal archive, 2022)


Figure 6.29 Glass beaker after bitumen draindown test (personal archive, 2022)

CHAPTER SEVEN
RESULTS AND DISCUSSION

7.1 Preliminary Marshall Design with Base Bitumen

The air void ratio is a more critical criterion for PAP than traditional asphalt pavements. Therefore, the mixture gradation that will give the optimum void ratio is an important criterion when designing the mixture. The prepared mixture must remain within the specification limits in terms of stability and permeability and must also be resistant to traffic loads while fulfilling its purpose as a permeable layer in field conditions.

This section's main purpose is to determine how much the changed gradation intervals affect the air void ratios and stability by conducting preliminary experiments. In the following sections, the acceptability of the designs with different performance tests will be discussed.

In line with this information, within the Type-2 gradation limits specified for PAPs in the Turkish Highways Technical Specification, four different gradation ranges have been determined for mixtures to be prepared with only limestone, and two different gradation ranges for mixtures to be prepared with a basalt-limestone combination. Gradation curves and gradation limits are shown in Figure 7.1, Figure 7.2, and Table 7.1, Table 7.2, respectively.

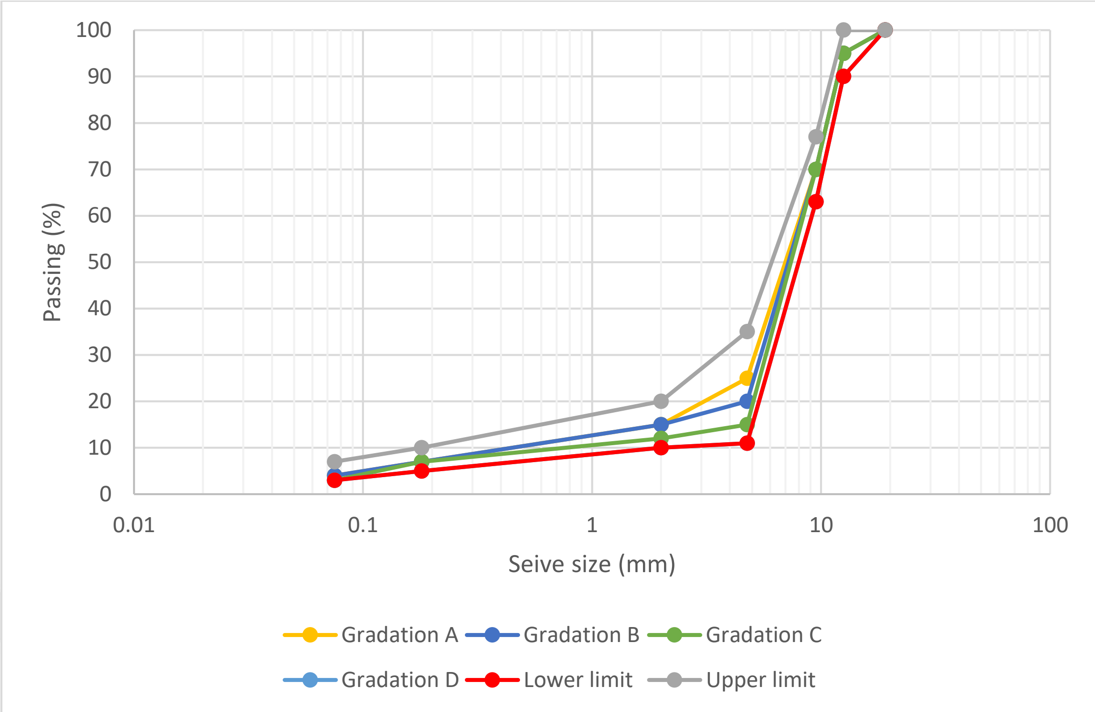


Figure 7.1 Gradation curves involving limestone aggregate

Table 7.1 Only limestone aggregate gradation types

Sieve Size		Gradation-A	Gradation-B	Gradation-C	Gradation-D	Lower and Upper Limit
in, No	mm	Passing	Passing	Passing	Passing	
3/4	19	100	100	100	100	100
1/2	12.5	95	95	95	90	90-100
3/8	9.5	70	70	70	63	63-77
No.4	4.75	25	20	15	11	11-35
No.10	2.00	15	15	12	10	10-20
No.80	0.18	7	7	7	5	5-10
No.200	0.075	4	4	3	3	3-7

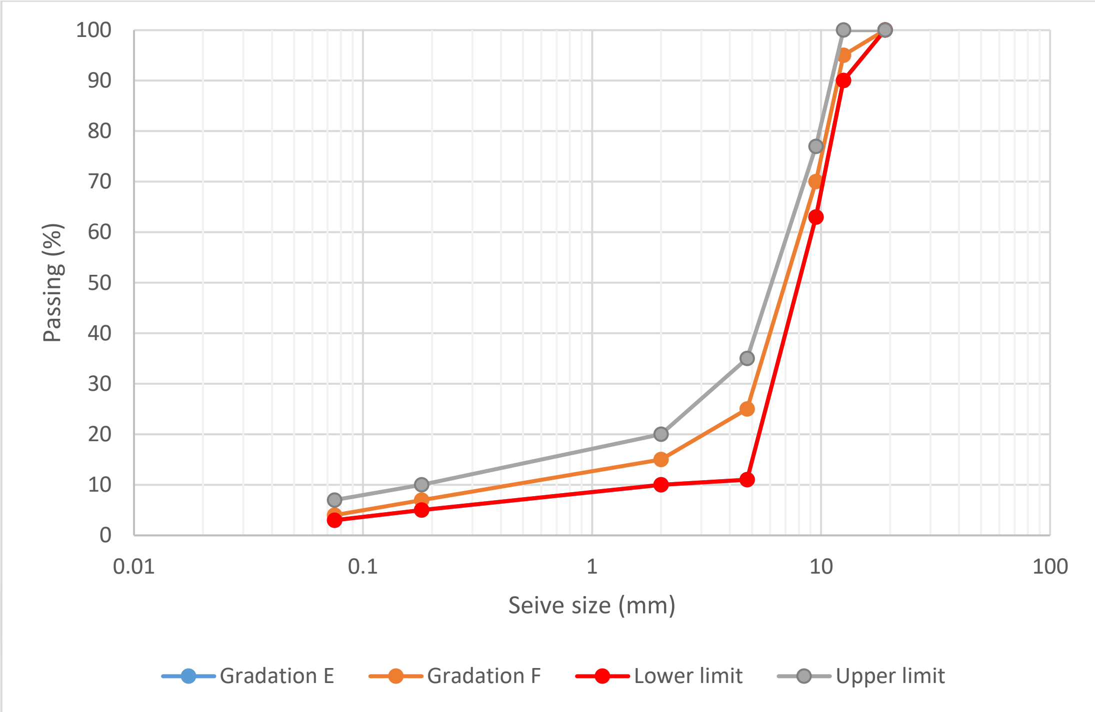


Figure 7.2 Basalt-limestone aggregate combination curves

Table 7.2 Basalt-limestone aggregate combination gradation types

Sieve Size		Gradation-E	Gradation-F	Lower and Upper Limit
in, No	mm	Passing	Passing	
3/4	19	100	100	100
1/2	12.5	90	95	90-100
3/8	9.5	63	70	63-77
No.4	4.75	11	25	11-35
No.10	2.00	10	15	10-20
No.80	0.18	5	7	5-10
No.200	0.075	3	4	3-7

Then, three samples were prepared for each gradation with 4.5% bitumen content (it should be noted that 4,5% is trial bitumen content. Exacts OBC will be determined in the following chapter. Mixture samples were prepared by compressing with a Marshall compactor at 145±5°C with 50 compactions on both sides as specified in the technical specification. The air void ratio and stability values of the test specimen, which were rested for 24 hours at room temperature, were found in accordance with the test standards. The air void ratio was found according to TS EN 12697-6 procedure D, which is defined to find the void ratio of PAPs.

7.1.1 Test with Limestone Aggregate

Table 7.3 presents the design sample's air void ratio and stability together with specification limits. Although all the gradation types provided the lower stability specification limit none of the gradation types provided air voids specification limit. However, Gradation-D was preferred in the samples to be prepared using limestone within the scope of this thesis. Since the samples were prepared with a bitumen content of 4.5%, it is predicted that the void ratio will probably increase at a lower bitumen content compared to the 4.5 % trial bitumen content. The following sections will explain the void ratio in different bitumen contents. Marshall stability values were within specification limits in all gradation sets.

Table 7.3 Results of different gradations with only limestone aggregate

Gradation Type	**Air Void Ratio (%)**		**Stability (kg)**	
	Results	**Specification limit**	**Results**	**Specification limit**
A	13.18	18-22	572	300 (min)
B	14.1		545	
C	17.14		405	
D	17.45		375	

7.1.2 Test with Basalt – Limestone Aggregate Mixtures

When the test results of the test specimens prepared for the basalt-limestone combination are interpreted, it is thought that it would be more appropriate to use the Gradation-F as a basalt-limestone aggregate combination in this thesis. Because it is noticed that the air void ratios calculated in the first aggregate combination are above the specification limits and the Marshall stability values are below the lower limit of 300 kg, it is thought that one of the main reasons why the gradation curve used in the limestone aggregate does not yield results in the basalt-limestone aggregate combination is that the basalt aggregates cannot find a suitable compaction surface with each other during compaction. Another reason, basalt aggregate has shown higher draindown due to the fact that basalt-type aggregate exhibits lower adhesion characteristics. Therefore, the draindown bitumen causes losses in terms of stability.

There are two types of aggregate gradation. Gradation E cannot stay within the specification limits due to lower stability value (lower than 300 kg) and air void ratio (greater than 22%). Although Gradation-F meets the specification limits in terms of stability value, it remains below the limit regarding air void ratio. However, Gradation-F was preferred in the samples to be prepared using a basalt-limestone combination within the scope of this thesis. Since the specimens were prepared with a bitumen content of 4.5%, the void ratio is predicted to increase at low bitumen content. The following sections will explain the void ratio in different bitumen contents. The results of the designed samples are illustrated in Table 7.4.

Table 7.4 Results of different gradations limits with basalt-limestone aggregate combinations

Gradation Type	**Air Void Ratio (%)**		**Stability (kg)**	
	Results	**Specification limit**	**Results**	**Specification limit**
E	23.05	18-22	161	300 (min)
F	17.25		443	

7.2 Mixing-Compaction Temperature Determination

Worldwide specifications often do not specify a method for determining the mixing-compaction temperatures of porous asphalt. In Turkey, compaction temperature was determined as 145±5 °C when using modified bitumen. The main purpose of this section is to determine optimum mixing-compaction temperatures. For this reason, three temperature levels (140°C, 145°C, and 150°C) have been taken in precount. So as the determine exact temperature level through particle loss test.

Three samples were prepared involving 4.0 % bitumen content take into above determined gradation level (This bitumen content is not the optimum bitumen content (OBC). OBC is explained in the following chapters.) of the mixture weight for each bitumen type (base(unmodified) and SBS, Elvaloy and Sasobit modified) at different mixing-compaction temperatures. The prepared PAMs were subjected to the abrasion test after being kept at room temperature for two days. Considering the effect of the room temperature in which the experiment was carried out on the test results, care was taken to conduct the test under appropriate conditions.

7.2.1 Mixing-Compaction Temperature Results with Only Limestone Aggregate

Particle loss values of mixtures prepared only limestone aggregate with base bitumen and SBS, Elvaloy, Sasobit modified bitumen at three different mixing-compaction temperatures are given in Figure 7.3 and Figure 7.4. The results shown are the average values of three samples prepared for each bitumen.

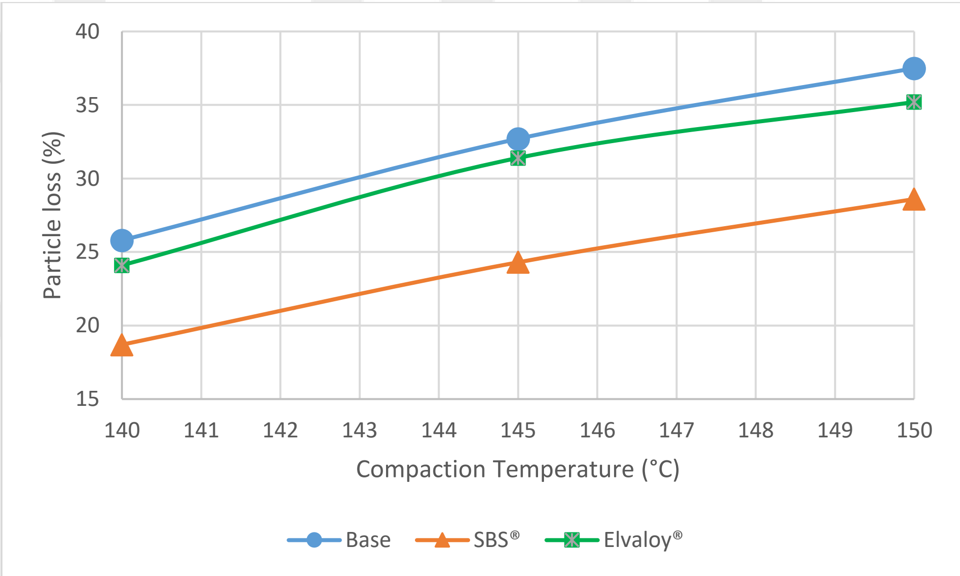


Figure 7.3 Particle loss results regarding limestone aggregate with PMB

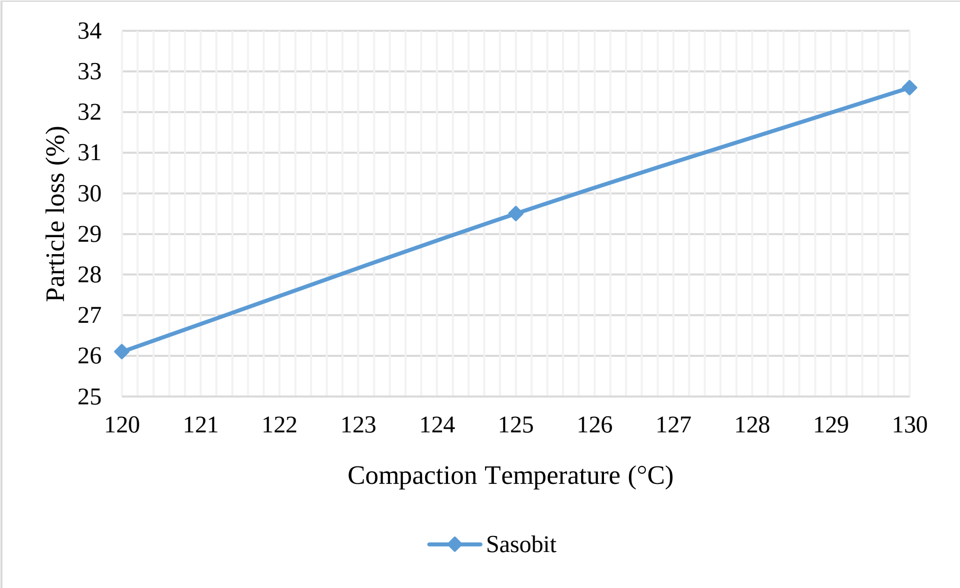


Figure 7.4 Particle loss results regarding limestone aggregate with Sasobit WMA additive

When the particle loss values of PAMs prepared using only limestone aggregate are examined, the compaction temperature of the sample is determined as 140 °C when unmofied bitumen and SBS Elvaloy polymer modified bitumen is used, and 120 °C when Sasobit modified bitumen is used. The main reason for determining these values is the compaction temperature at which the loss of parts is minimum.

7.2.2 Mixing-Compaction Temperature Results with Basalt-Limestone Combination

Samples were prepared with the basalt-limestone aggregate combination mixture, base bitumen and SBS, Elvaloy, Sasobit modified bitumen, and samples containing bitumen up to 4% of the mixture weight at three different mixing-compaction temperatures. The values were found by taking the average of the particle loss results of the mixtures prepared as three samples for each bitumen variety. Relevant particle loss values are shown in Figure 7.5 and Figure 7.6.

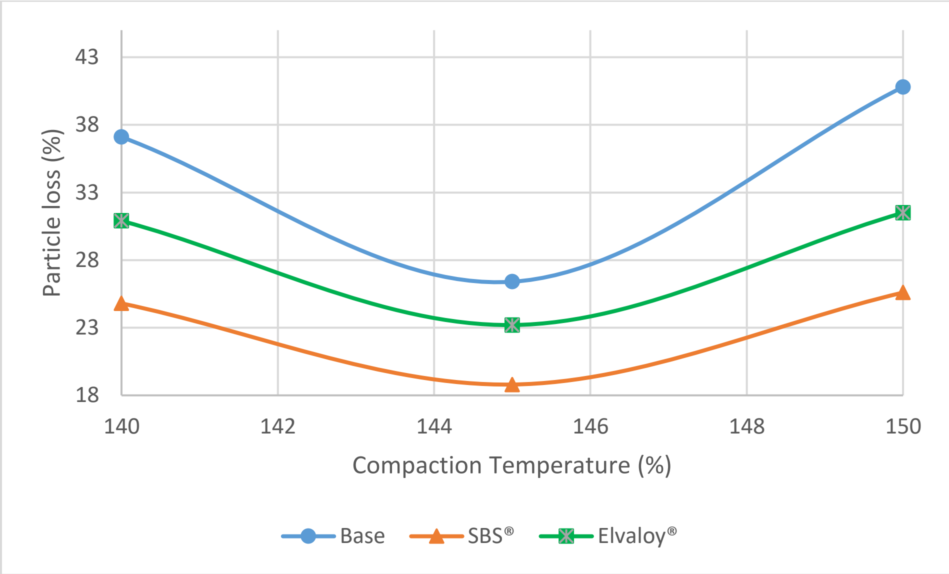


Figure 7.5 Particle loss results regarding basalt-limestone aggregate with PMB

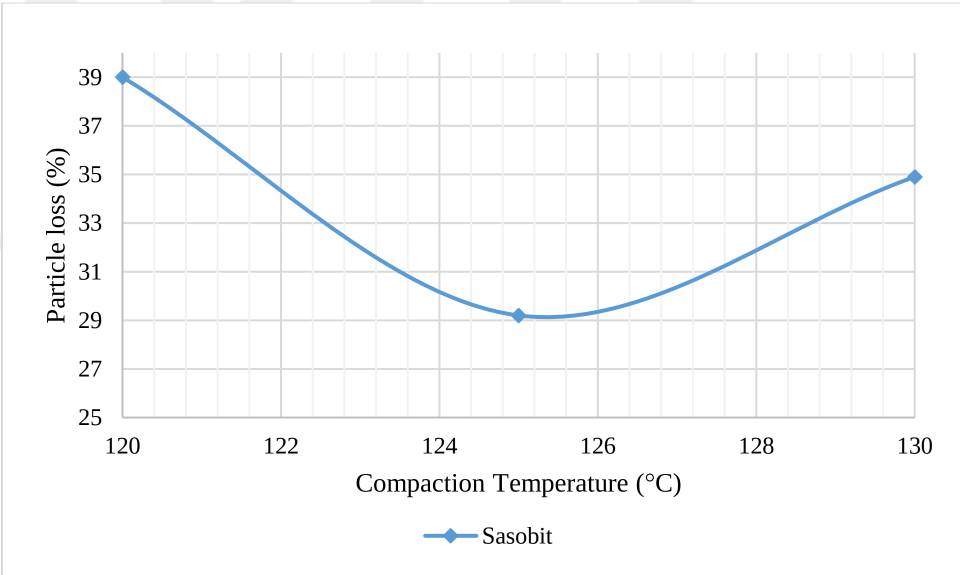


Figure 7.6 Particle loss results regarding basalt-limestone aggregate with Sasobit WMA additive

The optimum compaction temperature was determined according to the Cantabro loss values of the PAM samples prepared using a basalt-limestone aggregate combination mixture. This design gave the minimum particle loss value at a higher compaction temperature than the mixtures made using only limestone aggregate. It is thought that one of the reasons for this is the use of different aggregate gradations. For the design made with this aggregate mixture and using unmodifed bitumen and SBS, Elvaloy polymer modified bitumen, the optimum compaction temperature was determined as 145°C. When Sasobit modified bitumen was used, it was determined as 125°C. The main factor in determining these temperatures is the compression temperature, which gives the minimum value of particle loss.

When the results obtained using two different aggregate groups and four different bitumen types were compared, it was seen that a higher compaction temperature was required for the basalt-limestone aggregate combination mixture compared to the design using only limestone. The main reason for this is that a higher compression temperature is required in order to enable the aggregate particles to adhere to each other in the mixture so that two different aggregate types can perform as a single material in the mixture.

It is considered necessary to find an optimum mixing-compaction temperature with the Cantabro loss approach within the limits of the compaction temperature specified in the specification in order to better represent the field conditions of the mixture to be designed and applied to the field.

According to the results obtained, the mixing-compression temperature to be used in the experiment is summarized in Table 7.5.

Table 7.5 Selected compaction temperature

Bitumen Type	**Compaction Temperature, °C**	
	Limestone	**Basalt-limestone**
Base	140	145
SBS modified	140	145
Elvaloy Modified	140	145
Sasobit Modified	120	125

7.3 Particle (Cantabro) Loss Test Results

The particle (Cantabro) loss test is a test applied only to PAP. In porous asphalt design criteria of the Turkish Highways Technical Specification, the maximum particle loss rate is determined as 20%.

Samples with different bitumen contents were prepared using two aggregate types (limestone only and basalt-limestone combination), base and modified bitumen (SBS, Elvaloy, and Sasobit). The samples were tested after keeping them at room temperature for 48 hours.

The results are given in Figures 7.7 and 7.9, prepared with only limestone aggregate and basalt-limestone aggregate combination, respectively.

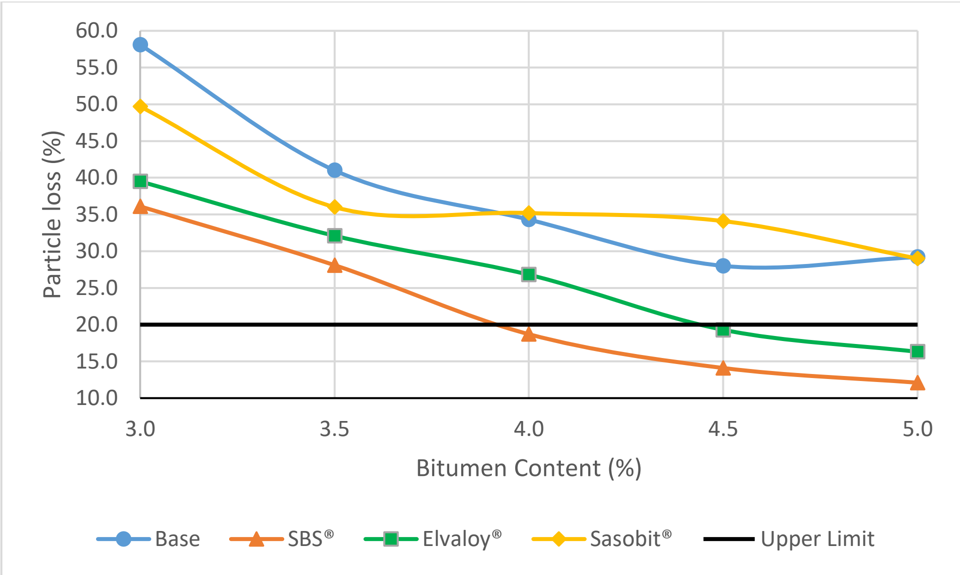


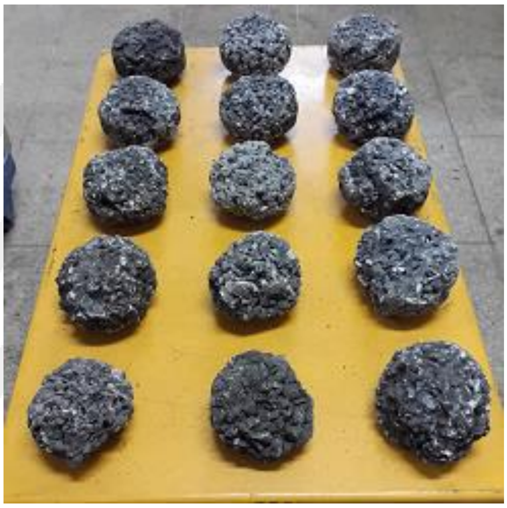
Figure 7.7 Particle loss results regarding limestone aggregate

It is seen that the bitumen should be used in a modified form in the particle loss test performed on the samples prepared only with limestone aggregate. Particle loss values of the samples prepared with base bitumen exceeded the specification upper limit. In addition, when bitumen modified with Sasobit was used, the particle loss value exceeded the specification upper limit. Therefore, it is seen that it is not appropriate to use only limestone aggregate and WMA additive in the design of porous asphalt. The specimen prepared using SBS and Elvaloy polymer-modified bitumen were below the 20% particle loss value, which is the maximum limit of the specification in specific bitumen contents (4%, 4.5%, and 5% for SBS, 4.5% and 5% for Elvaloy). Figure 7.8 illustrated the appearance of the specimens prepared with limestone after the particle loss test.

a)

b)

c)



d)

Figure 7.8 Samples after particle loss test prepared with limestone aggregate a) base bitumen, b) SBS pmb modified, c) Elvaloy pmb modified, d) Sasobit wma modified

In the particle loss results of the specimen prepared with the basalt-limestone combination, it is seen that the use of modified bitumen is required in this aggregate combination, as in the designs using only limestone. In the samples prepared with SBS and Elvaloy polymer-modified bitumen, 4%, 4.5%, 5%, and 4.5%, 5% bitumen contents, respectively, met the specification limit. However, Sasobit which contributed WMA did not give the desired values in the particle loss results. This combination, on the contrary, showed worse results than the mixtures prepared with base bitumen.

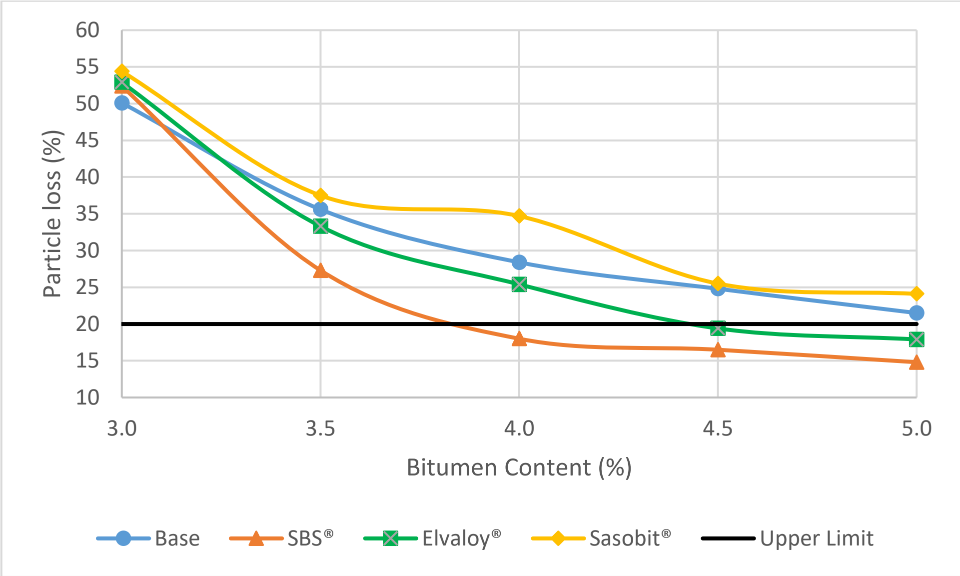


Figure 7.9 Particle loss results with basalt-limestone aggregate combination

Considering the particle loss results of the mixtures prepared with both aggregate types, it was observed that the samples prepared only with Sasobit as modified bitumen did not give the desired values. Therefore, within the scope of the thesis, it was decided not to use the WMA additive in determining the stability and air void ratios with the Marshall design, which is the next stage. The designs will be made with only SBS and Elvaloy modified bitumen and the results will be examined. Figure 7.10 shows the appearance of the specimen consist of basalt-limestone aggregate combination after the particle loss test.

a)

b)

c)

d)

Figure 7.10 Samples after particle loss test prepared limestone- basalt aggregate combination a) base, b) SBS modified, c) Elvaloy modified, d) Sasobit modified

7.4 Marshall Mix Design Results

Marshall design was made based on the determined gradation range, compaction temperature, and particle loss results. The air void ratios and stability values of the samples produced in this section were examined and their compliance with the specification was determined. Then, using these data, it is aimed to determine the OBC for porous asphalt. The method for finding the optimum bitumen content is explained in the next section.

The specimen was prepared using SBS and Elvaloy polymer-modified bitumen. Optimum additive contents were chosen as 5% for SBS and 1.5% for Elvaloy based on past studies (Sengoz, B., & Isikyakar, G. 2008). In addition, it aims to examine modifiers' effect on the design by preparing samples with base bitumen. The height, diameter, and dry weight of all prepared samples were determined. Using these values, air void ratios were calculated in accordance with TS EN 12697-6 procedure D. The obtained values are illustrated in Figure 7.11 and Figure 7.12, respectively, only limestone and basalt-limestone combinations.

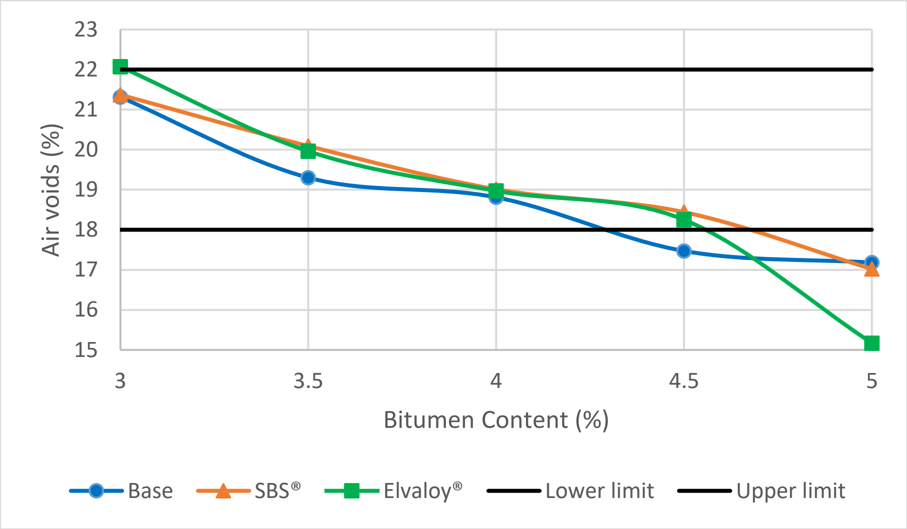


Figure 7.11 Air void ratios regarding limestone aggregate

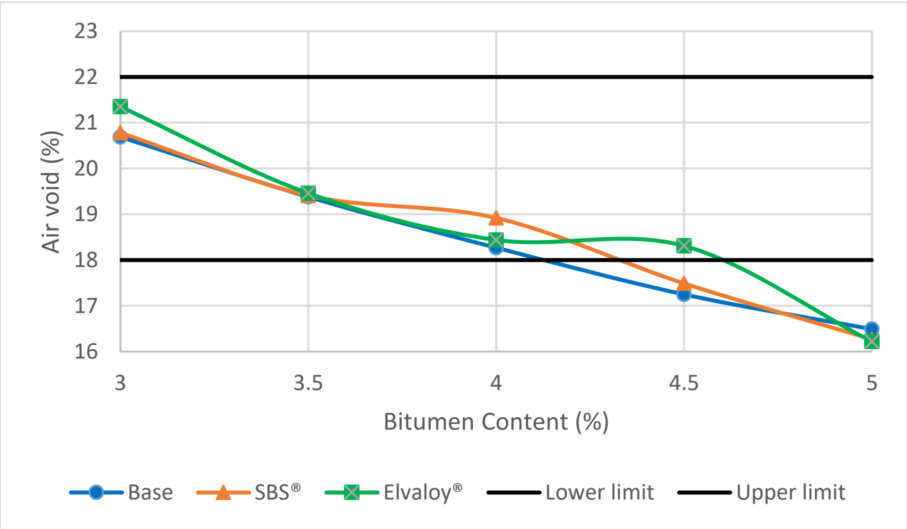


Figure 7.12 Air void ratios for used basalt-limestone combination aggregate

After the air void ratios were determined, the yield and stability of the samples were measured using the Marshall stability and flow test device. Marshall stability values are shown in figures 7.13 and 7.14 respectively only limestone and basalt-limestone combinations. The test results are given in detail in Appendix A.

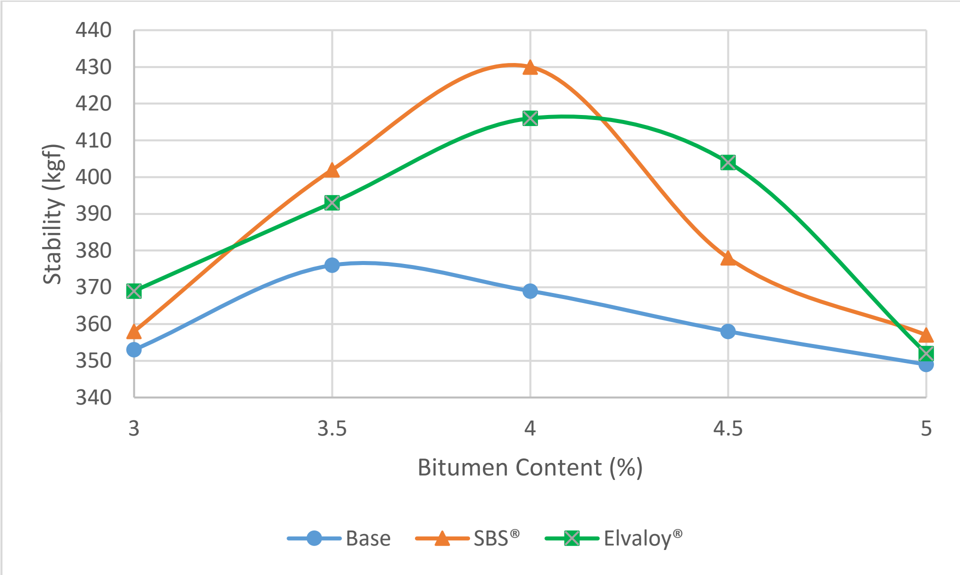


Figure 7.13 Marshall stability values regarding limestone aggregate

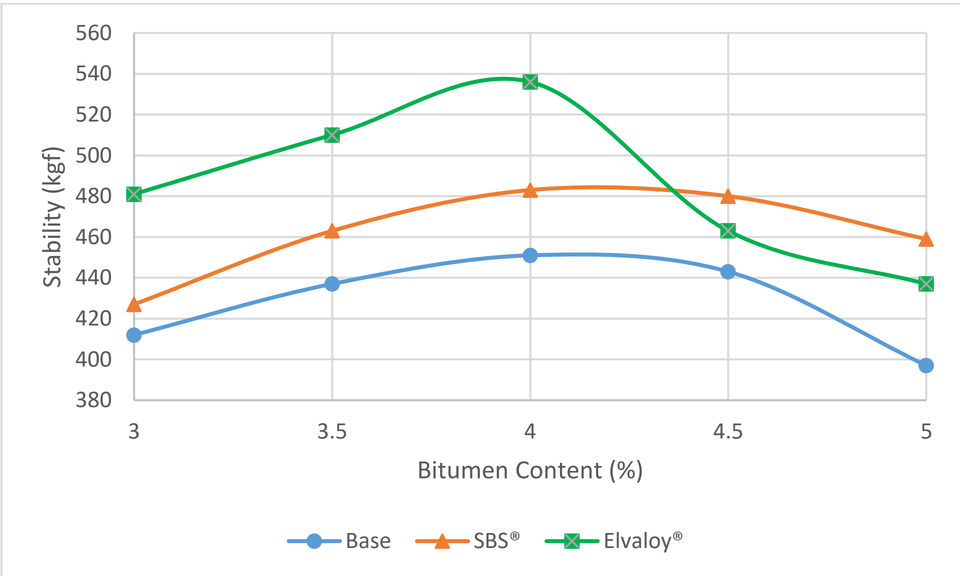


Figure 7.14 Marshall stability values for used basalt-limestone combination aggregate

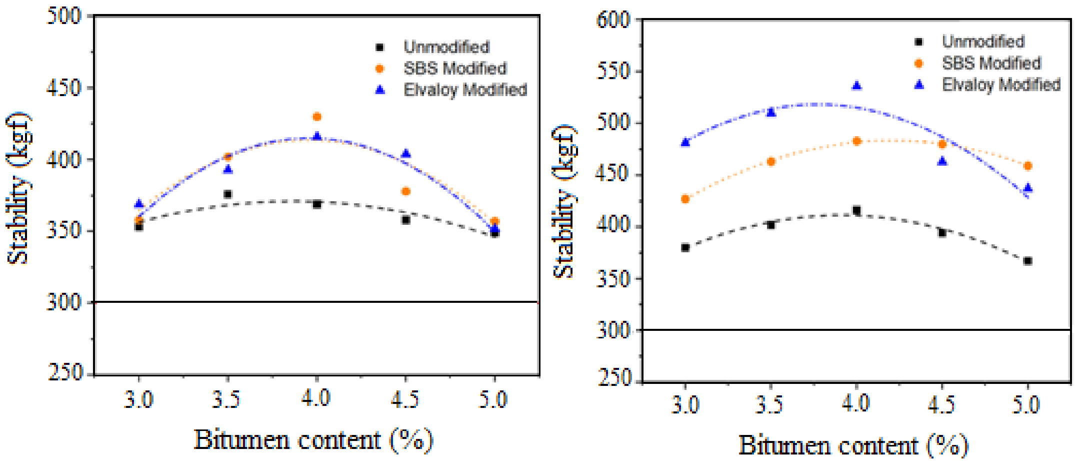
When the test results of the specimens consist of limestone aggregate for the base, SBS and Elvaloy polymer-modified bitumen are examined, all the mixtures for all bitumen contents provided stability above the minimum value of 300 kg, which is the minimum limit of the specification. Similarly, the stability results for the basalt-limestone aggregate combination for all specimens demonstrated values above the minimum stability limit. It can also be noticed that the basalt-limestone combination depicted better results in terms of stability because of the fact that basalt aggregate is more durable.

However, when the air void content values were examined for the limestone samples, it was seen that the samples prepared with base bitumen did not meet the requirement at 4.5% and 5.0% bitumen content. While the samples prepared with polymer-modified bitumen did not meet the specification of air void content limits of 18-22% at the 5.0% bitumen content for the SBS modified samples and at 3% and 5.0% for the Elvaloy modified samples. For the basalt-limestone aggregate combination, the air void content results showed that the test specimen with base bitumen and SBS modified bitumen fell out of the standard limits at 4.5% and 5.0% bitumen content. In comparison, samples prepared with Elvaloy did not meet the specification air void limits at a bitumen content of 5.0%.

7.5 Optimum Bitumen Content Determination

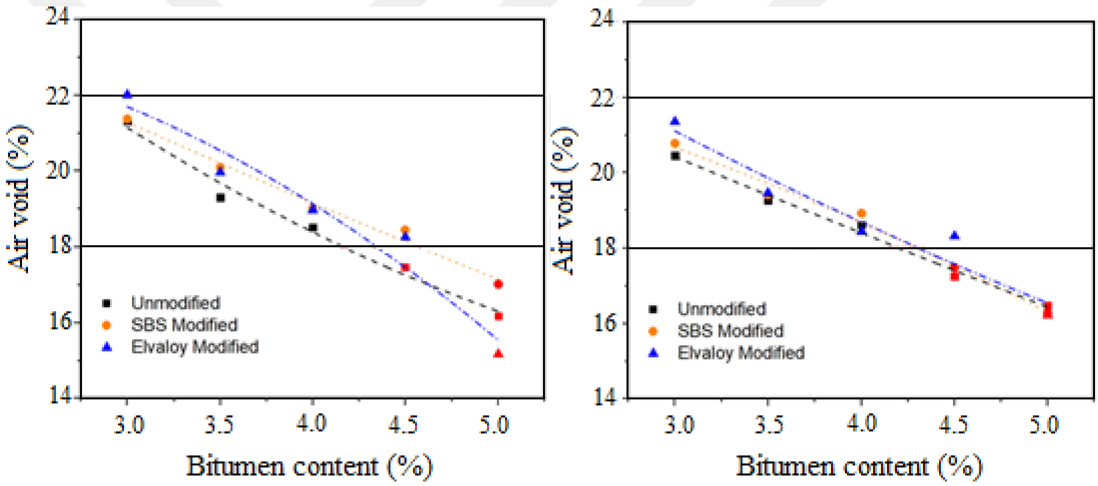
7.5.1 Turkish (KGM) Specification Results

The base bitumen, SBS, and Elvaloy modified porous asphalt samples produced by Marshall design method were tested for stability, air void content, and cantabro particle loss so as to calculate the OBC as presented in Figure 7.15-7.17 for both Series#1 (only limestone) and Series#2 (combination of limestone and basalt aggregate). Accordingly, the optimum bitumen content is defined as the value that can provide minimum stability of 300 kg, air void content between 18-22%, and maximum particle(cantabro) loss of 20%. In the figures, the upper and lower limit values of the standard were indicated as straight black lines while the samples which do not meet the criteria were specifically shown in red color.



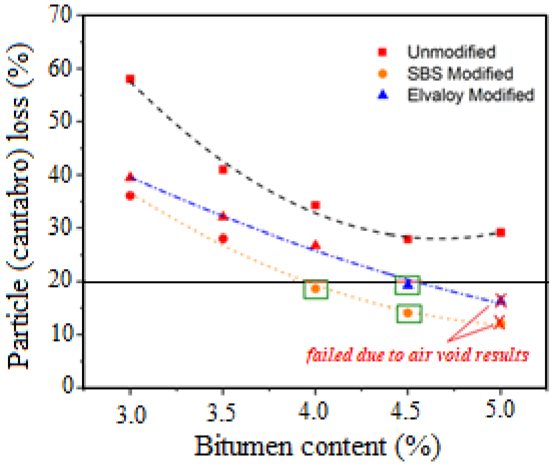
a) b)

Figure 7.15 Marshall Stability results of the mixtures a) Series#1 b) Series#2

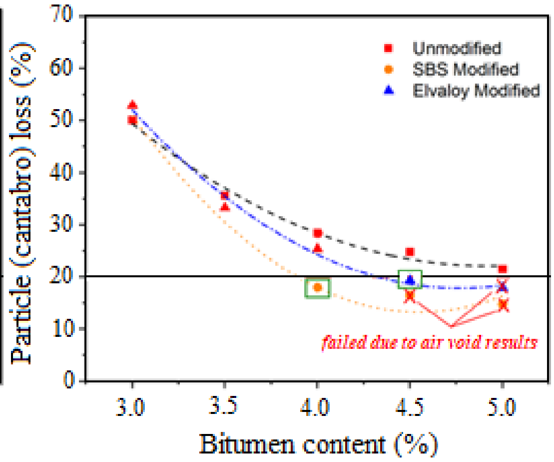


a) b)

Figure 7.16 Air void results of the mixtures a) Series#1 b) Series#2



a)



b)

Figure 7.17 Particle (cantabro) loss results of the mixtures a) Series#1 b) Series#2

As it is seen in Figure 7.15, Marshall stability results of specimens for both Series#1 and Series#2 are above the standard limit (300 kg). The polymer modification provided better Marshall stability value compared to prepared with unmodified bitumen samples. It can also be noted that, the stability values of the mixtures prepared with Series#2 are higher than the one with Series#1 for each bitumen content. This is associated with the better durability properties of the basalt aggregate. Additionally, it is observed that, the polymer modified samples had similar Marshall stability results for the mixtures prepare with limestone aggregate. However, when the basalt aggregate is included Series#1, Elvaloy modified mixture yielded significant improvement in terms of stability compared to SBS modified mixture.

As can be seen in Figure 7.16, it is obvious that the air void content declines by the rise in the bitumen content. The air void results met the criterias up to 4.5% bitumen content for both aggregate types prepared with base and polymer modified bitumens. On the other hand, when the bitumen content utilization is 4.5%, the unmodified mixture prepared with both aggregate series failed to meet the criteria. For the same bitumen content, the Elvaloy polymer modified mixture involving both aggregate series provided the acceptable air void content. Similar evaluation can not be made for the SBS polymer modified mixture involving basalt aggregate. Both of the unmodified and polymer modified samples containing of 5% bitumen did not meet the specification requirement for each aggregate series.

As depicted in Figure 7.17, the particle loss test results express that the samples prepared with unmodified bitumen do not meet the 20% upper limit of the specification for any bitumen content or aggregate series. For all samples prepared with unmodified and modified bitumen, the addition of basalt aggregate resulted in higher cantabro loss. Polymer modified samples meets the requirement when the bitumen content was 4.5% and 5% for both aggregate series. Additionally, SBS modified samples containing 4% bitumen satisfied the criteria for both aggregate series.

Regarding determination of optimum bitumen content based on Figure 7.15-7.17, it is not convenient to use the unmodified bitumen for the PAP production regardless of the aggregate type. For the Elvaloy modified samples, the OBC was designated as 4.5% for both type of aggregate series since the other bitumen contents did not meet the standard limits. Related to SBS modified samples involving Series#1, the optimum bitumen content could be 4 or 4.5% since they both met the criterias. However, as indicated in Fig 7.15; the stability values yielded better results with 4% SBS polymer modified bitumen content compared to the one 4.5%. Therefore 4% is determined as optimum bitumen content for Series#1. On the other hand when the basalt aggregate is introduced 4% is accepted as optimum SBS polymer modified bitumen content, since other bitumen contents failed to meet the criterias.

7.5.2 The determined OBC for ASTM D7064 Standard

The choice of OBC according to the ASTM 7064 specification is based on the values of air voids, Cantabro loss, and bitumen draindown of the samples. Related datas are shown in Table 7.6 and the highlighted cells in the table are representing the values which did not meet the specification limits. For Series#1, 4% and 4.5% bitumen contents of the mixture prepared with SBS polymer-modified bitumen can be selected as the OBC according to the specification. Series #1 meets the lower and upper specification limits only at a bitumen content of 4.5%for the mixture prepared with Elvaloy polymer-modified bitumen. Therefore, the OBC of the Elvaloy modified mixture was determined as 4.5%.

For Series#2, the mixture using with SBS modified bitumen meets the lower and upper limits of the specification only at a bitumen content of 4.0%. Therefore, 4.0% was defined as the OBC of the SBS modified mixture. Regarding to Elvaloy modified samples, only 4.5% bitumen content met the specification limits, which yielded the OBC as 4.5%. The OBC results are illustrated in Figure 7.18.

Table 7.6 OBC results based on ASTM D7064 Specification

Bitumen Type	Properties	Series#1					Series#2				
		Bitumen content (%)					Bitumen content (%)				
		3	3.5	4	4.5	5	3	3.5	4	4.5	5
SBS Polymer modified	Air Void Ratio (%)	21.37	20.09	19.01	18.44	17.02	20.78	19.41	18.92	17.49	16.27
	Particle (Cantabro) loss (%)	36.1	28.1	18.7	14.1	12.1	52.4	27.3	18	16.5	14.8
	Draindown	0.01	0.03	0.07	0.1	0.16	0.02	0.06	0.1	0.18	0.24
Elvaloy polymer modified	Air Void Ratio (%)	22.07	19.96	18.97	18.25	15.17	21.35	19.46	18.44	18.31	16.22
	Particle (Cantabro) loss (%)	39.5	32.1	26.8	19.3	16.3	52.9	33.3	25.4	19.4	17.9
	Draindown	0.02	0.03	0.06	0.11	0.19	0.04	0.08	0.1	0.12	0.23

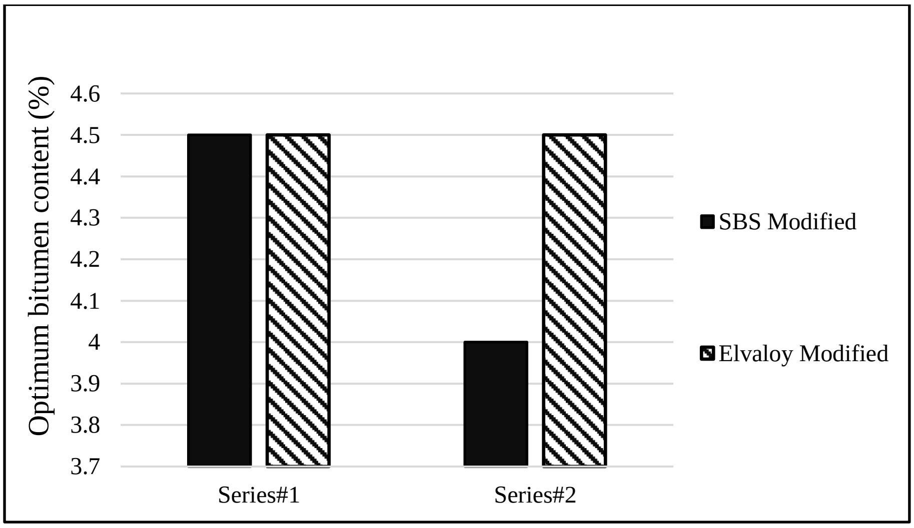


Figure 7.18 OBC for ASTM D7064 specification

7.5.3 The determined OBC for Chinese Standard

The OBC of the samples prepared according to the Chinese specification is determined by calculating the average of the lower and upper limit bitumen values. For Series#1, the OBC of the mixture prepared with SBS modified bitumen was determined as 4.05 %, while it was determined as 4.1 % for the samples consist of Elvaloy polymer-modified bitumen.

For Series#2, the OBC of the samples were determined as 4.15% and 4.1% for SBS and Elvaloy modified mixtures, respectively. The minimum and maximum bitumen content limits of the samples prepared with different polymers and aggregate series are shown in Figure 7.19-7.22 and the OBC results are presented in Figure 7.23.

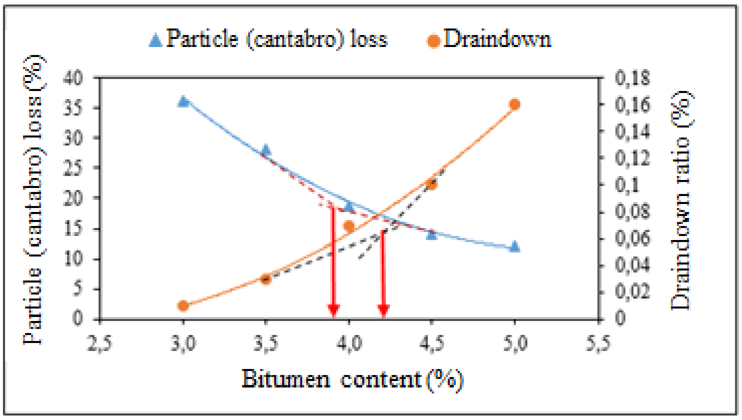


Figure 7.19 Series#1 with SBS polymer modified

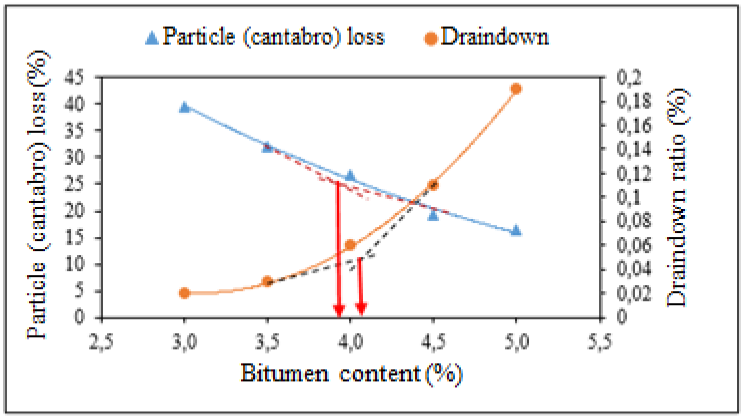


Figure 7.20 Series#1 with Elvaloy polymer modified

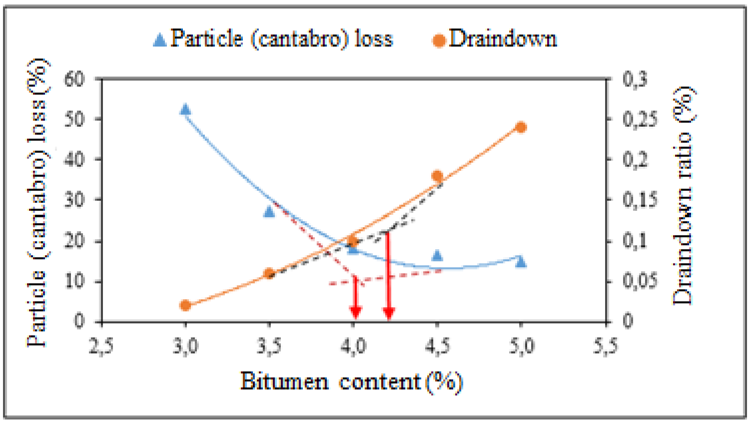


Figure 7.21 Series#2 with SBS polymer modified

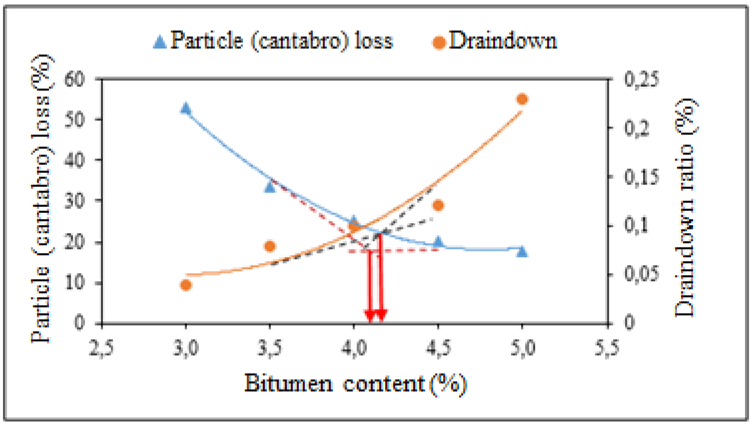


Figure 7.22 Series#2 with Elvaloy polymer modified

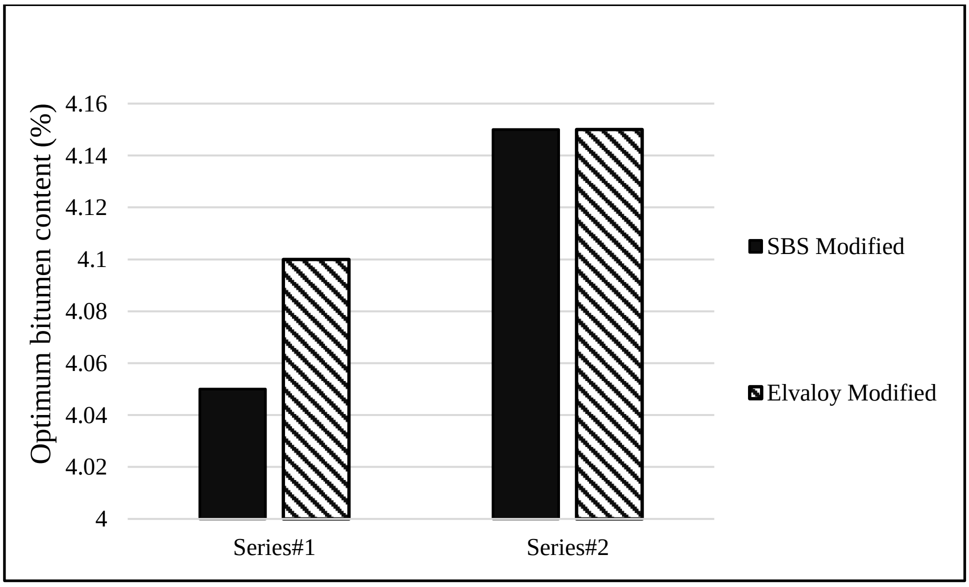


Figure 7.23 OBC for Chinese specification

7.5.4 The determined OBC for Malaysian Standard

The OBC of the samples obtained according to the Malaysian specification was determined by evaluating the particle loss and bitumen draindown properties of the samples. Cantabro loss and draindown test results of the samples are shown in Table 7.7 and the highlighted cells in the table are representing the values which did not meet the specification limits. As the prepared samples with SBS polymer-modified bitumen for Series#1 meet the lower and upper limits of the specification at 4.5% and 5.0% bitumen content, these percentages can be selected as the OBC range. Because the Cantabro loss values of the samples prepared with Elvaloy polymer-modified bitumen did not meet the specification limits for any bitumen content, the OBC could not be calculated for the samples prepared with this additive.

Given that the mixture prepared with SBS modified bitumen for Series#2 meets the lower and upper limits of the specification at only 5% bitumen content, this value was determined as the OBC. Similar with the Series#1, the Cantabro loss value of the samples prepared with Elvaloy modified bitumen did not meet the specification limit, when the aggregate series was chosen as Series#2. Therefore, the OBC could not be calculated for these samples. The OBC results are shown in Figure 7.24.

Table 7.7 OBC Results based on Malaysian Specification

Bitumen Type	**Properties**	**Series#1**					**Series#2**				
		Bitumen Ratio (%)					**Bitumen Ratio (%)**				
		3	**3.5**	**4**	**4.5**	**5**	**3**	**3.5**	**4**	**4.5**	**5**
SBS polymer modified	**Particle (Cantabro) loss (%)**	36.1	28.1	18.7	14.1	12.1	52.4	27.3	18	16.5	14.8
	Draindown	0.01	0.03	0.07	0.1	0.16	0.02	0.06	0.1	0.18	0.24
Elvaloy polymer modified	**Particle (Cantabro) loss (%)**	39.5	32.1	26.8	19.3	16.3	52.9	33.3	25.4	19.4	17.9
	Draindown	0.02	0.03	0.06	0.11	0.19	0.04	0.08	0.1	0.12	0.23

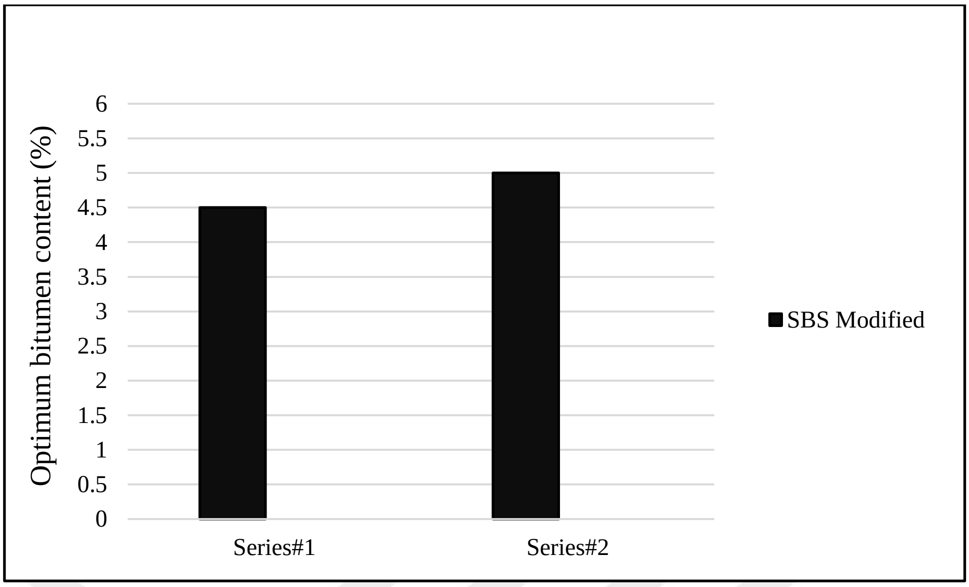


Figure 7.24 OBC for Malaysian specification

7.5.5 The determined OBC for Indonesian Standard

The OBC of the samples prepared according to the Indonesian specification was determined as the content that meets the specified limits of stability, Cantabro loss, air void, and bitumen draindown values of the samples. Concerned values are shown in Table 7.8 and the highlighted cells are representing the values which did not meet the Indonesian specification limits. The mixture prepared with SBS modified bitumen for Series#1 provides the minimum and maximum limits of the specification at 4.0%, 4.5%, and 5.0% bitumen contents. In this case, 4% bitumen content, which corresponds to the maximum stability, was determined as the OBS. Since the samples prepared with Series#1 and Elvaloy polymer-modified bitumen remained within the specification limits at only 4.5% bitumen, this value was determined as the OBC.

For Series#2, the prepared sample with SBS polymer-modified bitumen met the specification limits at 4.0% and 4.5% bitumen contents. Regarding to that,the OBC was chosen as 4.0%, which provides the maximum stability. Elvaloy modified bitumen only met the lower and upper specification limits at 4.5% bitumen content, which was selected as the OBC. The OBC results are shown in Figure 7.25.

Table 7.8 OBC Results based on Indonesian Specification

Bitumen Type	Properties	Series#1					Series#2				
		Bitumen Ratio (%)					Bitumen Ratio (%)				
		3	3.5	4	4.5	5	3	3.5	4	4.5	5
SBS polymer modified	Stability (kg)	358	402	430	378	357	427	463	483	480	459
	Air Void Ratio (%)	21,37	20.09	19.01	18.44	17.02	20.78	19.41	18.92	17.49	16.27
	Particle (Cantabro) loss (%)	36.1	28.1	18.7	14.1	12.1	52.4	27.3	18	16.5	14.8
	Draindown	0.01	0.03	0.07	0.1	0.16	0.02	0.06	0.1	0.18	0.24
Elvaloy polymer modified	Stability (kg)	369	393	416	404	352	481	510	536	463	437
	Air Void Ratio (%)	22.07	19.96	18.97	18.25	15.17	21.35	19.46	18.44	18.31	16.22
	Particle (Cantabro) loss (%)	39.5	32.1	26.8	19.3	16.3	52.9	33.3	25.4	19.4	17.9
	Draindown	0.02	0.03	0.06	0.11	0.19	0.04	0.08	0.1	0.12	0.23

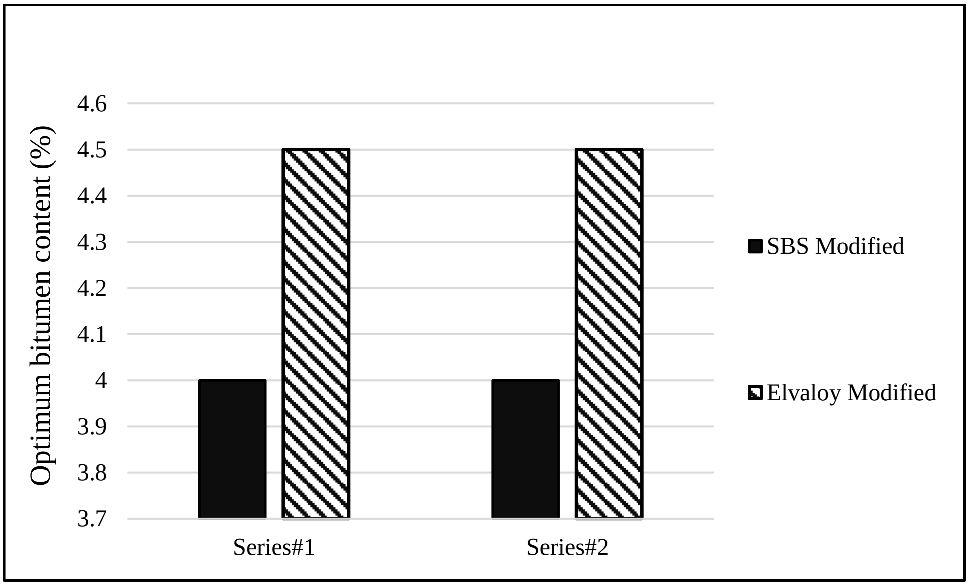


Figure 7.25 OBC for Indonesian specification

7.6 Indirect Tensile Strength Ratio (ITSR) Results

Once the gradation level and OBC for both types aggregates (Series#1 and Series#2) has been determined the ITS test has been tested on the prepared sample. ITS values of wet samples (ITSw) and dry samples (ITSd) together with ITSR values are presented in Figure 26.

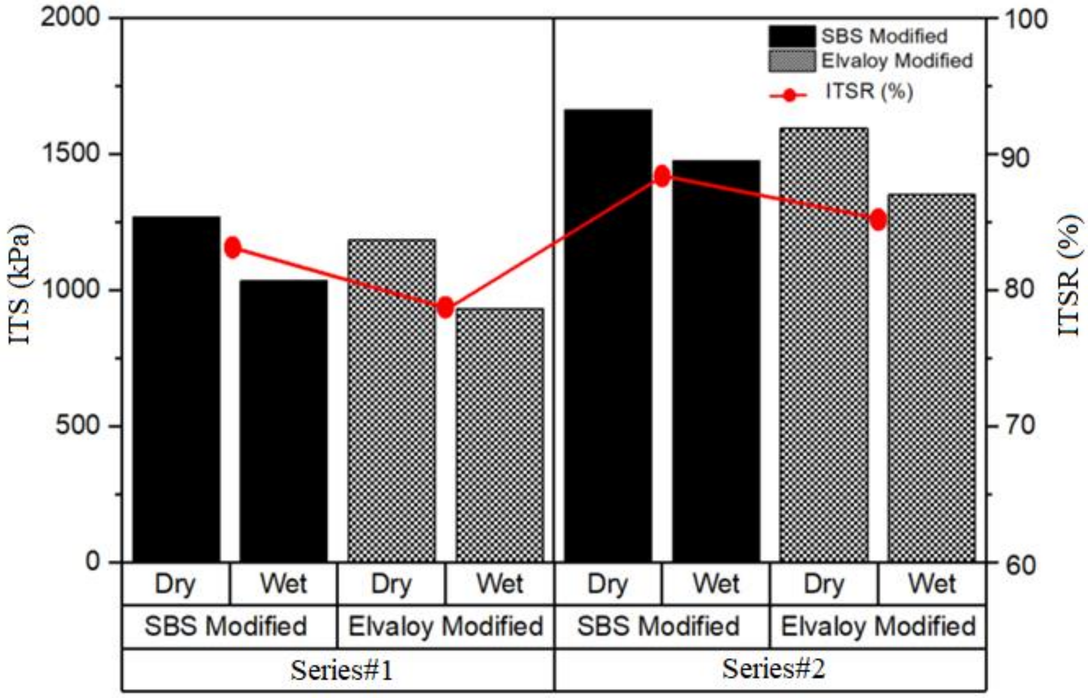


Figure 7.26 ITS and ITSR values of the samples

As indicated in Figure 7.26, the ITS results of dry specimen are higher than conditioned specimens. Additionally, regardless of the conditioning (wet or dry) and the type of the aggregate, the SBS modified samples had depicted slightly higher ITS value compared to the Elvaloy modified samples.

On the other hand it is possible to conclude that among the two types of aggregate series, the samples consists of Series#2 had substantially higher ITS value, which is due to the higher durability characteristics of the basalt aggregate.

As depicted in Figure 7.26, SBS modified samples yielded higher ITSR values in comparison to the Elvaloy modified samples. On the other hand, regarding to aggregate types, it was observed that samples having Series#2 had higher ITSR values, which is in concordance with previous results.

7.7 Permeability Results

The bitumen content directly affects the permeability properties of the sample. If the bitumen content in the PAP is higher than the optimum, the porous structure of the mixture fills with excess bitumen and results in blocked pores. This reduces the permeability of PAP, and the pavement cannot serve effectively.

In order to consider the permeability characteristics of the polymer modified PAMs prepared with the determined optimum bitumen contents, the permeability tests were conducted. The results are depicted in Figure 7.27.

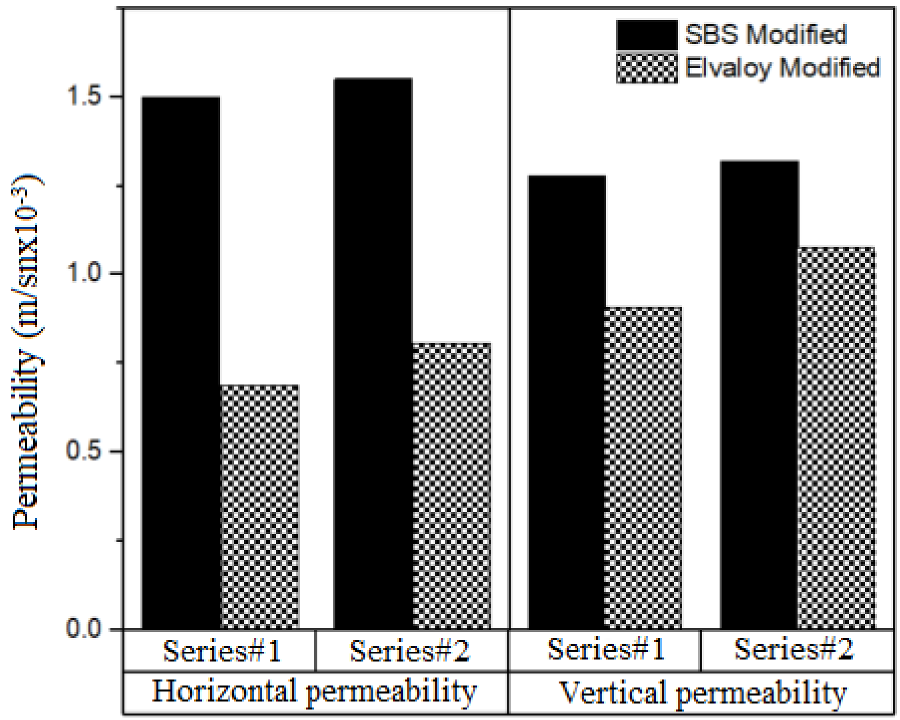


Figure 7.27 Permeability results of the modified samples

The results showed that for both series, the horizontal and vertical permeability for the SBS modified samples is higher ratio than the Elvaloy modified specimens. This can be associated with higher bitumen content regarding Elvaloy modified mixtures .

As indicated in Figure 7.27, among the both type of polymer modified mixtures, Series#2 which contains combination of limestone and basalt aggregate exhibited slightly higher permeability results. This is directly related with the basalt involving coarse aggregate proportion of the Series#2. The introduction of the basalt within the mixture requires more compaction effort which may affect permeability characteristics.

7.8 Bitumen Draindown Results

The draindown results are presented in Figure 7.28. As mentioned to the results, the draindown value (%) for the SBS polymer-modified specimen are higher than Elvaloy polymer-modified samples, for only limestone and combination of limestone and basalt aggregate (both series of aggregates). This is attributed both to the viscous behavior of SBS polymer modified bitumen as well as the less bitumen content within the SBS modified mixture.

Besides as indicated in Figure 7.28, the aggregate type within the mixture has an impact on the draindown results. The combination of limestone and basalt aggregate (Series#2) involving basalt aggregate has shown higher draindown due to the fact that basalt type aggregate exhibits lower adhesion characteristics (Karacasu & Akalin, 2020).

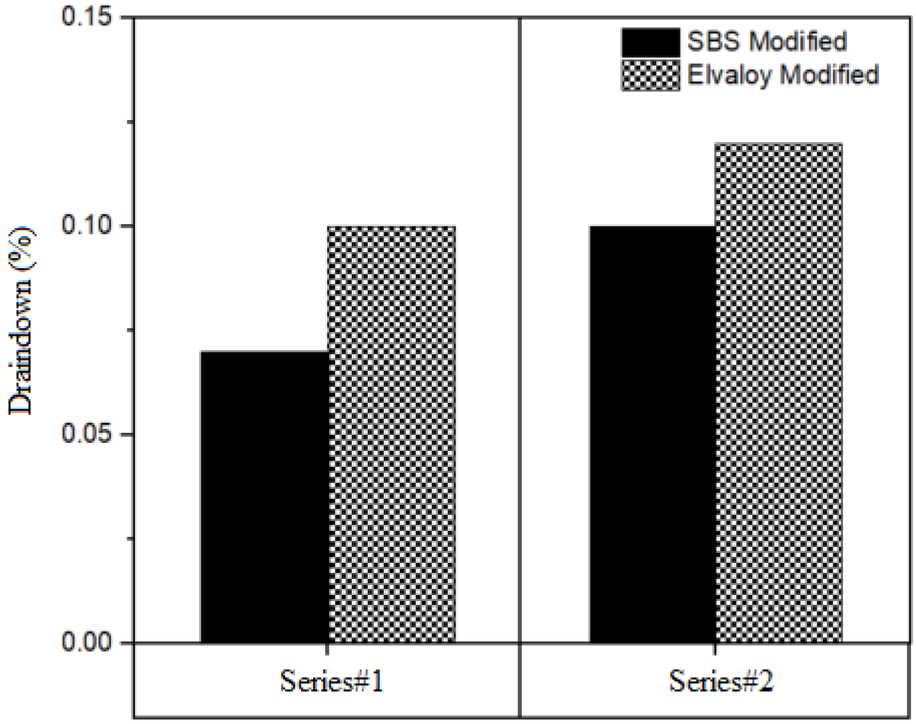


Figure 7.28 Draindown test result

CHAPTER EIGHT
CONCLUSION AND RECOMMENDATION

In this study, the performance of PAMs was investigated by using different polymer-modified bitumen additives (elastomeric and reactive elastomeric terpolymer) and WMA additive from a more environmentally friendly perspective. PAMs were prepared with two aggregate types (limestone and basalt-limestone combination) and three bitumen modifying additives. The necessary tests were carried out and compared the results of the mixtures. The results of the experiments have led to the following conclusions:

1. The air void ratio values of the specimens produced for the preliminary design vary between 13% and 18% when limestone aggregate is used and between 17% and 23% when a combination of basalt-limestone aggregate is used. In the Highways Technical Specification, porous asphalt air void ratios should be between 18-22% for type-2 gradation. In this case, the gradations with the air void ratio closest to 18%, which is the lower limit of the specification, were selected. Gradation-D was chosen for the samples to be prepared using only limestone aggregate, and Gradation-F for the mixtures to be prepared using the basalt-limestone aggregate combination.
2. Mixing-compaction temperatures were found to be 140°C for mixtures prepared with unmodified and polymer modified bitumen using limestone aggregate, and as 120°C for mixtures prepared using WMA modified bitumen. The compression temperature of the mixtures prepared with pure and polymer modified bitumen using a basalt-limestone aggregate combination was 145°C, and the compression temperature of the mixtures prepared using WMA modified bitumen was 125°C.
3. Based on the particle loss test results, SBS and Elvaloy polymer modified bitumen mixtures exhibited better results than unmodified and WMA modified bituminous mixtures in mixtures prepared using limestone and basalt-limestone aggregate combination in each bitumen content. It was observed that WMA mixtures could not give the desired performance in porous asphalt designs, since the particle loss values of the mixtures prepared using WMA additive remained above the specification upper limit value (20%). It is seen that the particle loss values of the

mixtures prepared with SBS polymer modified bitumen give better results in every bitumen content compared to Elvaloy. It has been observed that the mixture prepared with an SBS additive has a higher resistance to the forces created by the vehicle wheels.

4. From the point of Marshall mixture design parameters, the stability value of the designs containing basalt-limestone aggregate combination gives higher values than the designs prepared with limestone. While the mixtures prepared with SBS polymer modified additive and limestone aggregate alone gave higher results than Elvaloy polymer modified bitumen, the mixtures prepared with the combination of Elvaloy polymer modified bitumen and basalt-limestone aggregate gave higher stability than the mixtures prepared with SBS polymer modified bitumen. The air void ratios of the mixtures were higher in both aggregate types when they were prepared using SBS polymer modified bitumen.
5. While determining the OBC, both the particle loss value and the air void ratio of the prepared samples were checked. Therefore, the optimum bitumen content of the mixtures using SBS and Elvaloy polymer modified bitumen with both aggregate groups was determined as 4% and 4.5%, respectively.
6. Based on the ITS test, it was observed that the mixtures prepared in OBC with both additives and aggregate types were resistant to deformations caused by the temperature change of the water in the cavities. The ITS ratio was determined as 82% for the samples prepared with limestone aggregate and SBS polymer modified bitumen, and 89% when the basalt-limestone aggregate combination was used. The ITS ratio of the samples prepared with Elvaloy polymer modified bitumen was 80% when limestone aggregate was used, while it was 85% when the basalt-limestone aggregate combination was used. It is seen that the samples prepared with basalt-limestone aggregate combination are weaker against water-induced deformations.
7. According to the permeability test results, the vertical and horizontal permeability values of the mixtures produced in OBC with both PMB and aggregate types were in the range of 0.5×10^{-3}-3.5×10^{-3} m/sec specified in the specification. Both horizontal and vertical permeability values of the PAM produced by using SBS polymer modified bitumen additive were found to be higher for both aggregate

groups than those produced with Elvaloy polymer modified bitumen. The air void ratio datas of the samples produced with both mixtures are very close to each other. Therefore, the connection of the gap channels in the mixture produced with SBS polymer modified bitumen additive is better than the mixture produced with Elvaloy polymer modified bitumen additive.

8. Regarding the bitumen draindown test, the mixtures produced in OBC with both PMB and aggregate types remained below the maximum limit of 0.3% bitumen draindown. In the case of using both additives, it was observed that the leaching of the thick bitumen layer formed on the aggregates from the aggregate was low. Mixtures prepared using SBS polymer modified bitumen and two aggregate types gave lower bitumen draindown rate than mixtures using Elvaloy polymer modified bitumen. According to these test results, there is no need to use cellulose fiber, which prevents bitumen draindown in both mixtures.

In this study, it has been certify that PAP can be used effectively on highways with bitumen modification. It is strongly recommended to determine the applicability of the designs made in the laboratory environment with advanced performance tests, together with the core samples to be taken from the trial PAP cuts.

In addition, the WMA additive (Sasobit) used in the study did not exhibit desired results. In other studies, designs and performance experiments can be applied using different WMA additives.

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APPENDICES

APPENDIX A
MARSHAL MIX DESIGN

Table A.1 Marshall mix design data for unmodified bitumen regarding limestone aggregate

Specimen No	Bitumen		Specimen height (mm)				Specimen diameter (mm)				Weight in air	Bulk Specific Gravity	Max. Teo. Specific Gravity	Voids	V.M.A	V.F.A	Flow	Stability	Correlation Fact.	Corr. Stability
#	Wa,%	g	1	2	3	Avg.	1	2	3	Avg.	gr	Dp	Dt	Vh	%	%	mm	kgf		kgf
1	3.00	34.50	71.9	71.6	71.6	71.7	101.3	101.4	101.2	101.3	1170.6	2.027					2.13	407	0.826	336
2	3.00	34.50	71.9	71.9	72.2	72.0	101.3	101.5	101.4	101.4	1172.3	2.017					2.12	429	0.823	353
3	3.00	34.50	72.4	72.3	72.1	72.3	101.4	101.3	101.4	101.4	1175.7	2.017					1.60	451	0.819	369
												2.021	2.568	21.31	25.90	17.7	1.95			353
4	3.50	40.25	70.9	71.0	71.1	71.0	101.4	101.4	101.3	101.4	1178.6	2.058					1.67	467	0.839	392
5	3.50	40.25	71.2	71.1	71.4	71.2	101.4	101.1	101.2	101.2	1177.7	2.056					1.81	422	0.836	353
6	3.50	40.25	71.4	71.6	71.4	71.4	101.4	101.3	101.2	101.3	1185.6	2.060					2.08	462	0.832	384
												2.058	2.550	19.30	24.91	22.5	1.85			376
7	4.00	46.00	71.5	71.8	71.5	71.6	101.3	101.4	101.3	101.3	1182.7	2.050					1.37	434	0.828	359
8	4.00	46.00	71.5	71.0	71.0	71.2	101.4	101.4	101.4	101.4	1181.5	2.058					2.97	460	0.836	385
9	4.00	46.00	71.4	71.6	71.7	71.6	101.3	101.3	101.2	101.2	1186.2	2.060					1.81	438	0.828	363
												2.056	2.532	18.81	25.34	25.7	2.05			369
10	4.50	52.00	70.6	70.6	70.7	70.7	101.4	101.3	101.4	101.4	1178.8	2.068					2.12	410	0.845	346
11	4.50	52.00	70.4	70.6	70.4	70.5	101.3	101.4	101.4	101.4	1181.0	2.078					1.67	421	0.849	357
12	4.50	52.00	70.3	70.6	70.5	70.4	101.3	101.2	101.3	101.3	1180.2	2.082					1.33	435	0.851	370
												2.076	2.515	17.47	24.98	30.1	1.71			358
13	5.00	57.50	70.7	70.6	70.4	70.6	101.3	101.3	101.4	101.3	1174.0	2.064					1.87	412	0.847	349
14	5.00	57.50	70.8	70.6	70.9	70.8	101.4	101.4	101.4	101.4	1184.0	2.073					1.66	418	0.843	352
15	5.00	57.50	70.4	71.0	70.9	70.8	101.6	101.2	101.2	101.3	1180.0	2.070					1.78	409	0.843	345
												2.069	2.498	17.18	25.58	32.8	1.77			349

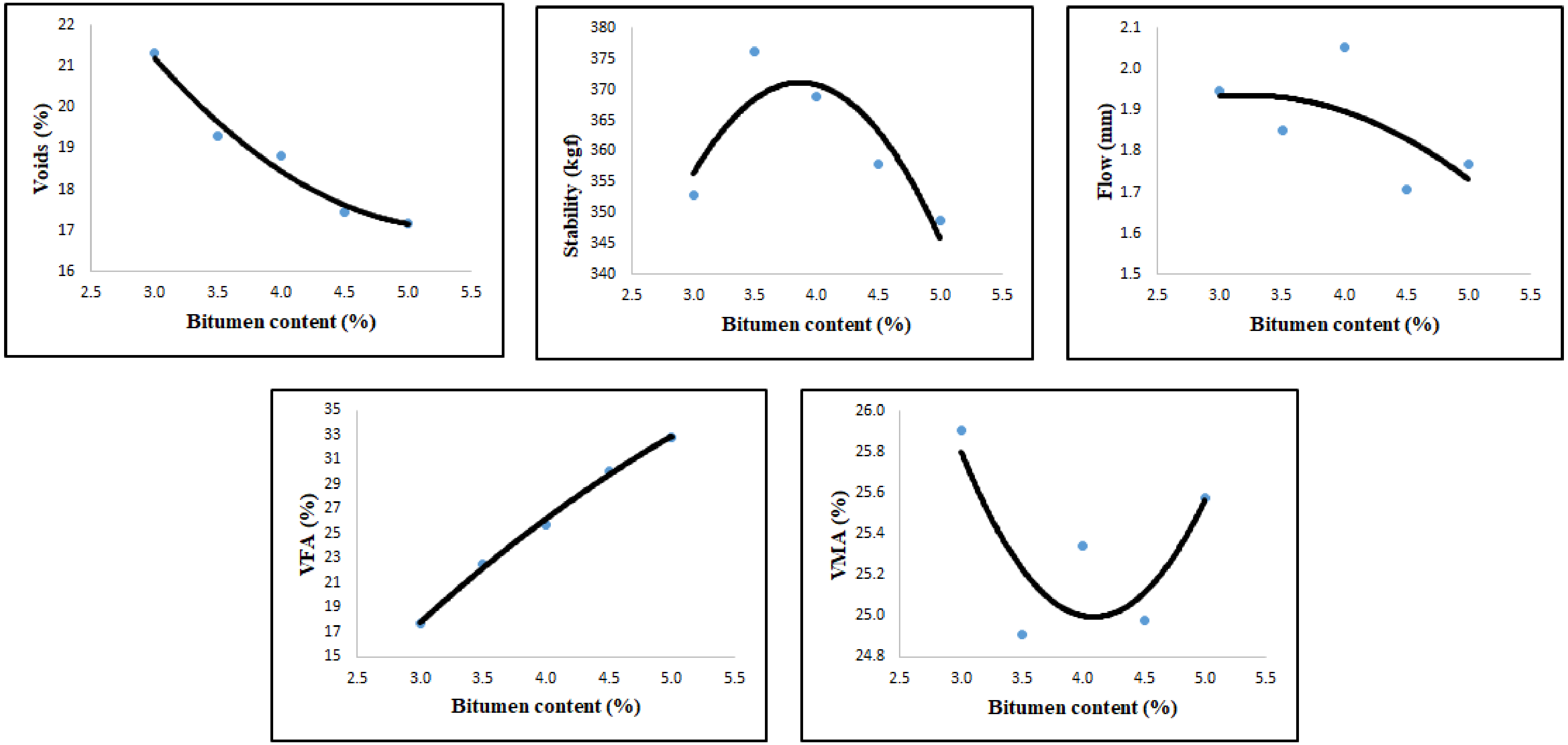

Figure A.1 Marshall mix design graphics for unmodified bitumen regarding limestone aggregate

Table A.2 Marshall mix design data for SBS PMB regarding limestone aggregate

Specimen No	Bitumen		Specimen height (mm)				Specimen diameter (mm)				Weight in air	Bulk Specific Gravity	Max. Teo. Specific Gravity	Voids	V.M.A	V.F.A	Flow	Stability	Correlation Fact.	Corr. Stability
#	Wa,%	g	1	2	3	Avg.	1	2	3	Avg.	gr	Dp	Dt	Vh	%	%	mm	kgf		kgf
1	3.00	34.50	72.2	72.4	72.5	72.3	101.4	101.2	101.3	101.3	1170.6	2.009					1.75	450	0.816	367
2	3.00	34.50	71.4	71.9	71.7	71.7	101.4	101.5	101.3	101.4	1172.3	2.027					2.34	432	0.827	357
3	3.00	34.50	72.2	71.9	72.4	72.2	101.2	101.5	101.2	101.3	1175.7	2.022					2.12	426	0.819	349
												2.019	2.568	21.37	25.95	17.7	2.07			358
4	3.50	40.25	71.7	71.8	71.9	71.8	101.2	101.4	101.3	101.3	1178.6	2.038					1.63	461	0.825	380
5	3.50	40.25	71.7	71.6	71.5	71.6	101.4	101.1	101.3	101.3	1177.7	2.043					1.84	520	0.829	431
6	3.50	40.25	72.2	72.6	72.4	72.4	101.4	101.3	101.3	101.3	1185.6	2.032					1.67	486	0.814	396
												2.038	2.550	20.09	25.64	21.7	1.71			402
7	4.00	46.00	71.4	71.6	71.2	71.4	101.2	101.4	101.4	101.3	1182.7	2.055					2.60	531	0.832	442
8	4.00	46.00	71.6	71.9	71.7	71.7	101.3	101.3	101.5	101.4	1181.5	2.043					2.20	496	0.826	410
9	4.00	46.00	71.8	71.5	71.7	71.7	101.3	101.2	101.4	101.3	1186.2	2.055					2.16	524	0.836	438
												2.051	2.532	19.01	25.52	25.5	2.32			430
10	4.50	52.00	71.5	71.7	71.4	71.5	101.3	101.3	101.4	101.3	1178.8	2.045					2.64	440	0.830	365
11	4.50	52.00	71.6	71.7	71.5	71.6	101.4	101.2	101.4	101.3	1181.0	2.046					2.53	463	0.829	384
12	4.50	52.00	70.8	71.2	71.0	71.0	101.4	101.3	101.3	101.3	1180.2	2.062					1.89	460	0.839	386
												2.051	2.515	18.44	25.87	28.7	2.35			378
13	5.00	57.50	70.7	70.7	70.6	70.7	101.3	101.3	101.3	101.3	1174.0	2.063					1.83	410	0.845	346
14	5.00	57.50	70.8	71.2	70.6	70.9	101.3	101.4	101.2	101.3	1184.0	2.074					2.30	436	0.841	367
15	5.00	57.50	70.3	70.6	70.2	70.4	101.3	101.3	101.3	101.3	1180.0	2.082					1.96	419	0.851	357
												2.073	2.498	17.02	25.44	33.1	2.03			357

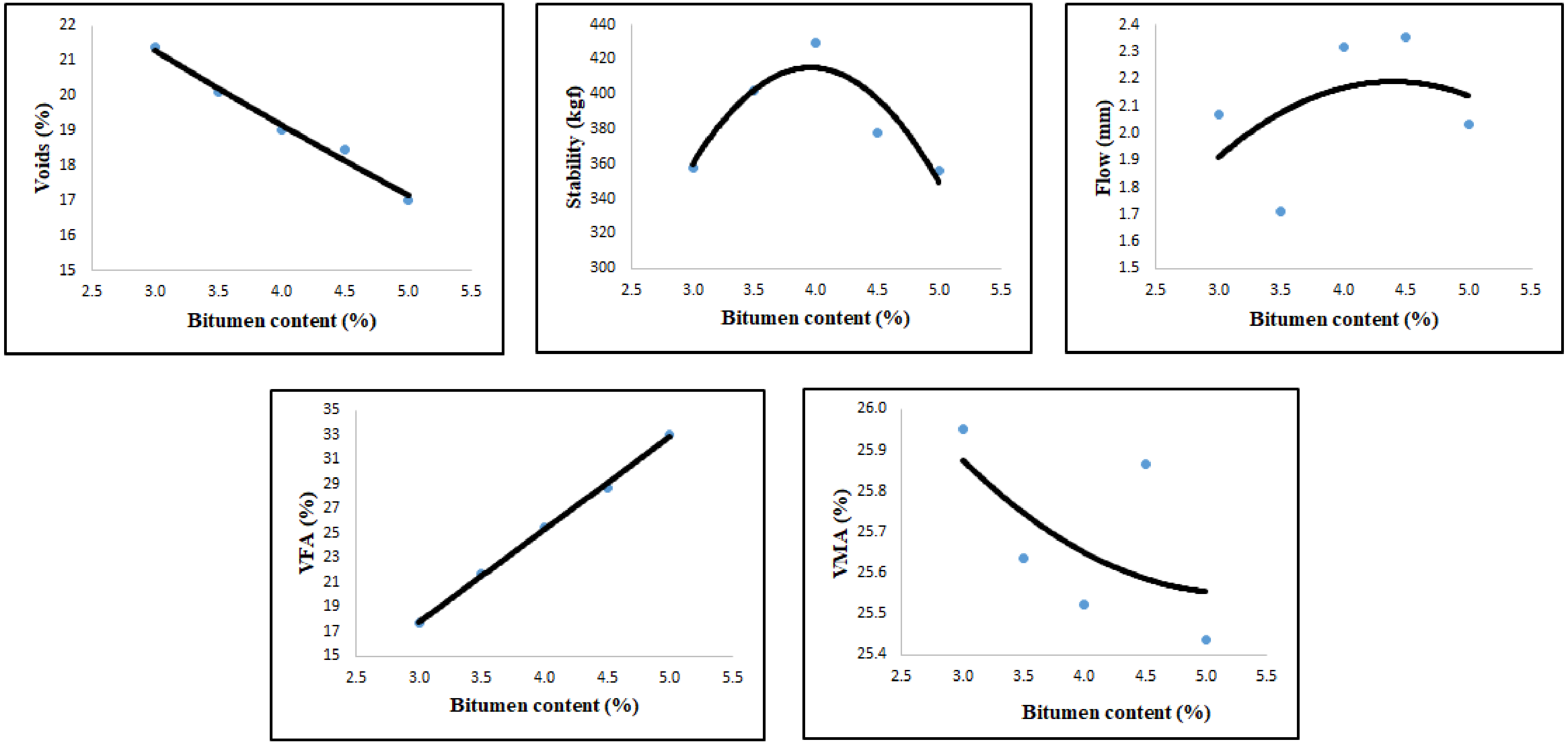

Figure A.2 Marshall mix design graphics for SBS PMB regarding limestone aggregate

Table A.3 Marshall mix design data for Elvaloy PMB regarding limestone aggregate

Specimen No	Bitumen		Specimen height (mm)				Specimen diameter (mm)				Weight in air	Bulk Specific Gravity	Max. Teo. Specific Gravity	Voids	V.M.A	V.F.A	Flow	Stability	Correlation Fact.	Corr. Stability
#	Wa,%	g	1	2	3	Avg.	1	2	3	Avg.	gr	Dp	Dt	Vh	%	%	mm	kgf		kgf
1	3.00	34.50	73.6	73.8	73.5	73.6	101.3	101.4	101.4	101.4	1179.2	1.985					2.68	405	0.785	318
2	3.00	34.50	72.5	72.1	72.5	72.4	101.3	101.3	101.3	101.3	1174.0	2.014					2.36	468	0.809	379
3	3.00	34.50	72.9	72.6	72.8	72.7	101.4	101.3	101.3	101.3	1175.1	2.004					2.47	510	0.803	410
												2.001	2.568	22.07	26.61	17.1	2.50			369
4	3.50	40.25	71.8	72.1	71.6	71.8	101.4	101.3	101.3	101.3	1182.5	2.043					1.99	480	0.821	394
5	3.50	40.25	72.7	72.8	72.7	72.7	101.3	101.3	101.4	101.3	1190.7	2.031					3.50	468	0.803	376
6	3.50	40.25	71.7	71.8	71.7	71.7	101.4	101.3	101.4	101.4	1185.5	2.049					2.14	496	0.823	408
												2.041	2.550	19.96	25.52	21.8	2.54			393
7	4.00	46.00	71.9	71.3	72.2	71.8	101.3	101.4	101.3	101.3	1192.8	2.062					3.01	520	0.821	427
8	4.00	46.00	72.0	72.1	71.9	72.0	101.3	101.4	101.4	101.4	1186.3	2.043					2.98	512	0.817	418
9	4.00	46.00	72.2	72.4	72.3	72.3	101.4	101.3	101.3	101.3	1194.7	2.051					2.78	496	0.811	402
												2.052	2.532	18.97	25.49	25.6	2.92			416
10	4.50	52.00	72.2	72.4	72.3	72.3	101.3	101.3	101.4	101.3	1180.1	2.025					2.42	424	0.837	355
11	4.50	52.00	71.5	71.2	72.4	71.7	101.4	101.3	101.4	101.4	1198.8	2.073					1.99	519	0.833	432
12	4.50	52.00	71.3	71.2	71.5	71.3	101.3	101.4	101.4	101.4	1190.9	2.070					2.01	500	0.847	424
												2.056	2.515	18.25	25.69	29.0	2.14			404
13	5.00	57.50	70.0	70.2	70.7	70.3	101.3	101.4	101.4	101.4	1197.8	2.112					2.24	452	0.851	385
14	5.00	57.50	70.0	70.2	70.2	70.1	101.3	101.3	101.4	101.3	1188.8	2.103					2.34	405	0.855	346
15	5.00	57.50	69.1	69.2	69.0	69.1	101.3	101.3	101.4	101.3	1193.3	2.142					2.69	373	0.875	326
												2.119	2.498	15.17	23.78	36.2	2.42			352

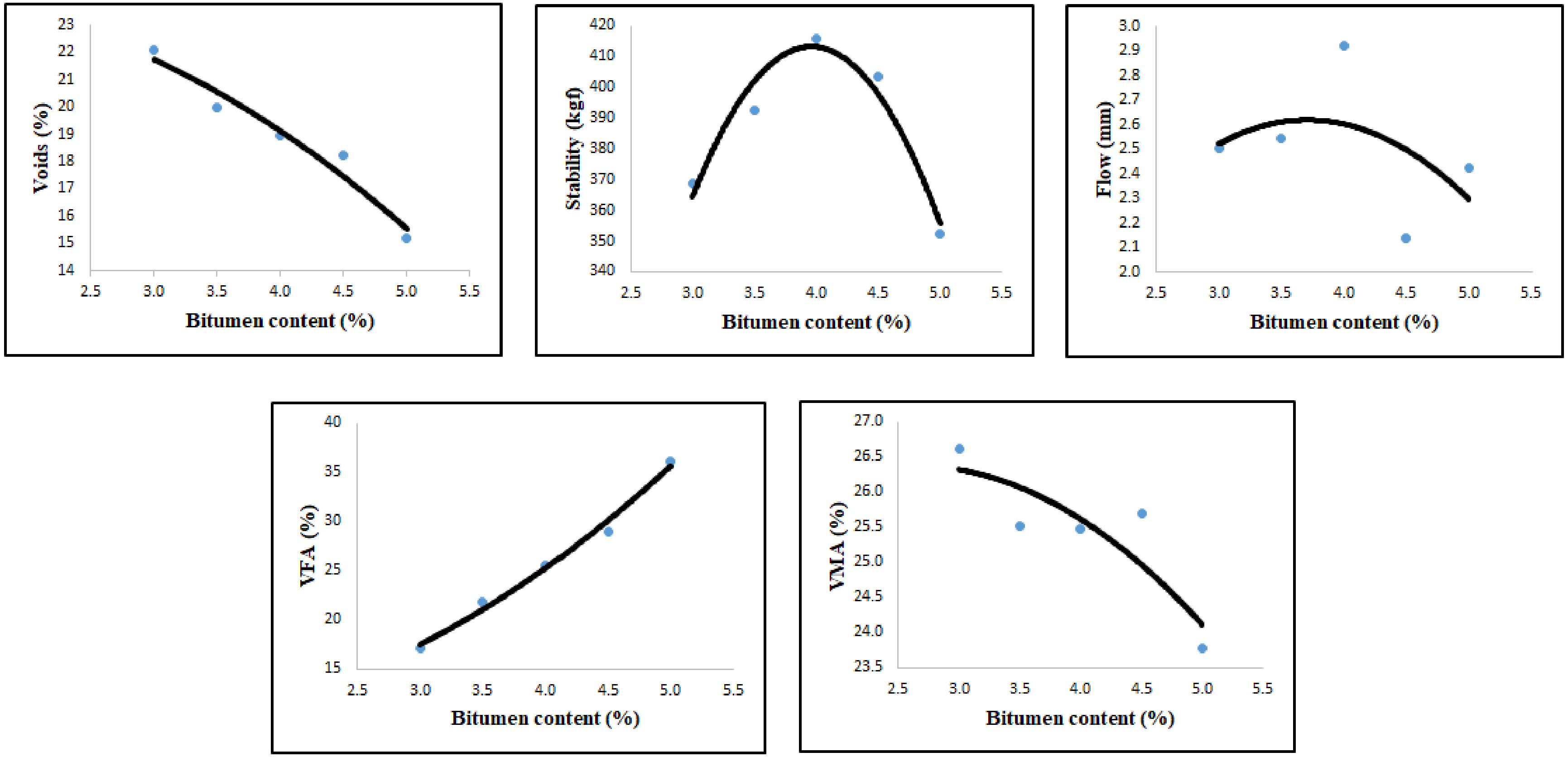

Figure A.3 Marshall mix design graphics for Elvaloy PMB regarding limestone aggregate

Table A.4 Marshall mix design data for unmodified bitumen with basalt-limestone aggregate combination

Specimen No	Bitumen		Specimen height (mm)				Specimen diameter (mm)				Weight in air	Bulk Specific Gravity	Max. Teo. Specific Gravity	Voids	V.M.A	V.F.A	Flow	Stability	Correlation Fact.	Corr. Stability
#	Wa,%	g	1	2	3	Avg.	1	2	3	Avg.	gr	Dp	Dt	Vh	%	%	mm	kgf		kgf
1	3.00	34.50	73.5	73.1	73.5	73.4	101.4	101.3	101.4	101.4	1172.8	1.982					1.82	476	0.789	376
2	3.00	34.50	73.0	73.4	73.1	73.2	101.5	101.5	101.4	101.5	1178.8	1.993					1.68	490	0.793	389
3	3.00	34.50	72.4	72.6	72.2	72.4	101.3	101.1	101.4	101.3	1170.5	2.008					1.92	488	0.789	385
												1.994	2.507	20.45	24.32	15.9	1.81			383
4	3.50	40.25	73.5	73.8	73.4	73.6	101.4	101.5	101.4	101.4	1189.4	2.002					1.89	510	0.785	400
5	3.50	40.25	72.5	72.8	72.3	72.5	101.4	101.5	101.5	101.5	1184.8	2.021					1.79	536	0.807	433
6	3.50	40.25	72.7	72.7	72.9	72.8	101.5	101.3	101.5	101.4	1180.1	2.008					2.40	498	0.801	399
												2.010	2.490	19.27	24.08	20.0	2.03			411
7	4.00	46.00	72.1	72.4	72.5	72.3	101.3	101.4	101.6	101.4	1178.3	2.017					1.66	548	0.811	444
8	4.00	46.00	72.8	72.8	73.2	72.9	101.5	101.3	101.4	101.4	1183.1	2.010					2.37	560	0.799	447
9	4.00	46.00	72.7	72.5	72.9	72.7	101.3	101.3	101.4	101.3	1179.4	2.013					1,79	527	0,803	423
												2,013	2,473	18,61	24,34	23,5	1,94			438
10	4.50	52.00	72.3	72.5	72.5	72.4	101.5	101.5	101.5	101.5	1189,8	2,031					2,11	451	0,809	365
11	4.50	52.00	72.4	72.3	72.3	72.3	101.4	101.5	101.5	101.5	1190.8	2.037					2.88	502	0.811	407
12	4.50	52.00	72.7	72.5	72.7	72.6	101.5	101.5	101.3	101.4	1191.6	2.031					1.89	483	0.805	389
												2.033	2.457	17.26	23.96	28.0	2.29			387
13	5.00	57.50	72.9	73.0	73.1	73.0	101.5	101.4	101.5	101.5	1195.6	2.026					1.82	450	0.797	359
14	5.00	57.50	72.8	72.7	73.0	72.8	101.4	101.5	101.5	101.5	1196.4	2.032					1.68	471	0.801	377
15	5.00	57.50	72.2	72.1	72.1	72.1	101.5	101.4	101.5	101.5	1198.6	2.056					1.92	446	0.815	363
												2.038	2.441	16.51	24.12	31.6	1.81			366

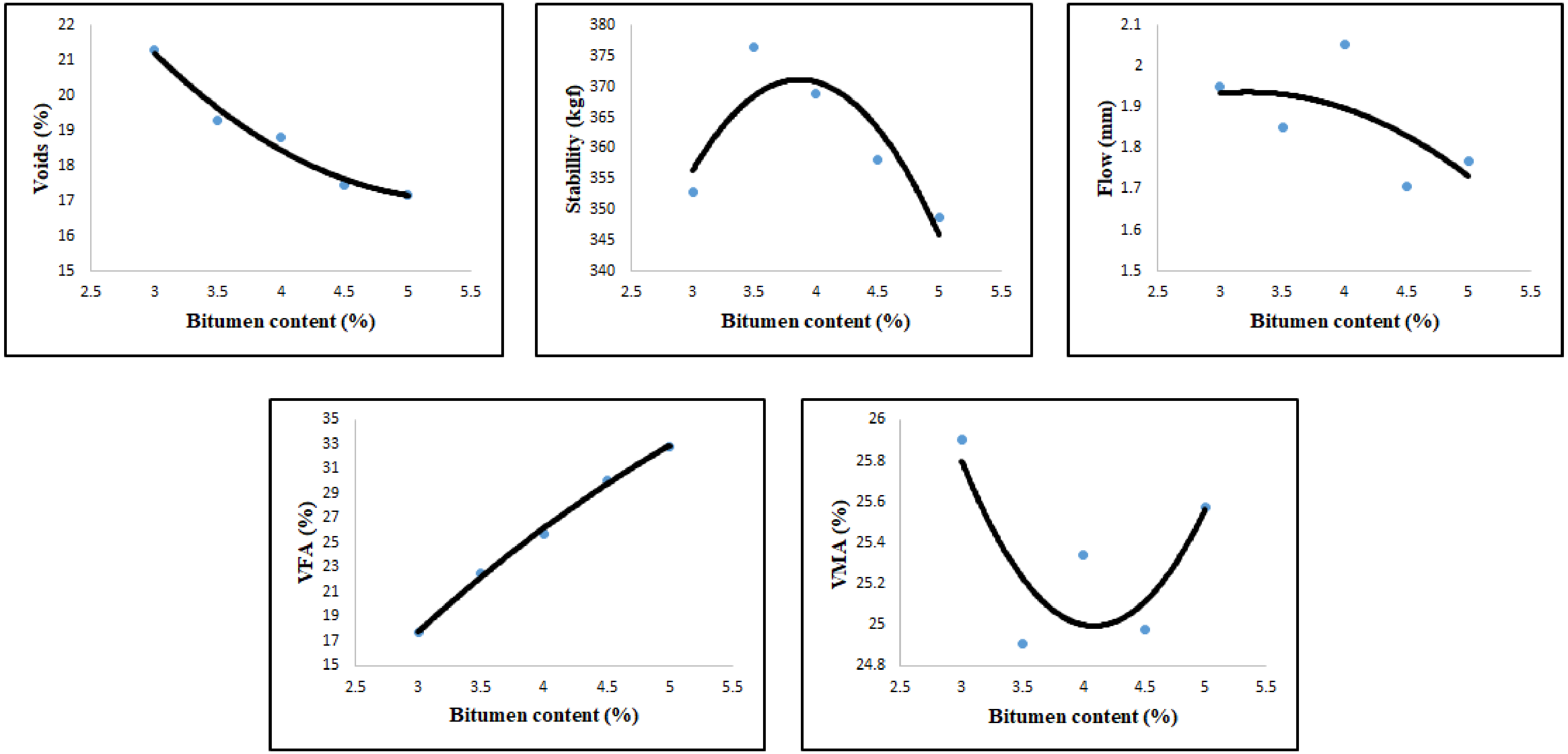

Figure A.4 Marshall mix design graphics for unmodified bitumen with basalt-limestone aggregate combination

Table A.5 Marshall mix design data for SBS PMB with basalt-limestone aggregate combination

Specimen No	Bitumen		Specimen height (mm)				Specimen diameter (mm)				Weight in air	Bulk Specific Gravity	Max. Teo. Specific Gravity	Voids	V.M.A	V.F.A	Flow	Stability	Correlation Fact.	Corr. Stability
#	Wa,%	g	1	2	3	Avg.	1	2	3	Avg.	gr	Dp	Dt	Vh	%	%	mm	kgf		kgf
1	3.00	34.50	73.7	73.4	73.6	73.6	101.3	101.3	101.4	101.3	1168.6	1.971					2.26	556	0.785	436
2	3.00	34.50	72.3	72.4	72.5	72.4	101.4	101.3	101.4	101.4	1177.5	2.016					2.45	581	0.809	470
3	3.00	34.50	73.0	73.5	71.8	72.8	101.3	101.3	101.4	101.3	1156.7	1.972					1.89	467	0.801	374
												1.986	2.507	20.78	24.63	15.7	2.20			427
4	3.50	40.25	73.2	73.4	73.2	73.3	101.3	101.4	101.4	101.4	1177.4	1.991					3.18	570	0.791	451
5	3.50	40.25	72.2	72.6	72.1	72.3	101.4	101.3	101.3	101.3	1178.7	2.023					3.09	599	0.811	486
6	3.50	40.25	73.0	73.1	72.8	73.0	101.4	101.3	101.3	101.3	1179.6	2.006					3.44	566	0.797	451
												2.007	2.490	19.41	24.22	19.9	3.23			463
7	4.00	46.00	73.0	73.1	73.1	73.1	101.3	101.4	101.6	101.4	1180.2	2.000					3.00	574	0.795	456
8	4.00	46.00	72.3	72.1	72.2	72.2	101.5	101.3	101.4	101.4	1181.4	2.027					2.34	636	0.813	517
9	4.00	46.00	73.8	73.1	73.4	73.4	101.3	101.3	101.4	101.3	1177.6	1.989					2.67	601	0.789	474
												2.005	2.473	18.92	24.63	23.2	2.67			483
10	4.50	52.00	73.3	72.6	72.8	72.9	101.4	101.4	101.3	101.4	1184.2	2.014					4.26	612	0.799	489
11	4.50	52.00	73.0	72.2	72.3	72.5	101.3	101.4	101.3	101.3	1186.2	2.030					5.13	595	0.807	480
12	4.50	52.00	72.9	72.9	72.8	72.8	101.3	101.3	101.3	101.3	1195.9	2.038					3.71	587	0.801	470
												2.027	2.457	17.49	24.17	27.6	4.37			480
13	5.00	57.50	72.2	71.6	72.1	72.0	101.4	101.4	101.3	101.4	1185.7	2.043					3.46	519	0.817	424
14	5.00	57.50	72.0	72.1	72.0	72.0	101.3	101.3	101.3	101.3	1191.3	2.053					3.61	634	0.817	518
15	5.00	57.50	72.6	72.4	72.6	72.5	101.4	101.3	101.3	101.3	1190.7	2.037					3.38	538	0.807	434
												2.044	2.441	16.27	23.91	31.9	3.48			459

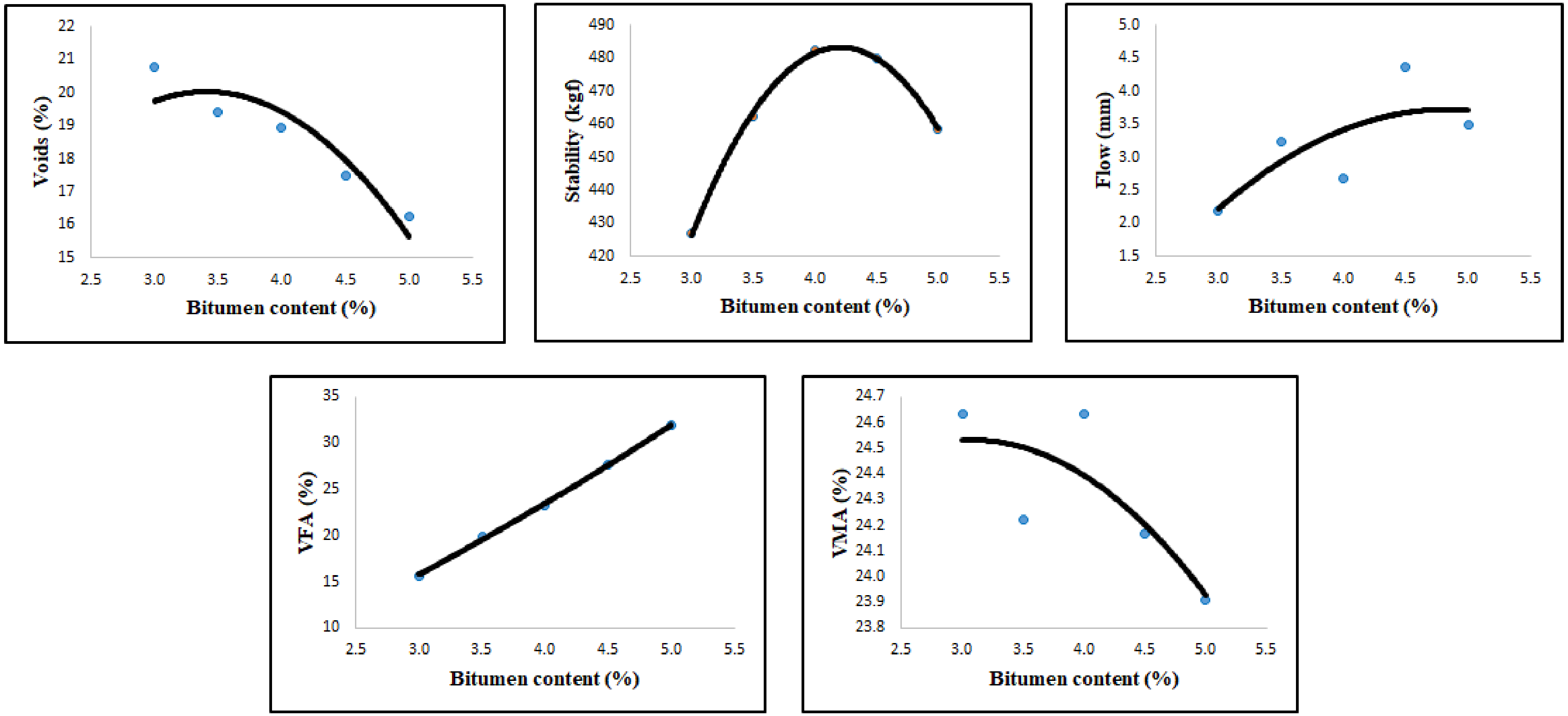

Figure A.5 Marshall mix design graphics for SBS PMB with basalt-limestone aggregate combination

Table A.6 Marshall mix design data for Elvaloy PMB with basalt-limestone aggregate combination

Specimen No	Bitumen		Specimen height (mm)				Specimen diameter (mm)				Weight in air	Bulk Specific Gravity	Max. Teo. Specific Gravity	Voids	V.M.A	V.F.A	Flow	Stability	Correlation Fact.	Corr. Stability
#	Wa,%	g	1	2	3	Avg.	1	2	3	Avg.	gr	Dp	Dt	Vh	%	%	mm	kgf		kgf
1	3.00	34.50	74.7	74.5	74.8	74.7	101.3	101.4	101.4	101.4	1177.5	1.955					3.13	586	0.763	447
2	3.00	34.50	73.2	73.0	73.1	73.1	101.3	101.3	101.4	101.3	1174.3	1.993					3.43	647	0.795	514
3	3.00	34.50	74.1	74.2	74.5	74.3	101.3	101.3	101.5	101.4	1178.2	1.967					3.28	625	0.771	482
												1.972	2.507	21.35	25.18	15.2	3.28			481
4	3.50	40.25	73.0	73.1	73.0	73.0	101.4	101.3	101.3	101.3	1183.1	2.009					4.16	632	0.797	504
5	3.50	40.25	73.1	72.9	73.0	73.0	101.4	101.4	101.3	101.4	1185.2	2.013					3.32	626	0.797	499
6	3.50	40.25	73.8	74.0	73.7	73.8	101.3	101.4	101.3	101.3	1186.3	1.994					4.15	674	0.781	526
												2.005	2.490	19.46	24.27	19.8	3.88			510
7	4.00	46.00	72.2	71.7	72.1	72.0	101.3	101.3	101.4	101.3	1186.1	2.043					3.86	640	0.817	523
8	4.00	46.00	73.7	73.5	73.4	73.5	101.3	101.4	101.3	101.3	1190.0	2.008					3.52	739	0.807	596
9	4.00	46.00	73.6	74.0	73.2	73.6	101.4	101.4	101.4	101.4	1188.4	2.001					4.13	621	0.785	487
												2.017	2.473	18.44	24.18	23.8	3.84			536
10	4.50	52.00	73.7	73.6	73.9	73.7	101.3	101.3	101.3	101.3	1188.6	2.001					4.52	669	0.783	524
11	4.50	52.00	73.1	73.3	73.4	73.3	101.3	101.3	101.4	101.3	1190.4	2.015					2.93	495	0.811	401
12	4.50	52.00	73.4	73.4	73.2	73.3	101.4	101.4	101.3	101.4	1186.2	2.005					4.58	586	0.791	464
												2.007	2.457	18.31	24.92	26.5	4.01			463
13	5.00	57.50	73.4	73.0	73.1	73.2	101.3	101.3	101.4	101.3	1198.1	2.032					3.40	465	0.793	369
14	5.00	57.50	71.9	71.8	71.8	71.8	101.3	101.4	101.4	101.4	1197.2	2.067					4.13	667	0.821	548
15	5.00	57.50	72.2	72.8	72.4	72.5	101.4	101.4	101.3	101.4	1190.9	2.038					3.78	488	0.807	394
												2.045	2.441	16.22	23.86	32.0	3.77			437

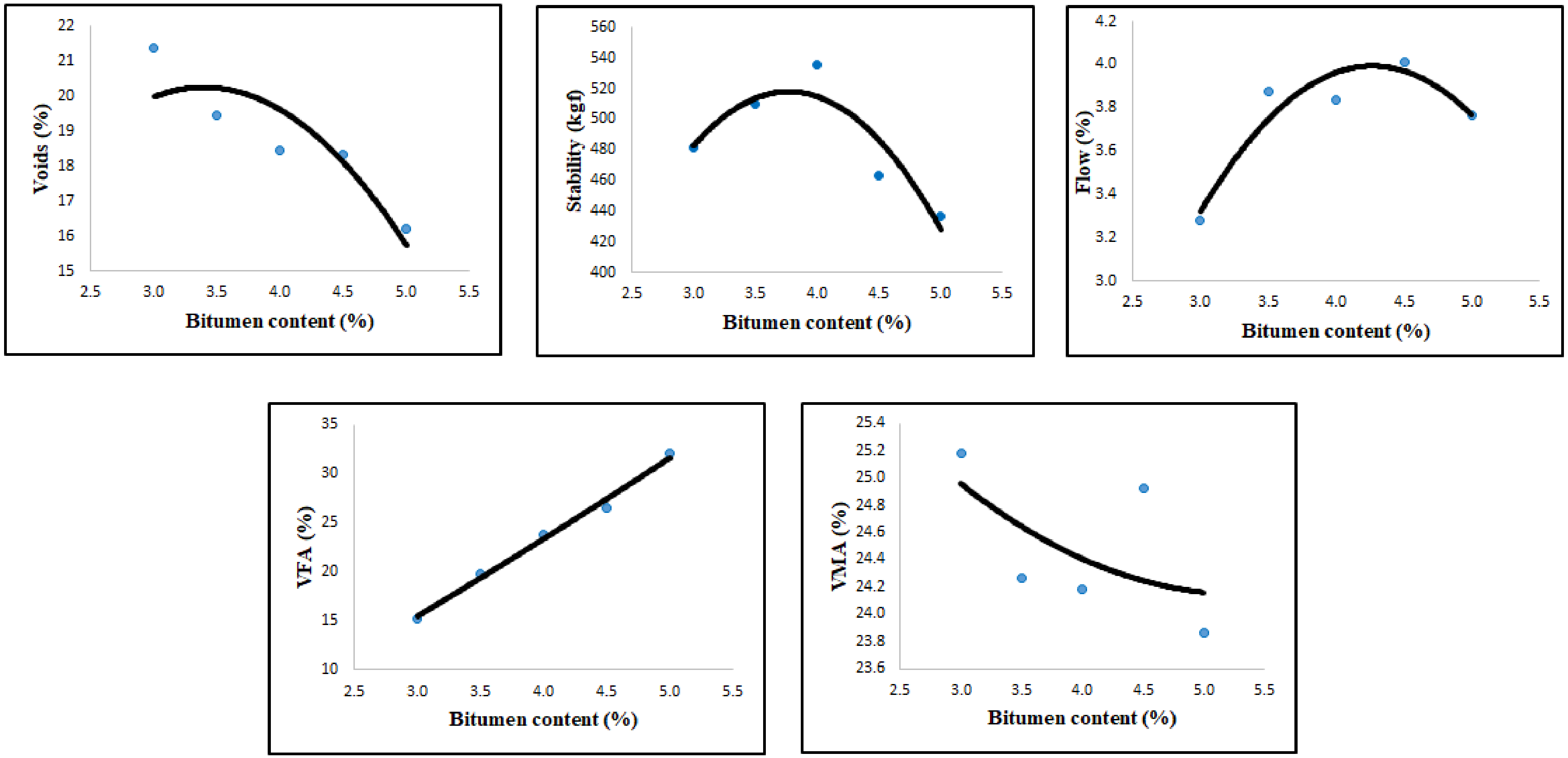

Figure A.6 Marshall mix design graphics for Elvaloy PMB with basalt-limestone aggregate combination