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HIGH PERFORMANCE POROUS ASPHALT MIXTURES AS ALTERNATIVE MATERIALS FOR ROAD PAVEMENT WEARING COURSE

REPORT-1

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By

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1.0 INTRODUCTION

Porous asphalt is an innovative road surfacing technology, which allows water to seep into the asphalt mix through its continuos air voids. Porous asphalt is normally used as wearing course material and always laid on an impervious base course. It can be effective in enhancing traffic safety particularly during rainy weather, reducing hydroplaning potential and having good skid resistance properties at high speed. The use of porous asphalt also reduces traffic noises and glare at night and on wet surfacings. In addition porous asphalt exhibits superior resistance against permanent deformation.

Nevertheless, porous asphalt is not a panacea. Field observations in Japan and elsewhere indicate problems particularly related to clogging and disintegration. Probably the main problem is loss of permeability due to clogging by road dust and detritus, tyre wear by-products, etc.

The search for a solution to mitigate clogging has led to the development of twinlay. Twinlay is a double layered construction of porous asphalt developed in the Netherlands and was first reported at the Eurasphalt and Eurobitume Congress in 1996 held at Strasbourg. From the perspective of resistance to clogging, the two-layered porous asphalt construction has the potential to better resist clogging compared to conventional single layer porous asphalt. Several roads in Netherlands are paved with this type of construction. The two-layered concept proved to be effective in reducing traffic noise. Additional benefits over and above those from conventional porous asphalt layer were also observed. This includes the water discharge capacity and the ease of cleaning maintenance.

However, in this investigation, the optimum relative thickness for porous asphalt layer will be optimised based on permeability, Marshall stability, clogging, disintegration, creep, wheel tracking test and overcompaction.

This Milestone Report I presents the results of new gradations designed for porous asphalt in a variety of maximum aggregate sizes, namely 20, 14 and 10 mm; by applying the theory of packing. Almost all porous asphalt gradation used worldwide are developed based on empirical methods and do not consider the packing behavior of aggregate mass.

2.0 SCOPE OF WORK

The scope of work during this stage of the research focus on the development of porous asphalt aggregate gradation with varying maximum aggregate sizes by determining the proportion of aggregate fractions that gives rise to minimum dry aggregate porosity using a dry aggregate vibratory compactor. Modification from this minimum void gradation will be made if it does not meet the target mix porosity and permeability.

3.0 OBJECTIVES

The main objective of this stage of the research is to develop suitable combinations of aggregate gradation for double layer porous asphalt. In principle, the upper layer which is made up of fine porous asphalt overlies a coarser bottom layer of porous asphalt. Both individual layers should exhibit favorable mix properties particularly in terms of permeability and strength.

4.0 AGGREGATE GRADING DESIGN BY PACKING METHODS

Packing implies the arrangement of the particles which fit together to fill voids. The shape of particles, which in turn influence porosity of the mix, influences the mode of packing. Gradation of aggregate is also an important factor, which influences the void content in the mixture.

Many experimental studies have been made, with mixtures of two or more particle sizes to obtain minimum void, mainly by using smaller particles. Aljarallah and Tons (1980), studied the void content in two-size aggregate component system. He produced curves showing minimum voids in the system against percentage of the larger size fraction for a number of size ratios. Abdullah W.S et. al (1998), concluded that using design mixtures by Lees methods for dense unbound aggregate mixtures produced the densest mixtures for all five selected gradations. Lees has simplified the difficult task of developing a method of aggregate grading, taking into consideration factors influencing packing of aggregate, namely, variation in the aggregate's shape, size and texture, compactive effort, and boundary conditions, by separating the aggregates into nearly'single-sized' or nearly 'one-sized' fraction or component, Abdel-Jawad and Abdullah (2002).

A porous asphalt aggregate grading consists predominantly of coarse aggregate. The fine aggregate fractions are added so as not to bulk or interfere with the interlock of the coarse aggregate matrix but to leave enough voids to maintain a pervious structure. Aljarallah and Tons (1980), developed the packing volume concept (Vp) which defined packing volume as the volume which a rock particle occupies in mass of one-size (mono-volume) particles. The packing volume encompasses not only the surface capillaries (micro surface voids) but also the volume of surface macro dips and valleys (macro surface voids).

5.0 POROUS ASPHALT

Porous asphalt inhibits ponding water on road surfacings. Therefore, porous asphalt surface courses can eliminate aquaplaning potential, reduces splash and spray apart from reduce glare. The consequence is enhanced driving comfort. Various terminology has been coined to designate porous asphalt such as pervious macadams, whispering asphalt or political asphalt. In Korea, the term ECOPHALT is used, probably to register the positive role played by porous asphalt for the environment. However, porous asphalt appears to be the most internationally accepted terminology and will be used in this research.

5.1 Double Layer (Twinlay) Porous Asphalt (PA)

The two-layered concept proved to be effective not only in reducing traffic noise, but also additional benefits over and above those of conventional PA layers. Double-layer PA has a large drainage capacity, not so quickly contaminated as single layer PA and if clogged double layer PA are relatively easier to clean (Van Buchove, 1996).

In the Netherlands, the top layer of PA 2/4 is a refined version of the standard twinlay. The percentage of binding agent a modified binder in the top layer is somewhat higher than for normal twinlay. The base layer of twinlay M consists of PA 11/16 and has a thickness of 4.5 cm, similar to ordinary twinlay.

Next, twinlay has in comparison with conventional dense asphalt concrete, the following advantages:

1. The fine top layer offers acoustic advantages through the fine surface texture. The coarse bottom later provides in combination with the fine top layer also required damping of sound.
2. The top layer prevents coarse dirt or temporary a large a mount of dirt from entering in to the construction.
3. Dirt, which nevertheless penetrates in the wearing course, can be removed with less effort. The dirt is absorbed at the top of the thin top layer, from where it can easily be removed with existing cleaning techniques.
4. The difference in airflow resistance between the top and bottom layer has a positive effect on the self-cleaning capacity caused by traffic.
5. The bottom layer is more permeable compared to conventional porous asphalt through which the sideways discharge of water improves considerably.

5.2 Properties of Porous Asphalt

Among of common measured properties of porous asphalt properties include voids in mix, Marshall stability and permeability. These properties are influenced by gradation; maximum aggregate particle and bitumen content.

1. Porosity

   To a large extent, porosity or void content determines the mix permeability. In porous asphalt, the quantity and gradation of coarse aggregate determines porosity, hence permeability. In general, porosity can be increased by increasing the proportion of the coarse mineral aggregate and reducing the amount of fine aggregate fraction, Cabrera and Hamzah (1996). Abdullah W.S, et. al (1998) concluded that increasing void in the mineral aggregates and air voids in the mix by choosing the coarse aggregate gradation for the asphalt mix makes it porous and water permeable.

2. Permeability

   Permeability is an important engineering property of a bituminous mix. It indicates the degree of void interconnection to form capillary channels that allows the passage of a permeant.

   The coefficients of permeability (k), in units of cm/sec, reported by a number of researchers differ greatly. Average k values of 0.16-0.41 cm/sec. Permeability loss is imminent when voids close up. Increasing the binder content reduces permeability since the excess binder replaced the air voids.

3. Marshall Stability

   In porous asphalt, the source of stability is from aggregate interlock enhanced by the coarse aggregate. Generally, Marshall stability values for porous asphalt are significantly lower than that of dense asphalt. Marshall stability increases as the gradation becomes less open graded by incorporating more fines.

6.0 MATERIALS AND THEIR PROPERTIES

6.1 Aggregates

Crushed aggregates supplied by Kuad Quarry Sdn. Bhd. were used in this investigation. Coarse aggregate is defined as material retained on the 3.35-mm. The aggregates were sieved in to their respective size range or fractions. Their respective specific gravity and water absorption for each fraction is shown in Table 1. The other properties of aggregate fractions used is shown in Table 2.

6.2 Filler

Filler is basically material passing the 75 micron sieve. The aggregate gradation indicates a 4.0% filler requirement. It is customary to use hydrated lime as filler in porous mixtures to resist stripping but in quantity not exceeding 2.0%. Hence, the additional 2.0% comprised of Ordinary Portland Cement (OPC). The specific gravity for hydrated lime and OPC is respectively 2.406 and 3.16.

6.3 Binder

Bitumen penetration grade 60/70 supplied by Shell Malaysia was used as the binder. The bitumen properties is shown in Table 3.

7.0 METHODOLOGY

Selection of aggregate gradation corresponding to 20, 14 and 10 mm maximum size involves three stages, namely:

1. Determination of material properties including aggregates, binder and filler
2. Determination of dry porosity of dry aggregate fractions

3. Determination of porous asphalt mixture properties which includes mixture volumetric properties, permeability and Marshall stability.

Twinlay is made up of the top and base layer. The maximum aggregate size for the base layer is 20 mm while two maximum aggregate sizes 14 and 10 mm will be used for the top layer.

7.1 Dry Aggregate Compaction

To assess the packing behavior of the aggregates used, a vibratory mode of compaction was utilized. A schematic sketch of the vibratory compactor used is shown in Figure 1. The vibratory compactor consists of a mould secured to a vibrating table. A 4-kg steel cylinder surcharge was placed on top of the sample to achieve a uniform compacted surface. The optimum time and frequency has to be determined prior to using this equipment.

The laboratory process consisted of weighing the blend of fractions A plus xB (x varied in steps of 10%) aggregate fraction, mixing the aggregates in a bowl and pouring them into the mould from a constant height. After placing the surcharge on top of the aggregate specimen in mould, the compactor was switched on for optimum time and frequency of vibrating, after which the heights of the sample at three equally spaced points to the nearest 0.01 were recorded. The results used for the design are the average of two tests. The weight of aggregate used in this investigation was 1000 gram. The proportion of coarser fraction A and finer fraction B at the minimum porosity is described as the balanced proportion. For the next step, blend (A.B) then becomes a new coarser componen into which the next finer componen C is added incrementally to result in a new minimum porosity mix (AB.C). The procedure was repeated by adding smaller fraction size D to coarser blend (AB.C).

Mix (ABC.D) represents the blend that gives the minimum porosity of the (ABC.D) aggregate matrix. If this matrix is then consider to be stable matrix which will provide strength and resistance to deformation, the next step is to vary the fine aggregate grading in order to achieve a target porosity that is considered suitable for pervious mix. The final fine aggregate size E need to be combined with mix (ABC.D) each starting with 5%, 10%, 15% and 20% of fraction E. Knowing the target porosity, the various aggregate

grading could be conceived. The method enables the design of grading with any desired porosity when the porosity of the coarse aggregate matrix is at its minimum.
The porosity P was determined from equation 1.

$$P = 100\,((1-(D/Dr)) \quad \ldots\ldots \text{Equation (1)}$$

Where,

P	: Porosity (%)
D	: Compacted density of dry aggregate
Dr	: Relative density of mixed aggregate (gr/cm^3)

Dr is obtained from equation 2.

$$Drn = 100/(\Sigma Pwi/Dri) \quad \ldots\ldots \text{Equation (2)}$$

Where Drn	: Relative density of the mixture of n aggregates
Pwi	: Percentages of aggregate from fraction i
Dri	: Relative density of aggregates from fraction I

7.2 Optimum Time and Frequency for Vibration

The time and frequency of vibrating influence dry aggregate porosity. When the vibrating equipment is vibrated at higher frequency, the maximum density will be achieved over shorter time duration. To determine the optimum time and frequency, trial runs on selected aggregate blends were carried out corresponding to frequencies 25, 30, 40 and 45 Hz and duration 25, 30, 45 60, 90, 120 and 180 s. The resultant density values for a given frequency were plotted against duration.

7.3 Design Gradation

To get a gradation that fulfills the requirements for porosity, stability and permeability; Marshall specimen mixes were prepared in accordance to a range of dry aggregate void for all maximum aggregate size selected. These mixtures were prepared with various fine aggregates contents combinations. The maximum proportion of fine aggregate including filler should not exceed 15%.

In case the mixtures do not satisfy the criteria for permeability, porosity, and stability, then the gradation may need modification. Higher porosities were possible by reducing aggregate fraction D. Fraction D was reduced to create more voids that can be filled up by the finer fractions.

7.4 Specimens Preparation and Test on Asphalt Mixtures

7.4.1 Experimental design

Marshall specimens were prepared after having determined the appropriate aggregate grading in a range of dry aggregate porosity. A 4.5 % and 4.0 binder and filler contents were chosen for all mixtures respectively. Aggregate was mixed with binder at 140°C and compacted 130°C. Specimens was compacted at 50 blows per face

The mixer was first calibrated and then set to the required mixing temperature. The mixing process started with firstly pouring the hot aggregates into the mixer and then mixed dry for 1/2 minute. This step ensured uniformity and homogeneity of the aggregate blend. The correct quantity of bitumen was added into the blended aggregates and mixing continued. In all experiment work, the amount of bitumen required was calculated as a percentage of the total mix. The mixing process ceased as soon as all of the aggregate was thoroughly coated with bitumen, which normally required 1 minute of mixing time. About 1100 g of the mix was then transferred to the Marshall mould. Using a metallic rod, the sample was tamped 15 times around the circumference and 5 times in the center to ensure uniformity of mix. The mix temperature was then monitored. Once the temperature of the unsegregated mix had dropped uniformly to the required compaction temperature, the partially compacted sample was then ready full compaction.

7.4.2 Permeability

The coefficient of permeability is an important property of porous asphalt. Consequently, its measurement constituted a major part of the study. In this report, the term permeability will be taken to mean the coefficient of permeability. Currently, there is no standard procedure for measuring permeability of porous asphalt. The equipment used for determining permeability in this investigation is a specially designed water permeameter, which was designed based on Leeds Water Permeameter. This equipment was designed to measure the permeability of Marshall specimens but can be adapted to measure the permeability of slabs for the wheel tracking.

Figure 2 shows the water permeameter. The design was based on the principle of the falling head permeameter, used to quantify the coefficient of permeability of fine-grained soils. In general, the permeability was measured by creating a hydraulic gradient across the specimen and then measuring the water flow over a period of time.

After compaction, the specimens were cooled in their mould. The specimens were then tested for permeability before extrusion to take advantage of the tight bond between the bituminous mix and the mould. The reported permeability coefficient was the average of three results.

The coefficient of permeability of the specimen k was computed from Equation 3.

$$k = 2.3\ [a.L/At]\ [\log (h1/h2)] \qquad \text{Equation (3)}$$

Where: k = coefficient of permeability (cm/s)

a = the cross sectional area standpipe (cm2)

h1, h2 = water in the standpipe fall from h1 to h2 (cm)

A = cross sectional area specimens (cm2)

L = height of specimens (cm)

7.4.3 Marshall Stability

Conventionally, the Marshall test is simply a method of measuring stability and flow. A number of mix design methods based on the test have been developed for dense mixes. Nevertheless, it was used as a rough guide to rank mixes in terms of strength and to assess the relative behavior of the porous mixes investigated.

In the stability test, the specimen was placed between two split breaking heads in an unconfined manner. Load was applied in a direction perpendicular to the longitudinal axis of the specimen at a constant rate of strain 50.8 mm/minute until failure. The specimens were normally tested at 60 °C for 30 minutes. The maximum load developed during the test is known as the stability in kN and the amount of deformation that took place up to the moment of failure is term as the flow in mm. The reported stability are the average of three readings.

8.0 RESULTS AND DISCUSSIONS

8.1 Optimum Time and Frequency of Vibration

From trial runs carried out to determine the optimum time and frequency of vibrating compactor, the relationships between density, frequency and time are plotted in Figures 3 and 4. From Figure 3, when vibrated at 40 Hz, the optimum duration is 60 s. At 35Hz and 30 Hz, the optimum duration are respectively 70 s and 80 s. The highest compacted dry aggregate density when vibrated over a duration ranging from 25 s to 180 s is achieved at 40 Hz frequency, as shown in Figure 4. Beyond frequency 45 Hz, the aggregate density reduces due to excessive vibration. The optimum time and frequency is 65 second and 40 Hz respectively and are the values adopted in the study of aggregate packing.

8.2 The Minimum Void of Coarse Aggregate

Table 4 shows the results of vibratory compaction of coarse aggregate for maximum aggregate sizes 20, 14 and 10. The dry aggregate porosity values for these maximum aggregate sizes are plotted as curves, as shown in Figures 5, 6, 7 and 8.

8.3 Permeability and Marshall Stability

The experimental results of Marshall Stability and permeability for mixtures made of maximum aggregate sizes 20, 14 and 10 mm at range dry aggregate porosity are as shown in Table 5. Figure 9 shows the relationship between permeability, stability, and total fine aggregate of mixtures at minimum dry porosity of coarse aggregates and almost all mixtures exhibit porosities above 20%. Some mixtures made of maximum aggregate size 14 and 10-mm were found to fulfill the requirement of permeability and stability. Mixtures made of maximum aggregate size 20 mm resulted in higher permeability, but lower stability. Thus, the gradation of this mixture was modified as shown in Figure 8 and the result of stability and permeability for this mixture is as shown in Table 6. The average values of porosity, coefficients of permeability and Marshall stability for selected gradation are shown in Table 7. From Table 7, it can be seen that all mixtures can fulfill the requirements of porosity, permeability and Marshall stability. The selected gradation can be seen in Figure 10.

8.4 Aggregate Grading Design

8.4.1 Aggregate Gradation for Top Layer

The porosity values of compacted dry aggregates for coarse aggregates used in this investigation are tabulated in Table 4. The dry aggregate porosity values resulting after mixing and compacting various proportions of aggregates from fractions B and C for the top layer was plotted as a curve. Minimum porosity is achieved when B and C are blended in the ratio of 50:50, in which the minimum porosity was found 41%. Combining fraction D aggregates with the optimum of B + C (50:50) resulted in the minimum void of three-componen system and was found to occur at 43% D. This blend gives the most stable coarse aggregate matrix. Therefore, adjusting the proportion to express this blend as a combination of B:C:D gives 28.5%:28.5%:43%. The results of investigation show that the minimum porosity of a system composed of several components is always smaller than that of a single component.

Using fractions of various fine aggregates in mixtures made of maximum aggregate size 14 mm resulted in a decrease in the permeability coefficient when the amount of fine aggregate was increased. The mixtures made of fine aggregate 10.5-15% resulted in the permeability 0.115 cm/second – 0.192 cm/second and stability 3.7 kN – 6.1 kN. Based on the relationship between permeability, stability and various fine aggregates as shown Figure 9 it was found that the amount of fine aggregate is 10.5%. At this condition, the values of permeability and stability were found to be 0.165 cm/second and 5.2 KN respectively.

The procedure of combining between fractions C and D for maximum aggregate size 10 mm is similar with procedure for the combination of fractions B and C of maximum aggregate size 14. The optimum composition C and D for achieving the minimum porosity is at comparison 46%: 54%.

The use of various fine aggregates to mixtures made of maximum aggregate size 10 mm resulted in mix the porosity above 20%. The porosity of mixtures made of maximum aggregate size 10 mm is higher than that made of maximum size aggregate 14 mm. This indicates that the smaller diameter particle has more surface area of aggregate for bitumen coverage. The amount of bitumen used in investigation is constant at 4.5% by total mix.

The mixtures were made of fine aggregate in the range of 10.5%-14%. The maximum aggregate size 14 mm resulted in the Marshall stability ranging from 2.5 kN to 5.7 kN and coefficient of permeability 0.082-0.219 cm/s. The highest permeability was achieved with the fine aggregate of 10.5%, but with lowest stability of 2.5 kN. As shown in figure 9 the amount of fine aggregate was found as 11.5%. At this condition, the values of permeability and stability are 0.15 cm/second and 5.4 kN respectively.

8.4.2 Aggregate Gradation for Base Layer

The porosity of mixtures made of maximum aggregate size 20 mm was found to be above 20%. The fine aggregate used was in range of 9.0 to 15.5% resulting in the permeability of 0.069 to 0.193 cm/s and Marshall stability between 3.0 kN to 5.9 kN. Based on this result, it can be seen that the mixture at maximum permeability 0.193 cm/second resulted in the lowest stability of 3 kN. Marshall stability and permeability was achieved after modified gradation with the fraction D 15-26% and fine aggregate 10%. However, the mixture made of fraction D 19% gave the permeability 0.19 cm/second and Marshall stability 5.4 kN. The permeability coefficient for mixture made with fraction D 19% is higher than the mixture made of fraction D 23%. On the contrary for stability, the mixture made of fraction D 23% is higher than the mixture made of fraction D 19%. Thus, mixture made of lower fraction D was found to give improved permeability. However, the values of permeability have to be controlled by Marshall stability value. In over all for base layer, the modified gradation met the requirement of porosity, permeability and Marshall stability.

9.0 CONCLUSIONS

1. The addition of fine aggregate to coarse aggregate matrix at minimum porosity results in mixtures that do not fulfill the minimum requirement for permeability for some porous aggregate gradations. In this investigation, the aggregate grading consisting of 20 mm maximum aggregate size, requires modification.
2. Blending 19% fraction D and 10% fine aggregate results in mixtures that fulfill the minimal requirement of permeability for mixtures made from maximum aggregate size 20 mm.

3. Blending fine aggregate in quantities of 12.7% and 11.5% respectively in mixtures consisting of maximum aggregate sizes 14 and 10mm, results in mix permeability 0.15 cm/sec and mix stability 6.0 kN and 5.4 k N respectively.

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APPENDIX

Table 1 Aggregate Properties Used in This Investigation

Aggregate size (mm)	Fractions (%)	Oven –Dried Rel. Density	Surface–Dried Rel. Density	Apparent Rel. Density	Water Abs. (%)
20-14	A	2.619	2.628	2.643	0.339
14-10	B	2.630	2.640	2.657	0.392
10-5	C	2.641	2.654	2.676	0.494
5-3.35	D	2.652	2.666	2.689	0.523
3.35-0.425	E	2.658	2.674	2.701	0.603
0.425-0.075	F	2.673	2.691	2.722	0.674

Table 2 Flakiness and Elongation Index Test Results

Type of Tests	Results
Flakiness Index	21%
Elongation Index	27 %
PSV	52.7%
ACV	26.05%
LAAV	9.81%

Table 3 Properties of Bitumen Penetration Grade 60/70

Type of Tests	Result
Specific Gravity	1.03
Penetration (25 °C, 5 sec.)(0.1 mm)	62.5
Softening point (°C)	49.5
Ductility (25 °C, 5 cm/sec.)(cm)	> 100

Table 4 Dry Aggregate Porosity for Maximum Size Aggregates 20, 14 and 10 mm

Aggregate	Fractions (%)				Total	Porosity
	A	B	C	D	(%)	(%)
1.Base layer - Max. size 20 mm	15.68	15.68	26.25	43	100	39.05
2. Top layer - Max. size 14 mm		28.5	28.5	43	100	38.30
- Max. sizes 10 mm			46	54	100	38.40

Table 5 Permeability, Marshall Stability, and Porosity Values of Mixtures at Minimum Porosity of Coarse Aggregate

A	B	C	D	Total	E	F	Filler	Total	Type	Por. (%)	Stab. (N)	Flow	Perm
Max. agg. Size 10 mm													
A	B	C	D	Total	E	F	Filler	Total	Type	Por. (%)	Stab. (N)	Flow	Perm
0	0	46	54	100	5	1.5	4	10.5	10MVmFa4.5A	27.8	2494	2.01	0.219
					7.5	0	4	11.5	10MVmFb4.5A	26.75	5450	2.88	0.15
					10	0	4	14	10MVmFc4.5A	23.56	5666	2	0.082
					0	7.5	4	11.5	10MVmFd4.5A	25.54	5037	2.35	0.115
					1.5	5	4	10.5	10MVmFe4.5A	25.04	5441	2.7	0.101
Max. agg. size. 14 mm													
A	B	C	D	Total	E	F	Filler	Total	Type	Por. (%)	Stab. (N)	Flow	Perm
0	28.5	28.5	43	100	1.5	5	4	10.5	14MVmFa4.5A	22	5184	2.28	0.132
					2	5.5	4	11.5	14MVmFb4.5A	21.36	5179	2.27	0.125
					5.5	2	4	11.5	14MVmFc4.5A	23	6089	3.32	0.15
					5	1.5	4	10.5	14MVmFd4.5A	25.26	3705	2.3	0.192
					5	2.7	4	11.7	14MVmFe4.5A	24.62	3749	2.03	0.172
					5	4	4	13	14MVmFf4.5A	24.04	5805	2.68	0.115
					7.5	2.2	4	13.7	14MVmFg4.5A	24.74	4239	1.95	0.134
					10	1	4	15	14MVmFh4.5A	24.11	5901	2.4	0.121
Max.agg. size 20 mm													
A	B	C	D	Total	E	F	Filler	Total	Type	Por. (%)	Stab. (N)	Flow	Perm
15.68	15.7	25.7	43	100	5	1.4	4	10.4	20MVmFa4.5A	27.8	3830	2.78	0.111
					5	2.6	4	11.6	20MVmFb4.5A	21.89	5179	2.43	0.086
					7.5	1.4	4	12.9	20MVmFc4.5A	22.68	5698	2.64	0.102
					7.5	2.6	4	14.1	20MVmFd4.5A	19.75	4860	3.7	0.091
					7.5	4	4	15.5	20MVmFe4.5A	20.41	5713	3.8	0.082
					10	1.4	4	15.4	20MVmFf4.5A	20.58	4509	3.8	0.095
					10	2.6	4	16.6	20MVmFg4.5A	19.83	4527	3.9	0.069
					7.5	0	4	11.5	20MVmFh4.5A	23.12	3181	3.4	0.153
					10	0	4	14	20MVmFi4.5A	22.03	4259	3.6	0.12
					0	5	4	9	20MVmFj4.5A	25.21	3041	3.5	0.193
					0	7.5	4	11.5	20MVmFk4.5A	19.99	5171	5.5	0.082
					0	10	4	14	20MVmFl4.5A	18.95	5863	4.7	0.063
					2	3	4	9	20MVmFm4.5A	23.11	5025	3.02	0.173

Permeability, in cm/second; and Flow in mm

Table 6 Permeability, Marshall Stability, and Porosity Values of Mixtures made of Modified Gradation

Max. agg. size 20 mm

A	B	C	D	Total	E	F	Filler	Total	Type	Por. (%)	Stab. (N)	Perm
23.89	23.89	33.21	19	100	4.5	1.5	4	10	20MMD1Fa4.5A	21.96	5298.9	0.187
					1.5	4.5	4	10	20MMD1Fb4.5A	22.35	4478.4	0.14
22.71	22.71	31.57	23	100	4.5	1.5	4	10	20MMD2Fa4.5A	20.69	5350.8	0.163
					1.5	4.5	4	10	20MMD2Fb4.5A	21.94	5211.2	0.151
22.12	22.12	30.75	25	100	4.5	1.5	4	10	20MMD3Fa4.5A	23.02	5569.5	0.132
					1.5	4.5	4	10	20MMD3Fb4.5A	22.42	5292.6	0.088

Permeability, in cm/second; and Flow in mm

Table 7 Average Values of Porosity, Permeability and Stability for Selected Gradations

Gradation	Result			Specification		
	Porosity (%)	Permeability (cm/second)	Stab. (kN)	Porosity (%)	Permeability (cm/second)	Stab. (kg)
• Base layer						
Max. Agg. 20 mm	21.96	0.190	5.3	> 20	0.10	> 500
• Top layer						
Max. Agg. 14 mm	23.0	0.165	5.2	> 20	0.10	> 500
Max. Agg. 10 mm	26.75	0.150	5.4	> 20	0.10	> 500

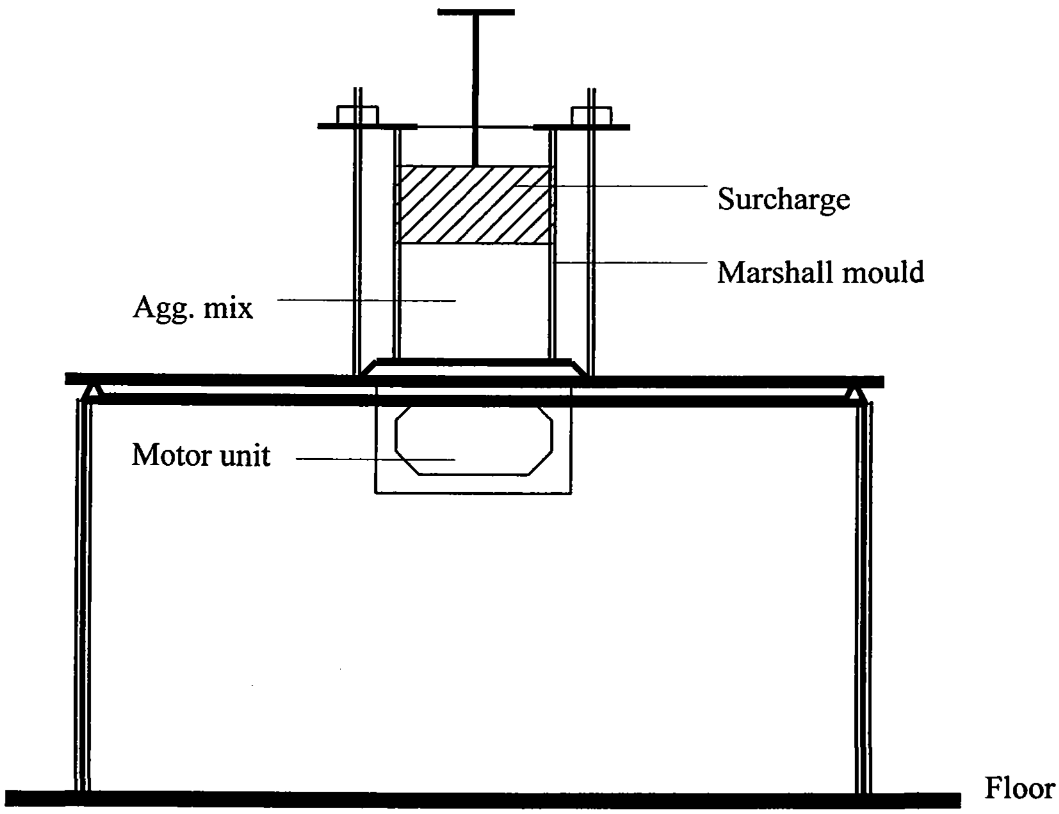

Figure 1 Schematic Sketch of the Vibratory Compactor

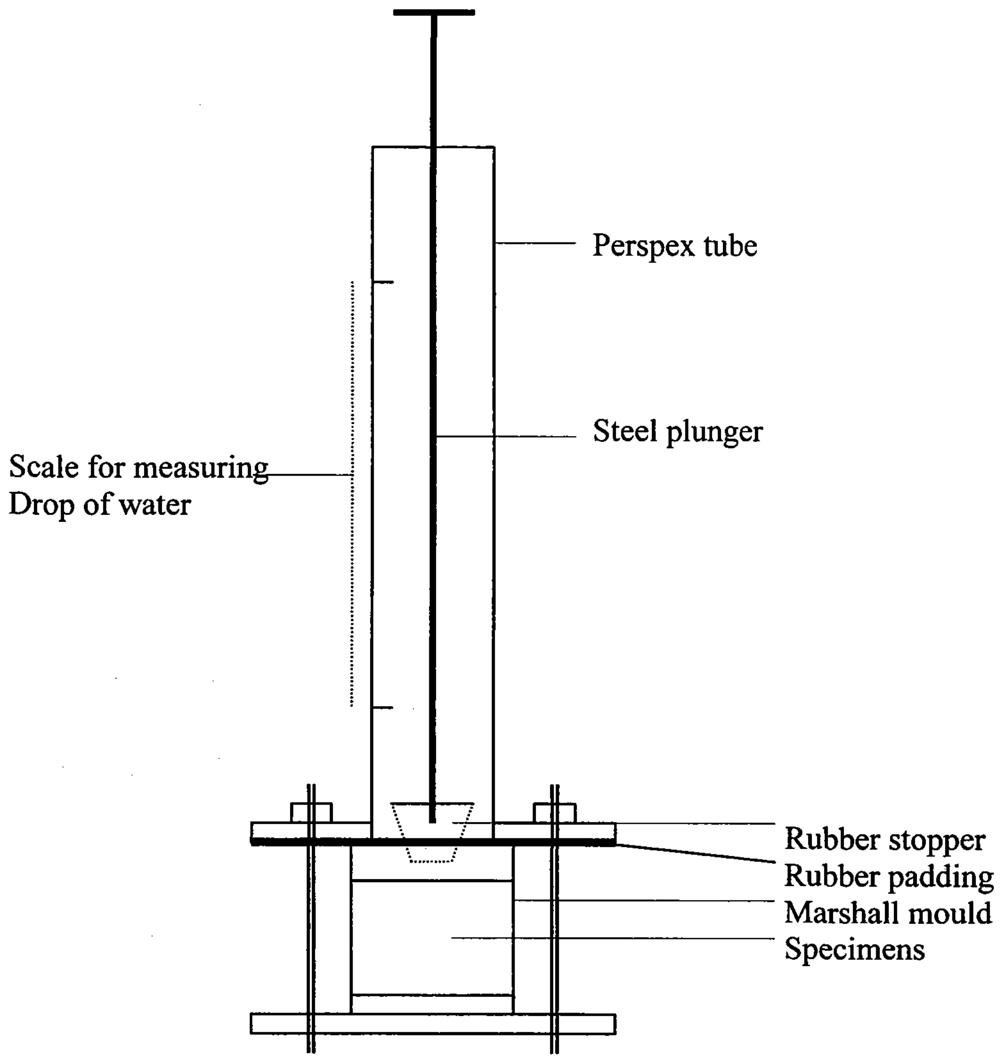

Figure 2 Schematic Sketch of Water Permeameter

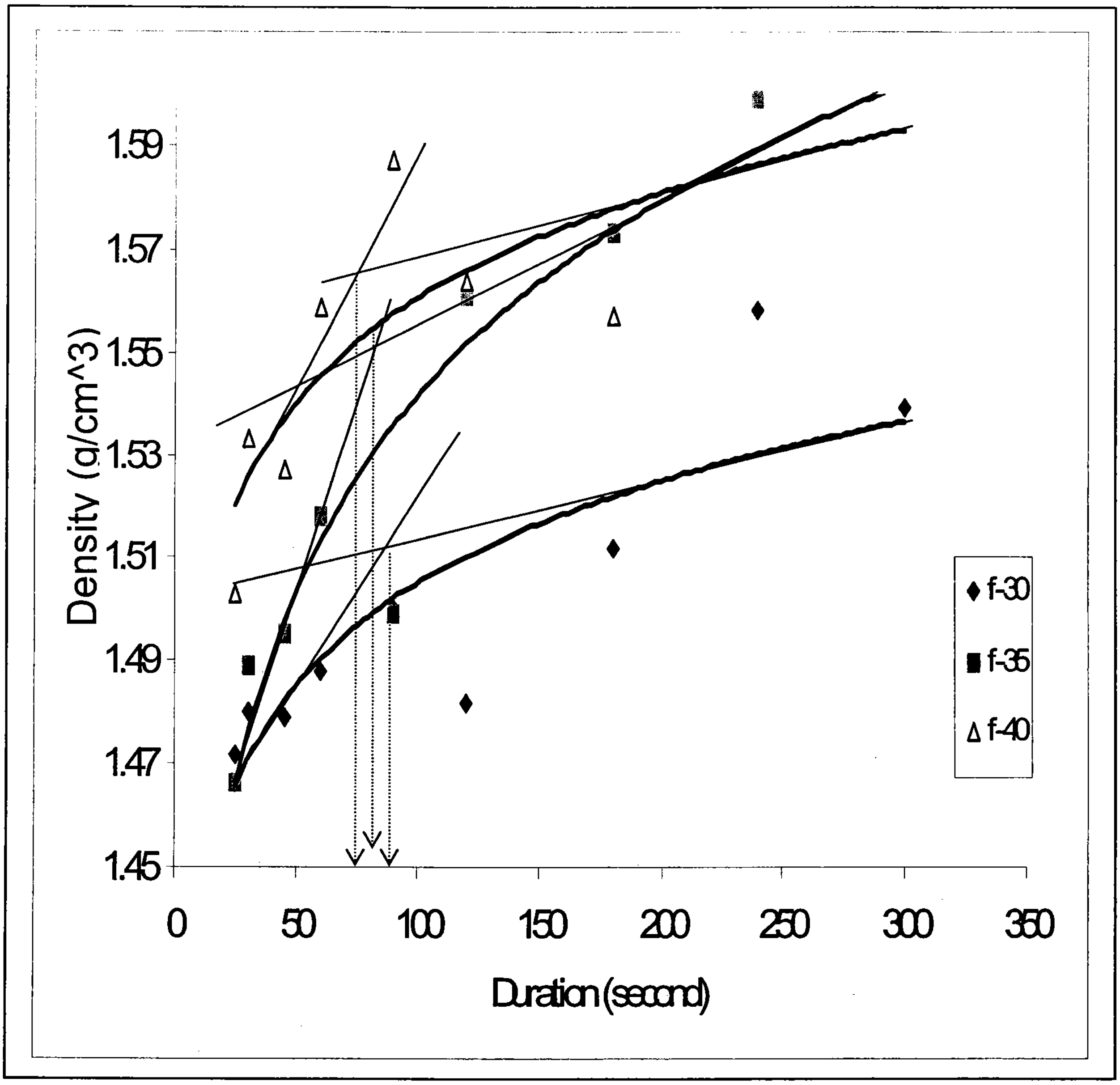

Figure 3 The Relationship between Duration and Density in Various Frequencies

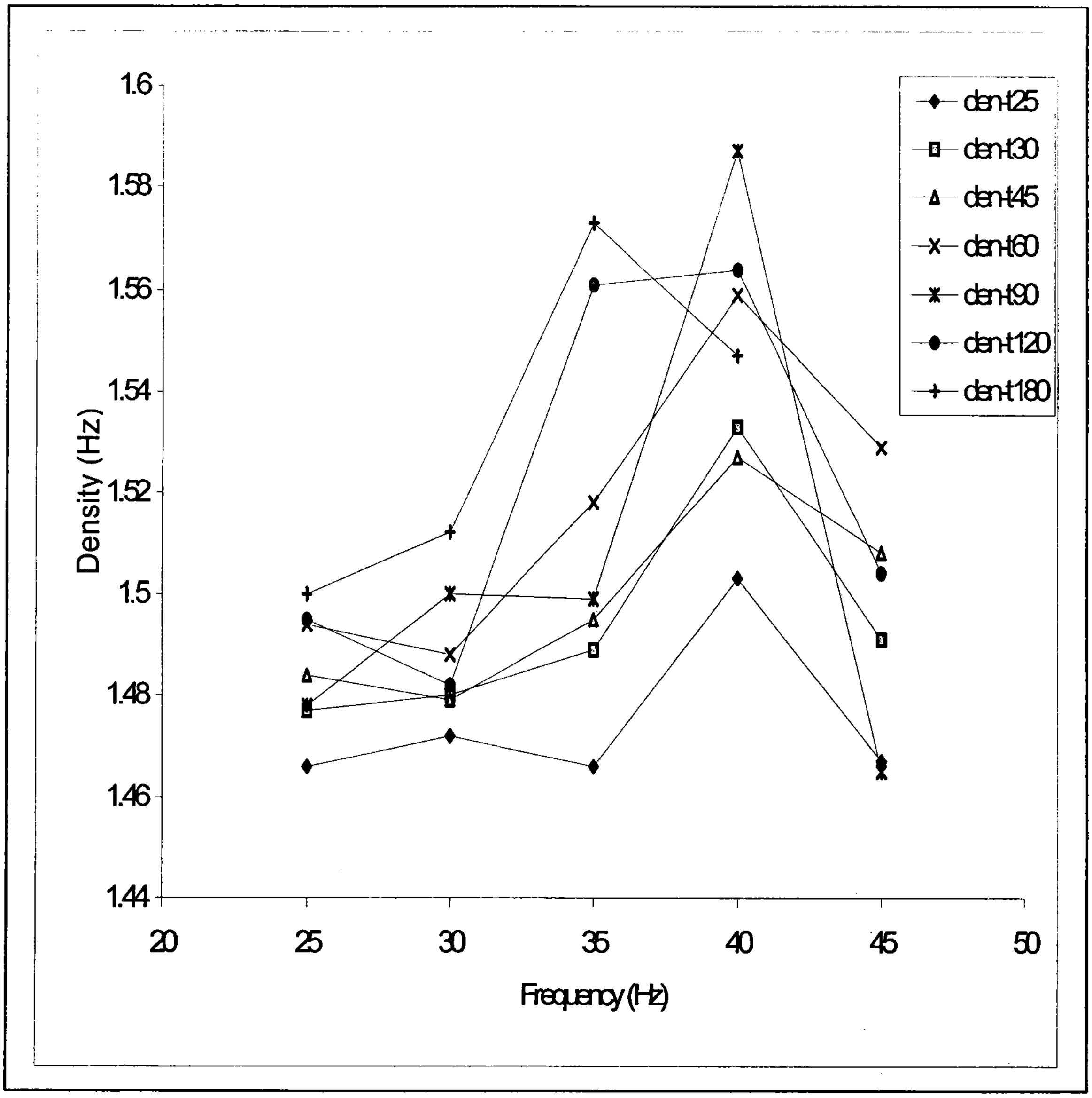

Figure 4 The Relationship between Frequency and Density in Various Vibrating Durations

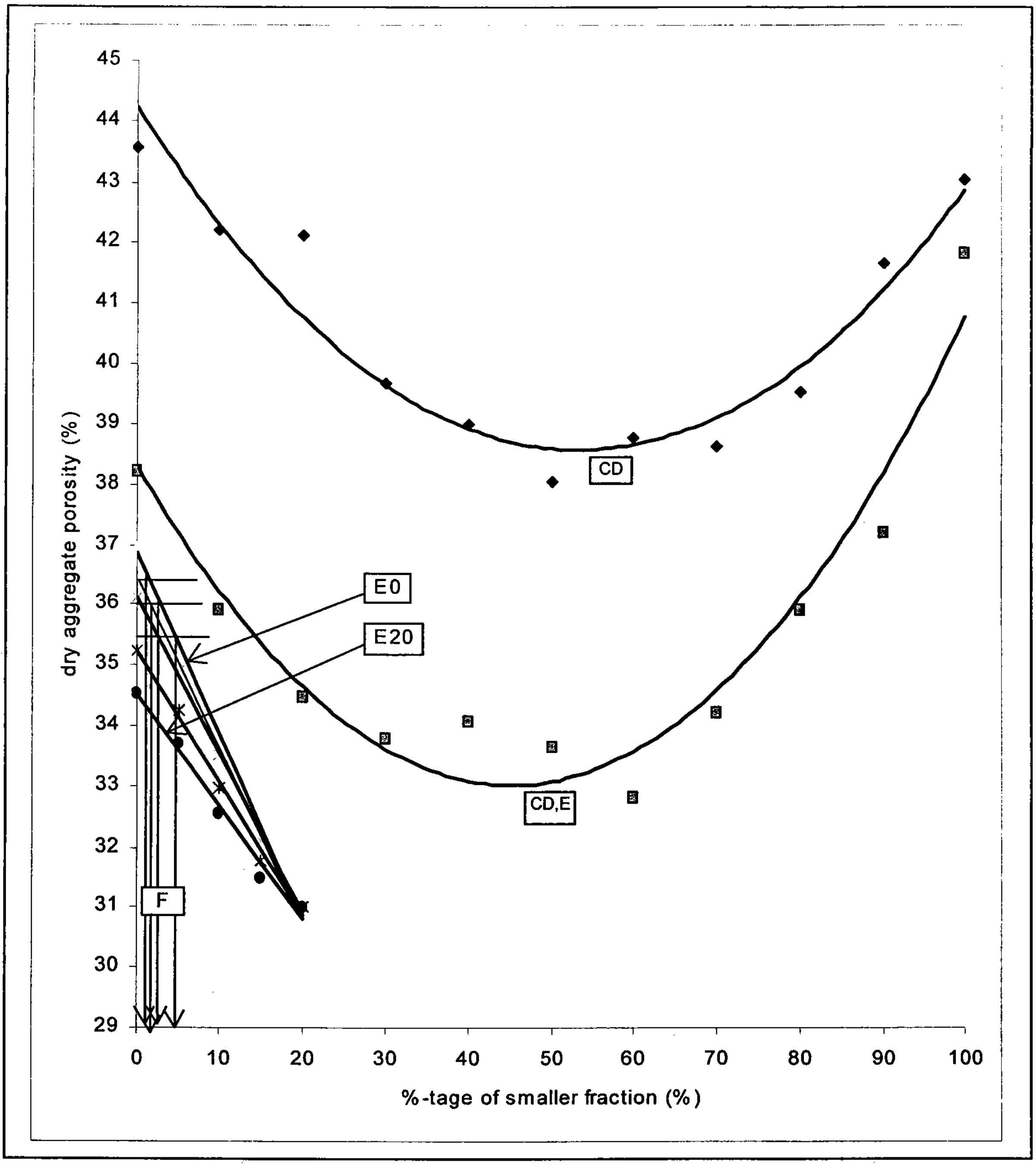

Figure 5 The Porosity of the four-Size Aggregates Used in this Investigation
(Max. Size Aggregate # 10 mm)

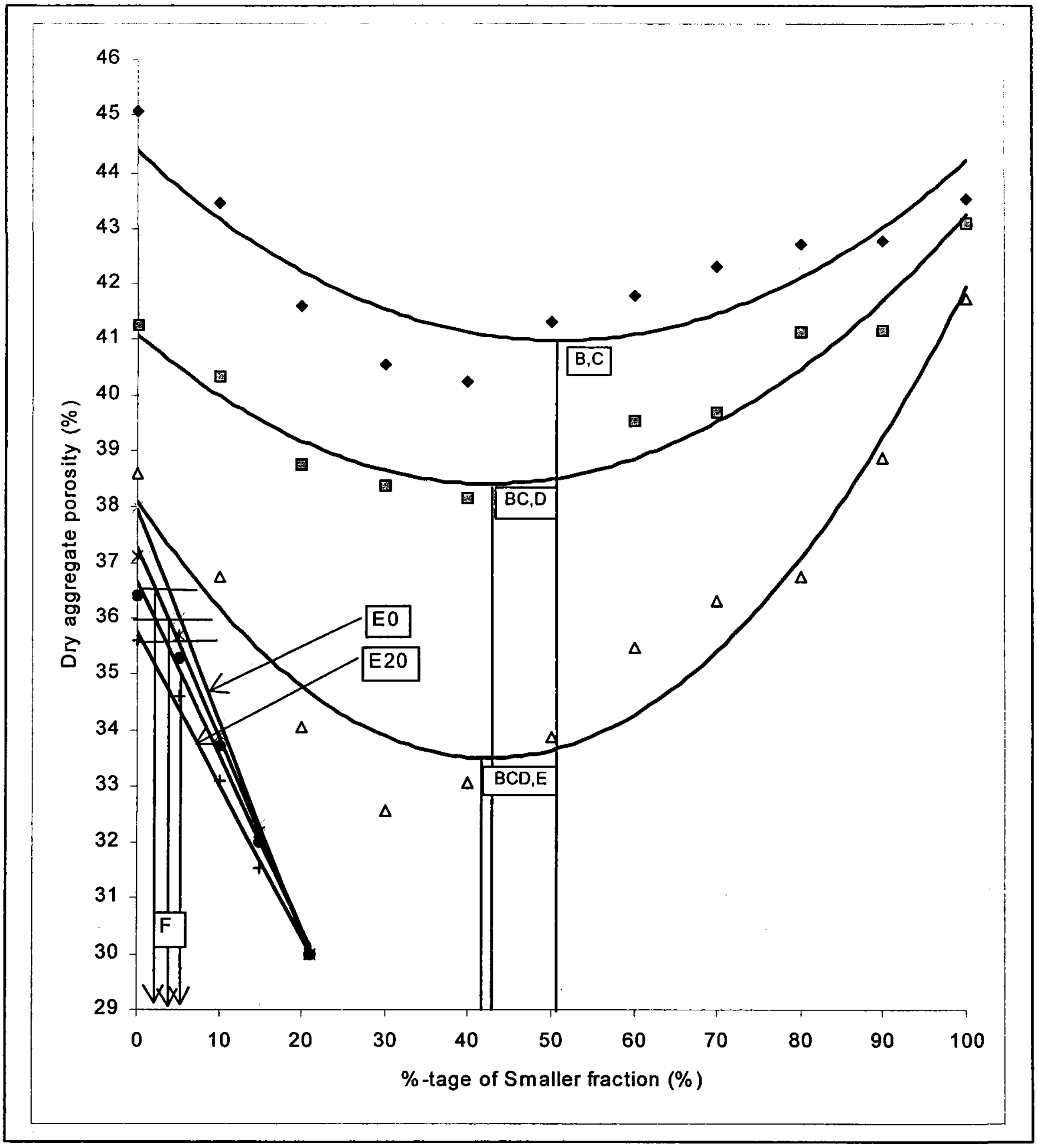

Figure 6 The Porosity of the Five-Size Aggregates Used in this Investigation (Max. Size Aggregate # 14 mm)

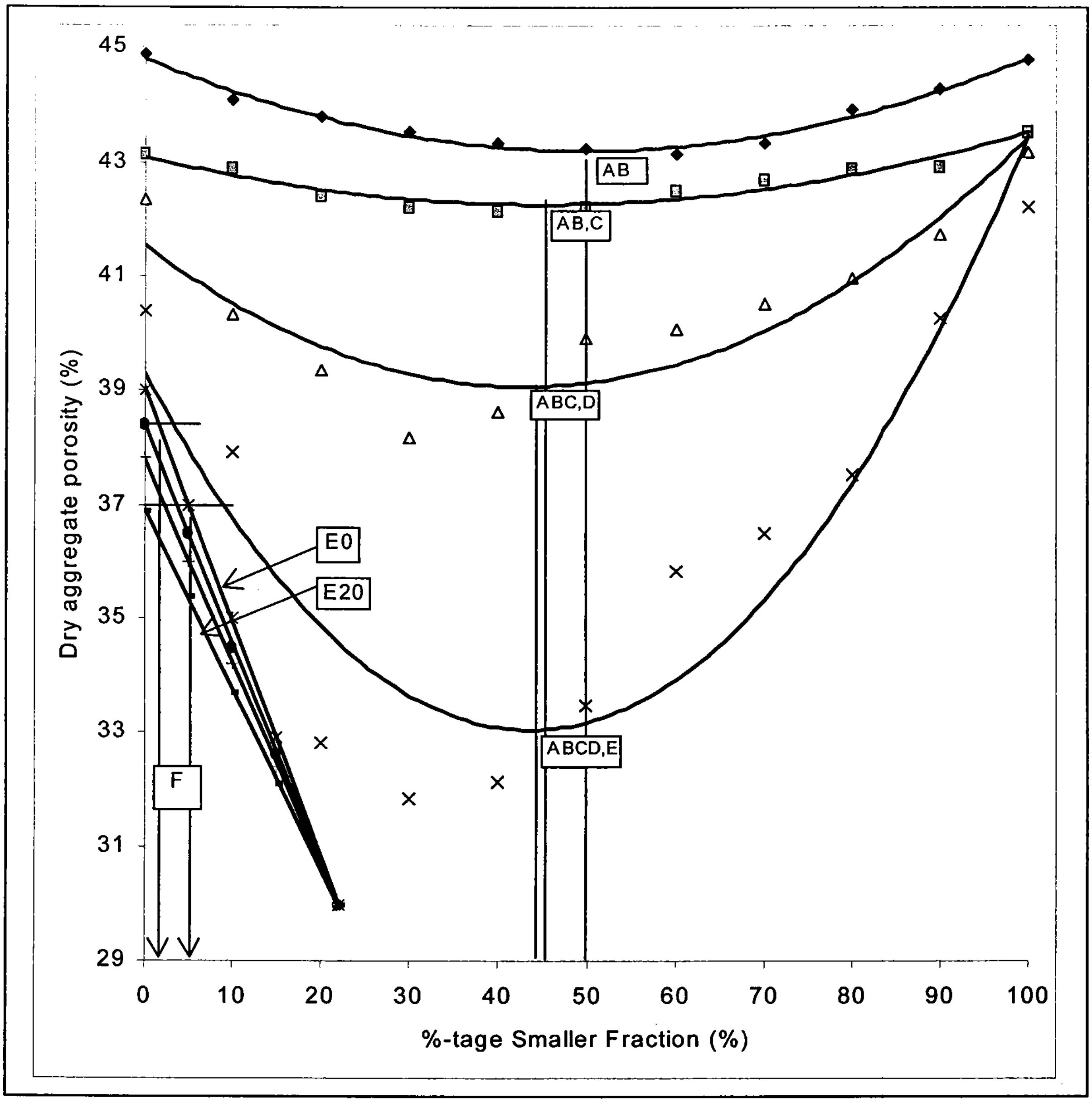

Figure 7 The Porosity of the Six-Size Aggregates Used in this Investigation
(Max. Size Aggregate # 20 mm)

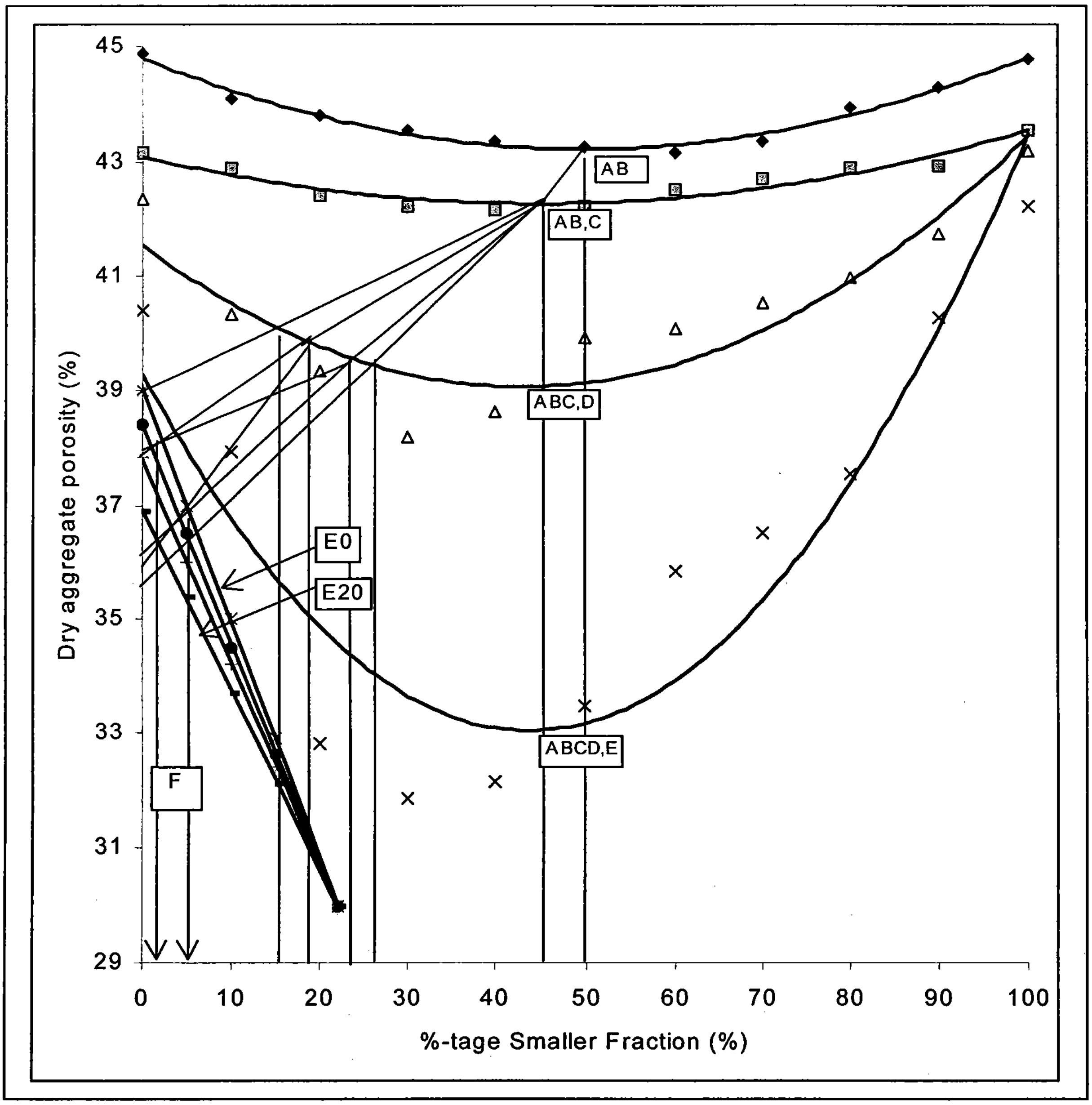

Figure 8 The Porosity of the Six-Size Aggregates Used in this Investigation
(Max. Size Aggregate # 20 mm, Modified Gradation)

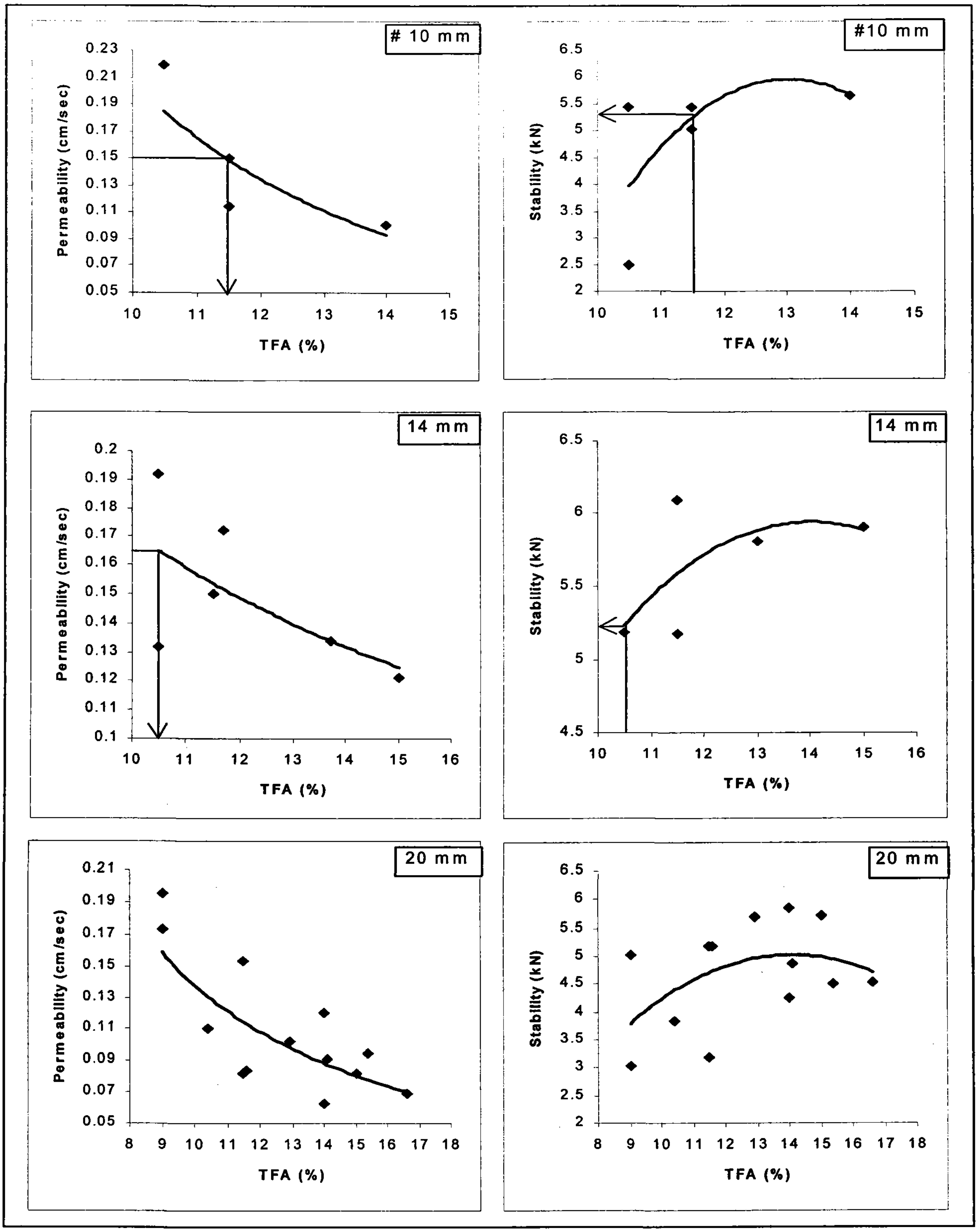

Figure 9 The Relationship between Permeability, Stability and Total Fine Aggregate at Minimum Void Dry Aggregate

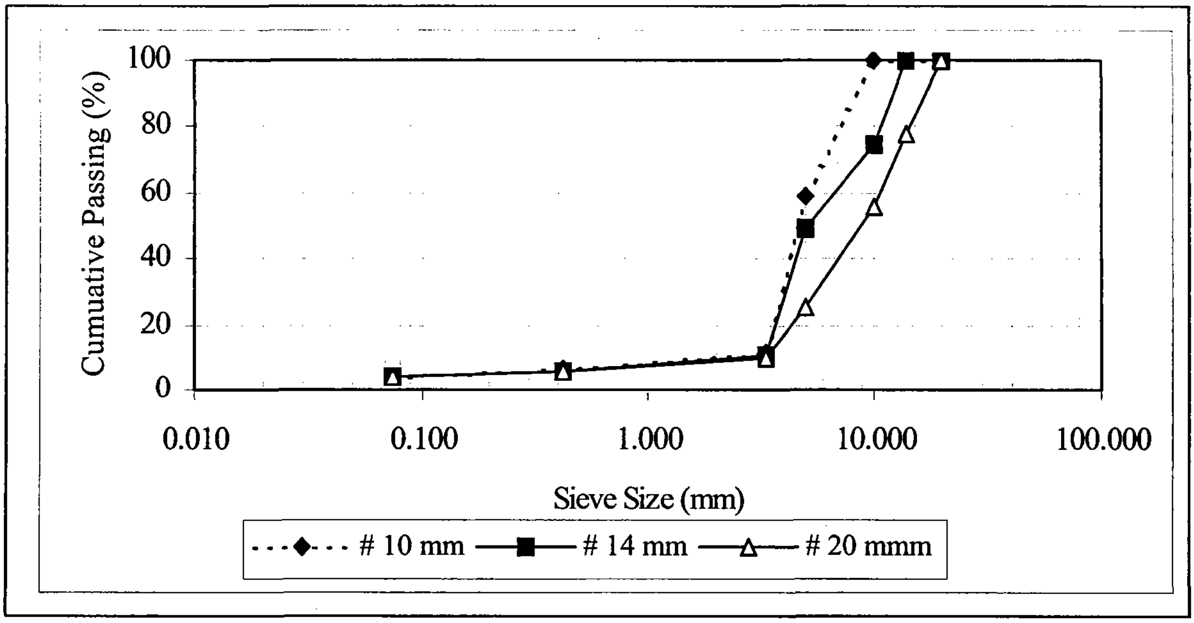

Figure 10 Selected Gradation for Mixtures Made of Maximum Aggregate Size 20, 14 and 10 mm

HIGH PERFORMANCE POROUS ASPHALT MIXTURES AS ALTERNATIVE MATERIALS FOR ROAD PAVEMENT WEARING COURSE

REPORT-2

OCTOBER 2003

By

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1.0 INTRODUCTION

The demands made on roads are ever increasing. Traffic volumes, tyre pressure, and loading have all increased over the last two decades. To meet the consequently more severe engineering criteria for success, road design has had to be similarly upgraded. Bituminous surfacing in Malaysia uses primarily the conventional 80/100-pen grade bitumen as a binder. Research work done by JKR and other research institutions overseas have shown that bitumen tends to harden while at the early stage of handling in storage, during mixing and in service.

It is not surprising that the bituminous surfacing in Malaysia failed primarily through cracking, more critically some of the bituminous surfacing suffer from surface down cracking as early as four years after laying, much earlier that their normal design life of seven to ten years. 80/100-pen grade available has a softening point of 42 –50^{O}C, and road Pavement temperature in Malaysia on the other hand ranges from 20^{O}C in the early hours of the day to as high 60^{O}C midday of hot day. Being visco-elastic material bitumen behaves as a viscous material at high temperature and consequently results in creep and permanent deformation (Mohamed and Zulakmal, 1997). This type of bituminous surfacing which has been used a long time in Malaysia, in this report will investigate into another type of surfacing in which researchers proved its better performance under heavy traffic and high temperature.Laboratory evaluation of porous asphalt was conducted to determine bitumen content of base bitumen (60/70) and polymer modified binder.

1.1 SCOPE OF WORK

The scope of this second report is limited on the determination of binder content for polymer modified bitumen and the conventional 60/70 bitumen, by using Cantabrian and binder drainage determination.

1.2 OBJECTIVES

The main objective of this report is to determine the minimum and maximum range of design bitumen content

2. LITERATURE REVIEW

2.1 Bitumen

Bitumen is composed of hydrocarbon and their derivatives and is therefore soluble in organic solvent such as carbon disulphide. At ambient temperature is a viscous liquid, although at very low temperatures it is almost a solid. It displays thermoplastic behaviour, that is, it softens gradually when heated to become liquid but returns to its original state when cooled. Bitumen is black or brown in colour, non-volatile and is resistant to chemical action. Bitumen is normally derived from the distillation of crude oil. By virtue of its water proofing and adhesive properties, bitumen is extensively used as a binder in the road construction industry. Bitumen is also susceptible to time of loading. A precise definition of bitumen can be found elsewhere (BSI, 1989).

2.2 Polymer Modified Binder (PMB)

Polymer modified bitumen is a modified bitumen obtained by incorporation of thermoplastic materials, synthetic thermohardening resins, powdered rubber or elastometers in bitumen (Fritz et al, 1992).

A polymer is a very large molecule compromising hundred or thousand of atoms formed by successive linking of one or two, occasionally more types of small molecules into chain or network structures (Hall, 1985). Despite large number of products from polymer, there are relatively a few types which are suitable for bitumen modification when polymer blended with bitumen should

- Maintain its premium properties during a long time /temperature/stress history
- Be capable of being processed by conventional mixing and laying equipment
- Be physically and chemically stable at storage, mix and service temperature
- Achieve a coating viscosity at normal mix temperature
- Be cost effective.

Polymers can be classified into four broad categories namely plastics, elastomers, fibers and additives/Coating. Plastics can in turn be subdivided into thermoplastics and thermo sets. Among the varies types of polymer mentioned above for asphalt modification, in this Investigation Styrene-Butadiene-Styrene (SBS) copolymers have probably been the most widely used to now.

2.4 Previous Investigation for Using SBS Binder

A large number of investigations of the relationship of binder properties and mix properties have been published. Only a few examples will be presented in a summarized form. Research done by (Choyce, 1989), (Khosla and Zahran, 1989) (Gschewendt and Sekera, 1993), (Sculler and Forsten, 1993), and (Srivastava and Baumgardner, 1993) has indicated that the addition of polymers, especially SBS to bitumen improves resistance to permanent deformation of asphalt mixes. The mix resistance to stripping which is based on retained Marshall stability resistance indicates SBS modified binder has a positive effect of water resistance of asphaltic mixtures (Beecken, 1992). The improvement in binder properties with the addition of SBS was found to manifest itself in the field and from laboratory wheel tracking and repeated load tests. In another wheel tracking investigation, the average deformation rate for a 200 pen bitumen modified with 6% SBS is equivalent to that of a 50/60 pen base bitumen. SBS modified bitumen does not require modifications to existing mixing plant. Only the mixing temperature needs to be raised by 10-20 °C. They can also be laid using a conventional paver since the SBS modified mix shows a compatible workability index with other conventional mixes (Hamzah, 1996). A published state art report conducted in SHRP (Coplantz et al 1993) describes American studies of the relationship between properties of modified on pavement performance one of these studies (Reese, 1989) indicates that the addition of bitumen slows down ageing process measured by penetration, viscosity and ductility.

2.3 Design Bitumen Content (DBC)

Traditional mix design methods which normally incorporate the Marshall test are not appropriate to design porous asphalt because of the insensitivity of the Marshall Stability values to variations in binder content. It is therefore appropriate specify the design binder content (DBC) for porous asphalt rather than the optimum binder content. The design binder content incorporates an upper and a lower limit. The lower limit of the DBC can be dictated by requirements to resist disintegration while the upper limit is specified to limit binder drainage yet maintaining a porous structure that would promote permeability.

Generally, the upper limit of the DBC should be sufficiently low to prevent binder run-off and to produce more porous mixes of higher permeability. On the other hand, the binder content should be sufficiently high to retard oxidation. A number of methods have been proposed to decide upon the upper limit of binder content that the aggregate skeleton can support without binder drainage before being laid and compacted. The method involved a trade off between resistance to disintegration, thickness of bitumen film coating and mix porosity. Porous asphalt disintegrates through loss of particles. To resistance to particle loss was evaluated through the newly developed Cantabrian test. In the test, a pre-weight Marshall specimen was subjected to a 300 drum revolution, with out steel spheres, in a Los Angeles drum at well-defined temperatures of either 18 °C or 25 °C.

3.0 METHODOLOGY

The methodology of evaluating the new binder centers on a laboratory characterization of the porous mixes based on the results obtained from the binder drainage, and Cantabrian tests. The binder drainage test was carried on loose samples while the Cantabrian test was carried out on cylindrical Marshall sample.

3.1 Binder Drainage Test

The binder drainage test is a simulative test developed by TRL, UK. Similar tests have been conducted for porous asphalt. An essential apparatus to carry out the binder drainage test is the perforated binder drainage basket. To speed up the test, a total of 5 units of such baskets were available at the Highway Engineering Laboratory of the USM.

3.1.1 Sample preparation for drainage test

The binder drainage test involved preparing a 1.1 kg porous mix and transferring it into the perforated basket, which in turn was placed, on a pre-weighed tray in an oven at the chosen binder drainage test temperature for 3 hours, the amount of binder drained was determined at the end of test period. In preparation of mixes for the binder drainage test, the mixing temperature was kept 5°C lower than the oven drainage temperature to ensure binder drainage took place during conditioning in the oven not during mixing in the mixer. At the end of the test, the mass of binder drained on the tray was determined. The test was repeated for a series of binder contents and the amount of material drained measured each time. Two samples were tested at each binder content. For mixtures prepared using base both binders, each of the mixing and binder drainage test temperatures were respectively 130 °C and 150 °C for base bitumen 60/70 and 175°C and 180°C for SBS.

The drained binder was partly composed of filler. To compensate this, the actual drained binder is computed from Equation (1).

$$R = 100 \times B\,[1-D/(B+F)]/(1100+B) \qquad \text{Equation (1)}$$

Where:

D = The mass of mass and filler drained (g)
B = The initial mass of binder in the mix (g)
F = The initial mass of filler in the mix (g)

3.3 Cantabrian Test

Porous asphalt disintegrates through loss of particles. The resistance to particle loss was evaluated by the Cantabrian test. The simulative test method adopted followed closely the procedure developed in Spain (Jimenez and Perez 1990). The test was carried out on Marshall samples.

To assess resistance to disintegration, one Marshall specimen was placed inside a Los Angeles drum that was normally used for testing abrasion of aggregates. Each specimen was subjected to 300 drum rotations without steel balls. The mass of the specimen after test was determined. Two specimens were tested per bitumen content. During the Cantabrian test, the average room temperature was measured.

The resistance to disintegration was expressed as a percentage of mass loss in relation to its initial mass as defined in Equation (2). The percentage of abrasion loss expresses the resistance to disintegration.

$$AL = \frac{m_i - m_f}{m_i} \times 100 \qquad \text{Equation (2)}$$

Where:

AL = Abrasion loss (%)

m_i = Initial mass of specimen (g)

m_f = Final mass of specimen (g)

3.1.1 Preparation of Marshall Samples

Porous asphalt specimens were prepared using materials whose properties are described in report I. Aggregates and fillers were batched in metal containers to produce one specimen weighing approximately 1.1 kg. The batched aggregates were placed in a thermostatically controlled oven at the desired mixing temperature for a period of at least 4 hours. The adopted mixing and compaction temperatures for a given binder type is shown in Table 1 in appendix .

Bitumen, which arrived in bulk, was subjected to no more than two cycles of heating. Sufficient quantity of bitumen during one laboratory session was placed in the oven at the required mixing temperature until it fully liquefied for mixing with the aggregates.

A 100-mm inner diameter steel mould and base plate were used in conjunction with the Marshall hammer. Both steel moulds and their corresponding base plates were placed in the oven together with the aggregates.

An electrically heated paddle mixer was used to blend the aggregates and bitumen. The mixer was first calibrated and then set to the required mixing temperature. Mixing of dry aggregates was accomplished for 1 minute. Then the correct amount of binder was poured onto the dry mixed aggregate and wet mixing continued for a further 1 minute. The amount of bitumen required was calculated as a percentage of the total mix. The mixer was transferred into the heated mould and partially compacted by means of 15 tamping. Full compaction was accomplished using the Marshall hammer once the mix temperature dropped to the desired compaction temperature shown in Table 1. Each specimen was subjected to 50 blows per face corresponding to the normal compaction level. Specimens were left overnight to cool prior to extrusion. Specimen weight and geometry were noted. Specimen volumetric properties will be based on its geometry. The specimens were then ready for the Cantabrian Test.

3.4 Design Binder Content Criteria

The binder content of porous asphalt could not be optimized as in the Marshall method of mix design for dense mixes. This is due to the insensitivity of the Marshall stability with variations in binder content. Besides, the binder content could not be optimized from the relationship between mix density versus binder content.

The choice of design binder content for porous mix is to satisfy the following criteria:

(a) A minimum binder content to assure resistance against particle losses resulting from trafficking.

(b) A maximum binder content to avoid drainage or runoff.

In other words, the lower and upper limits of the DBC are respectively determined from the Cantabrian and binder drainage tests.

4. TEST RESULTS AND DETERMINATION OF DESIGN BINDER CONTENT

4.1 Binder Drainage Test Results

The binder runoff data, presented as a graphical plot between retained binder content, corrected for filler content, versus mixed binder content for mixes prepared using base bitumen 60/70 for each maximum aggregate sizes are shown in Figure 9, 10 and 11, for SBS are shown in Figure 12, 13 and 14. The retained binder content and mixed binder contents are equal up to the point where drainage just begins. The maximum mixed binder content is the binder content at which the retained binder peaked to a maximum value. The mixed binder content M is assumed to coincide with a 0.3% drainage. The target binder content is equivalent to (M-0.3)%. Target binder content for mixtures made of base bitumen 60/70 and SBS are shown in Table 2 in appendix.

Based on the drainage test, target binder content as the upper limit content for SBS is higher than the base bitumen 60/70. These values will be lower when the maximum aggregate size is increased. The complete results was shown in table 5 and 6 in ppaendix.

4.2 Cantabrian Test Results

The relationship between abrasion loss versus binder contents for mixtures using base bitumen 60/70 for each maximum aggregate sizes are shown in Figures 3, 4 and 5, for SBS are shown in Figures 6, 7 and 8. Generally, the abrasion loss decreases as the binder content increases while the curve slopes downward and becomes flatter when a certain percentage of bitumen is exceeded. Based on this graphs can be determined the lowest limit content for SBS and base bitumen 60/70 as shown in Table 3.

The original Cantabrian tests conducted in Spain was carried out at a well-defined temperature of either 18°C or 25°C. The suggested maximum permitted abrasion loss is below 35% and generally lower than 30% if the test was carried out at 18°C (Jimenez and Gordillo 1990) or below 25% and generally lower than 20% if the test was carried out at 25°C (Jimenez and Perez 1990). The typical laboratory temperature at the time the test was done was about 29°C. Earlier studies by (Samat 2003) were made to study the relationship between abrasion loss and temperature, which is shown in Figure 8. At 29°C, the acceptable abrasion loss is 18%. With these criteria, the respective binder contents for base bitumen and SBS as shown in Table 3. These values will be lower when the maximum aggregate size is increased.

4.3 Design Binder Content (DBC)

In general, the Cantabrian test established the minimum binder content requirement while the binder drainage test establishes the upper limit of the DBC. Conventionally, the design binder content (DBC) lies between the maximum and minimum values described above are summarized in Table 4.

5. CONCLUSIONS

1. The increasing of maximum aggregate sizes 10, 14 and 20 mm in porous asphalt will decrease minimum binder content, maximum binder content and design binder content for base bitumen and SBS.
2. The upper limit and design binder content for SBS is always higher than base bitumen 60/70, except for minimum binder content.

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7. APPENDIX

Table 1 Mixing and Compaction Temperatures for Cantabrian

Binder Type	**Mixing Temperature (^{0}C)**	**Compaction Temperature (^{0}C)**
Base 60/70	140	130
SBS	170	155

Table 2. Target binder content for base bitumen 60/70 and SBS

Maximum Aggregate Sizes (mm)	Target Binder Content (%)	
	Base Bitumen 60/70	SBS
10	7.4	8.2
14	6.8	7.4
20	5.9	6.2

Table 3. The lowest limit content for Base Bitumen 60/70 and SBS

Maximum Aggregate Sizes (mm)	The Lowest Limit Content (%)	
	Base Bitumen 60/70	SBS
10	3.4	3.2
14	3.3	3.0
20	3.2	3.0

Table 4 Design Binder Content for Base Bitumen 60/70 and SBS

Maximum	Base Bitumen 60/70 content	SBS

Aggregate Size (mm)	(%)			(%)		
	Min.	Max.	DBC	Min.	Max.	DBC
10	3.4	7.4	5.4	3.2	8.2	5.7
14	3.3	6.8	5.0	3.0	7.4	5.2
20	3.2	5.9	4.5	3.0	6.2	4.6

Table 5. Results of Binder Drainage (60/70)

Tipe	Bit. Con. (%)	Wt. Tray (g)	Wt tray+bit.(g)	Wt. Bit (g)	B (g)	D (g)	F (g)	R (%)
Agg.14	5	3,63	3,63	0	57,9	0	44	5,0004
	5,5	3,66	3,66	0	64,02	0	44	5,4999
	6	4,6	4,6	0	70,21	0	44	5,9998
	6,5	4,94	5,06	0,12	76,47	0,12	44	6,4935
	7	10,42	11,71	1,29	82,8	1,29	44	6,9291
	7,5	3,63	5,82	2,19	89,19	2,19	44	7,3767
	8	3,66	11,35	7,69	95,65	7,69	44	7,5593
	8,5	4,6	28,23	23,63	102,19	23,63	44	7,1263
Agg.10	6	4,72	4,72	0	70,21	0	44,62	5,9998
	6,5	5,19	5,19	0	76,47	0	44,62	6,5
	7	6,23	6,33	0,1	82,8	0,1	44,62	6,9948
	7,5	3,59	5,4	1,81	89,19	1,81	44,62	7,3986
	8,4	4,7	26,96	22,26	100	22,26	44,62	7,0507
Agg.20	4,5	7,11	7,14	0,03	51,83	0,03	44,83	4,4984
	5	7,12	7,79	0,67	57,9	0,67	44,83	4,9678
	5,5	4,5	5,32	0,82	64,02	0,82	44,83	5,4585
	6	4,64	9,79	5,15	70,21	5,15	44,83	5,7312
	6,5	8,26	18,31	10,05	76,47	10,05	44,83	5,9614
	7	3,98	26,41	22,43	82,8	22,43	44,83	5,7701
	7,6	3,95	43,21	39,26	92	39,26	44,83	5,5036

R = (100 x B x (1-(D/(B+F)))/(1100+B)

Where:
R = Retained binder (%)
D= the mass of binder and filler drained (g)
B= the initial mass of binder in the mix (g)
F= the initial mass of filler in the mix (g)

Table 5. Results of Binder Drainage (60/70)

Tipe	Bit. Con. (%)	Wt. Tray (g)	Wt tray+bit.(g)	Wt. Bit (g)	B (g)	D (g)	F (g)	R (%)

Agg.14	6,5	6,15	6,15	0	73,7	0	42,41	6,4991
	7	6,05	6,73	0,68	79,8	0,68	42,41	6,9604
	7,5	5,3	7,5	2,2	86	2,2	42,41	7,3739
	8,5	6,15	25,59	19,44	98,5	19,44	42,41	7,3275
Agg. 10	7	6,32	6,32	0	78,9	0	41,91	7,0028
	7,5	5,28	6,72	1,44	85	1,44	41,91	7,4184
	8	6,48	8,61	2,13	91,1	2,13	41,91	7,8709
	9	6,82	17,59	10,77	103,6	10,77	41,91	8,3318
	9,5	6,22	31,8	25,58	110	25,58	41,91	7,9009
Agg. 20	5	6,23	6,35	0,12	56,5	0,12	42,93	4,9948
	5,5	6,11	7,72	1,61	62,5	1,61	42,93	5,4187
	6	6,86	13,04	6,18	68,5	6,18	42,93	5,6666
	6,5	6,45	22,04	15,59	74,6	15,59	42,93	5,6368
	7,5	6,88	32,61	25,73	87	25,73	42,93	6,0132
	8	7,62	34,76	27,14	93,3	27,14	42,93	6,4043
	8,5	6,37	45,23	38,86	99,7	40,86	42,93	6,0647

Where:
R = Retained binder (%)
D= the mass of binder and filler drained (g)
B= the initial mass of binder in the mix (g)
F= the initial mass of filler in the mix (g)

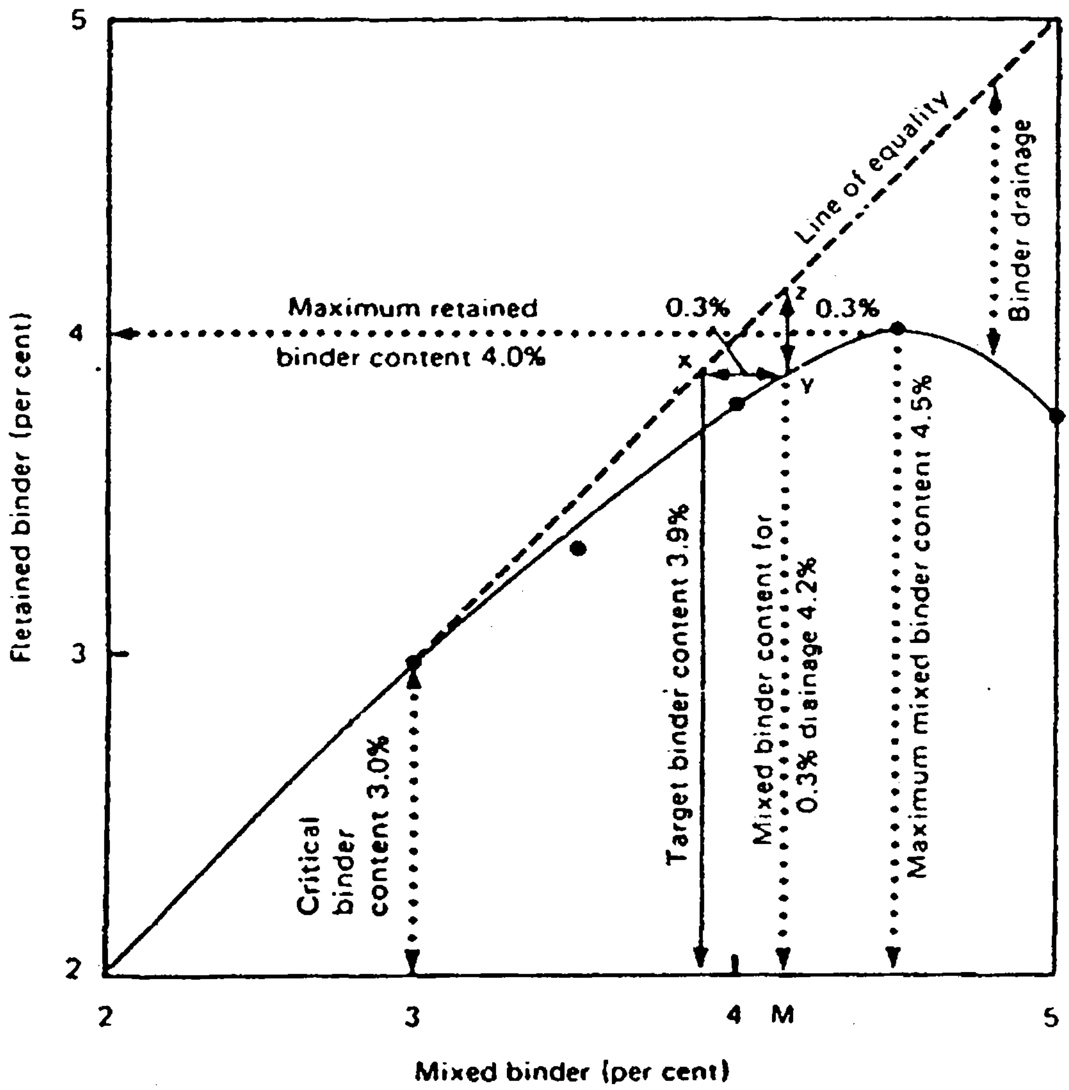

Figure 1 Typical Plot from the TRL Binder Drainage Test

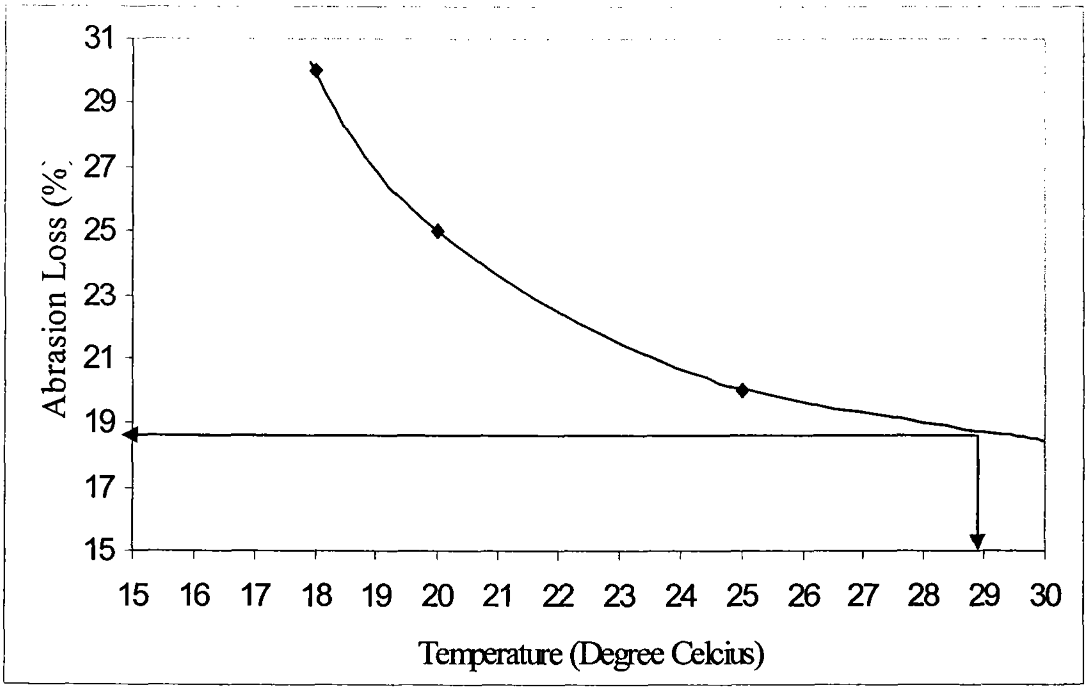

Figure 2. Relationship Between Abrasion Loss and Temperature

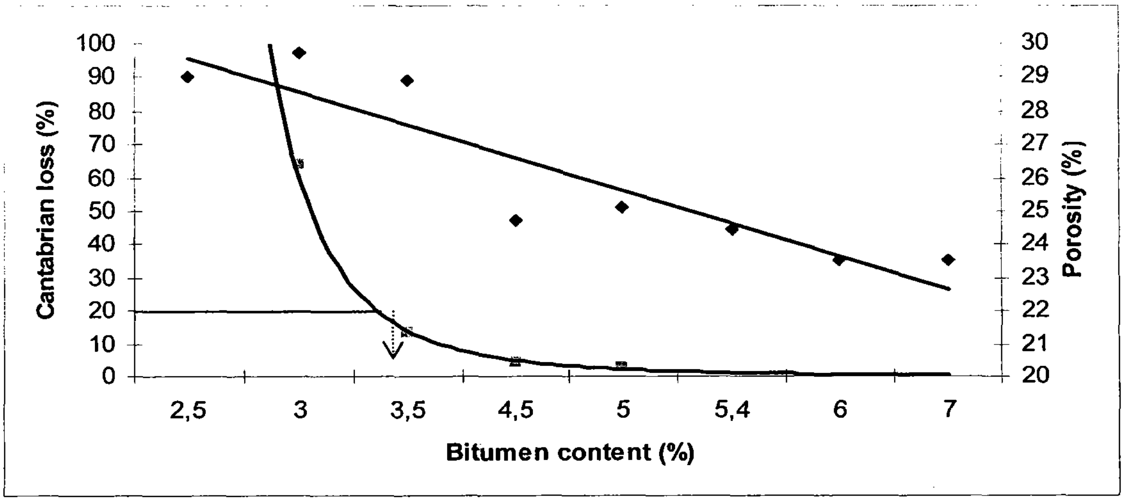

Figure 3: The relationship between Cantabrian loss, porosity and BC for max. agg. Size 10mm

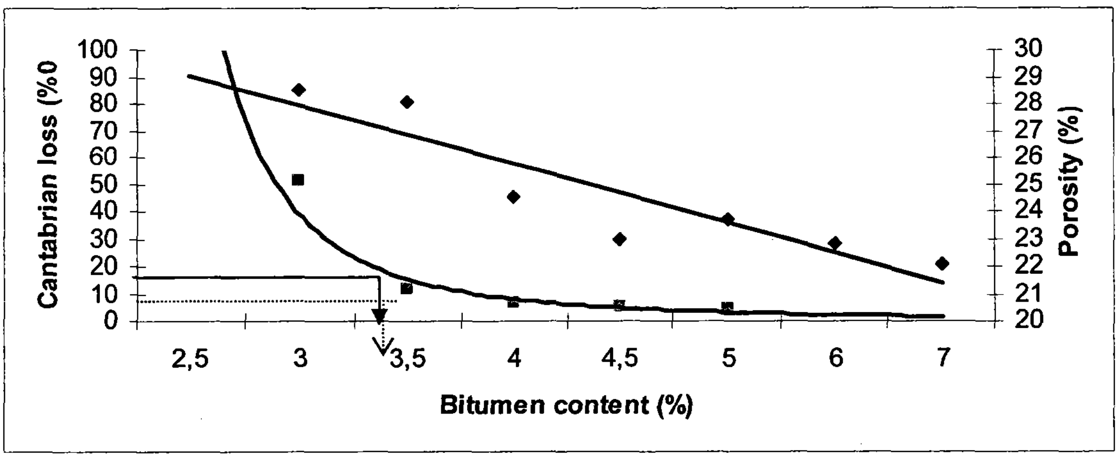

Figure 4: The relationship between Cantabrian loss, porosity and BC for max. agg. Size 14mm

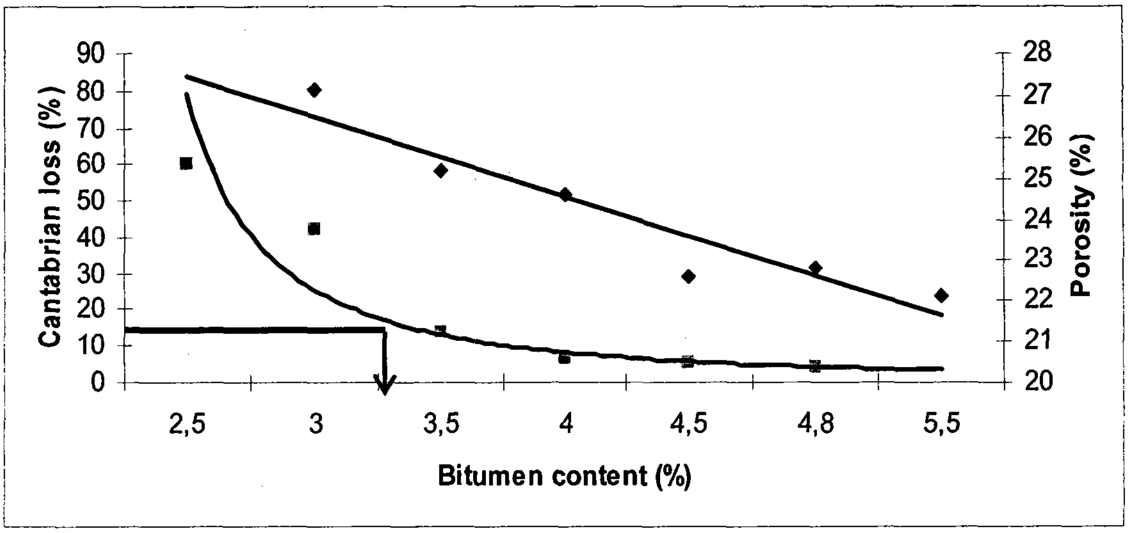

Figure 5: The relationship between Cantabrian loss, porosity and BC for max. agg. Size 20mm

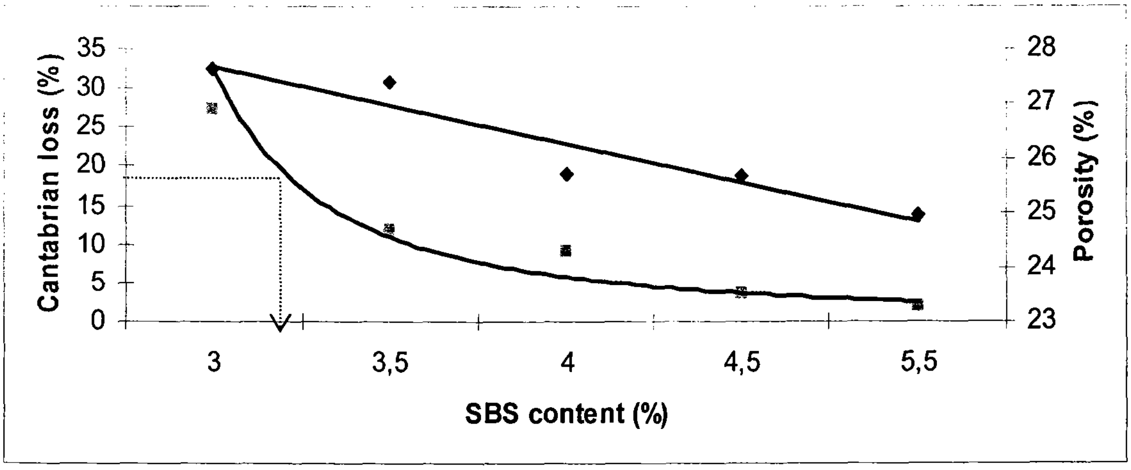

Figure 6: The relationship between Cantabrian loss, porosity and SBS content (Agg. Size 10 mm)

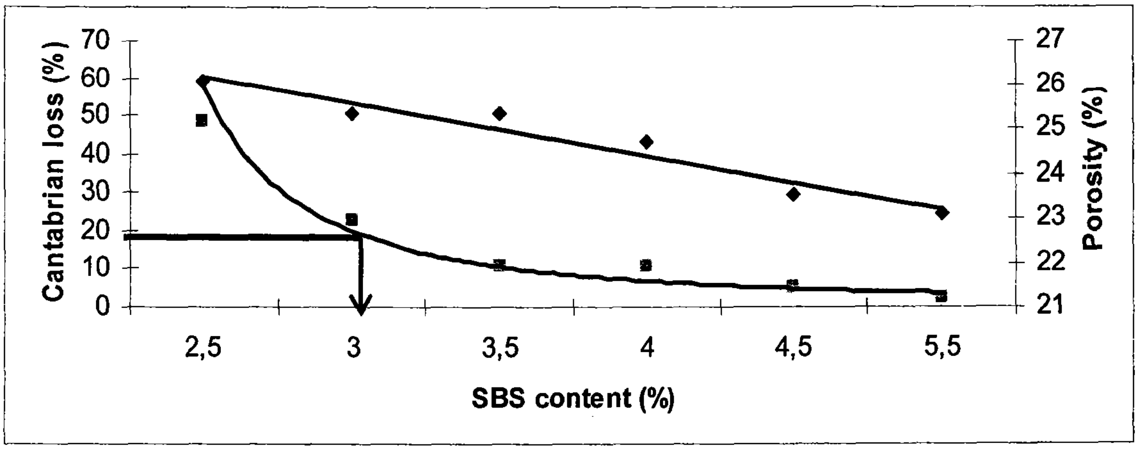

Figure 7: The relationship between Cantabrian loss, porosity and SBS content (Agg. Size 14 mm)

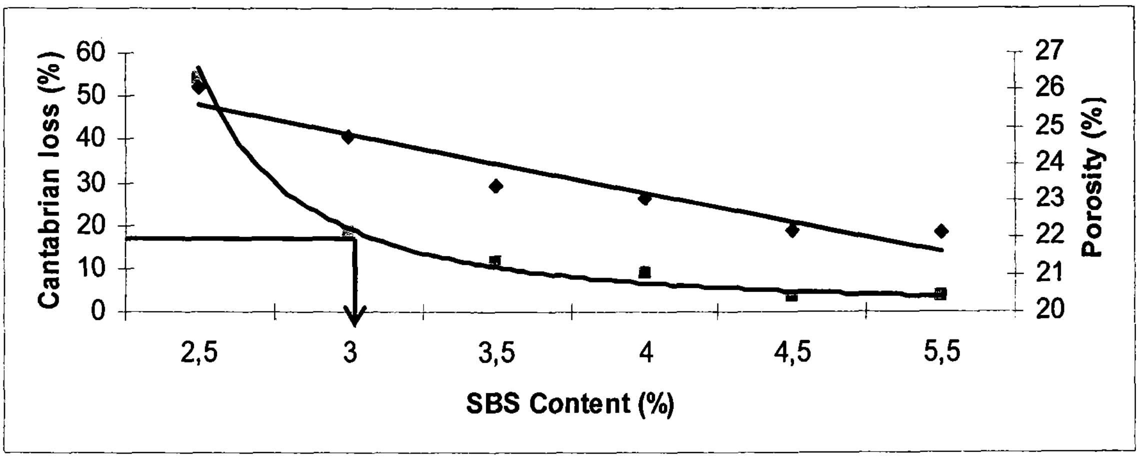

Figure 8: The relationship between Cantabrian loss, porosity and SBS content (Agg. Size 20 mm)

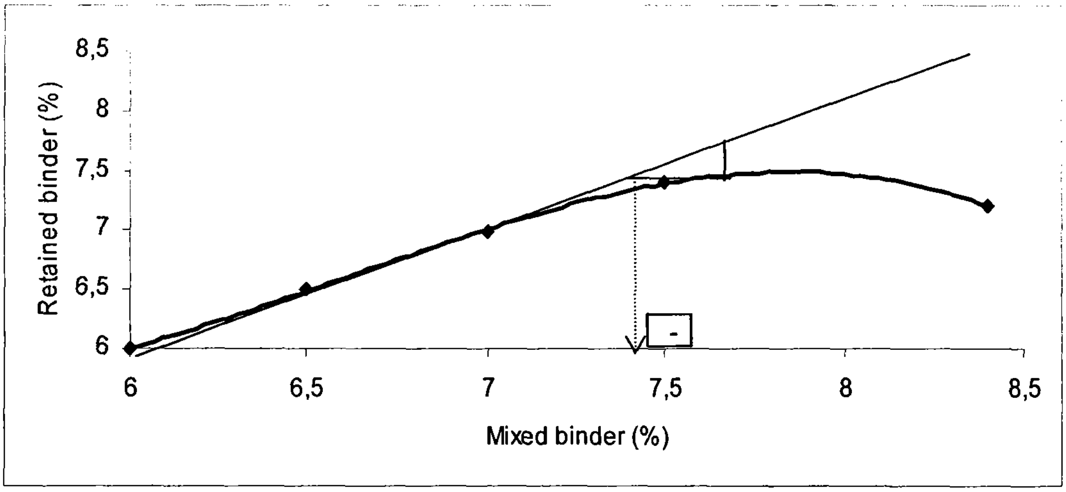

Fig. 9 The result of drainage test for determining the upper limit content of BC (agg. max. 20 mm)

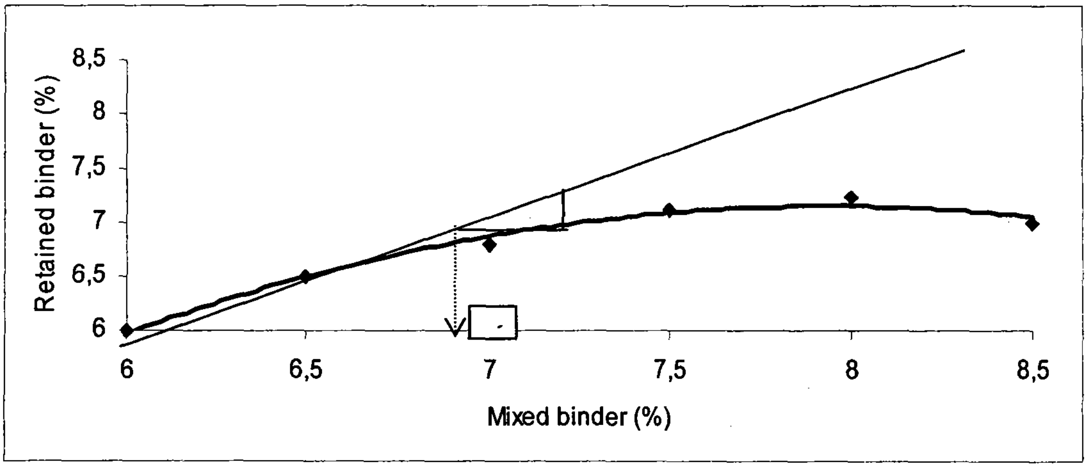

Fig. 10 The result of drainage test for determining the upper limit content of BC (agg. max. 20 mm)

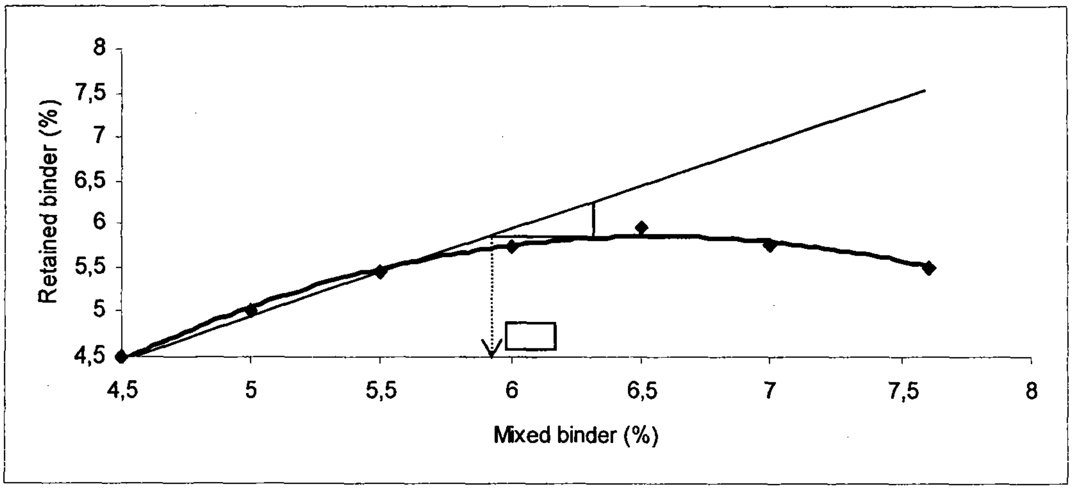

Fig. 11 The result of drainage test for determining the upper limit content of BC (agg. max. 20 mm)

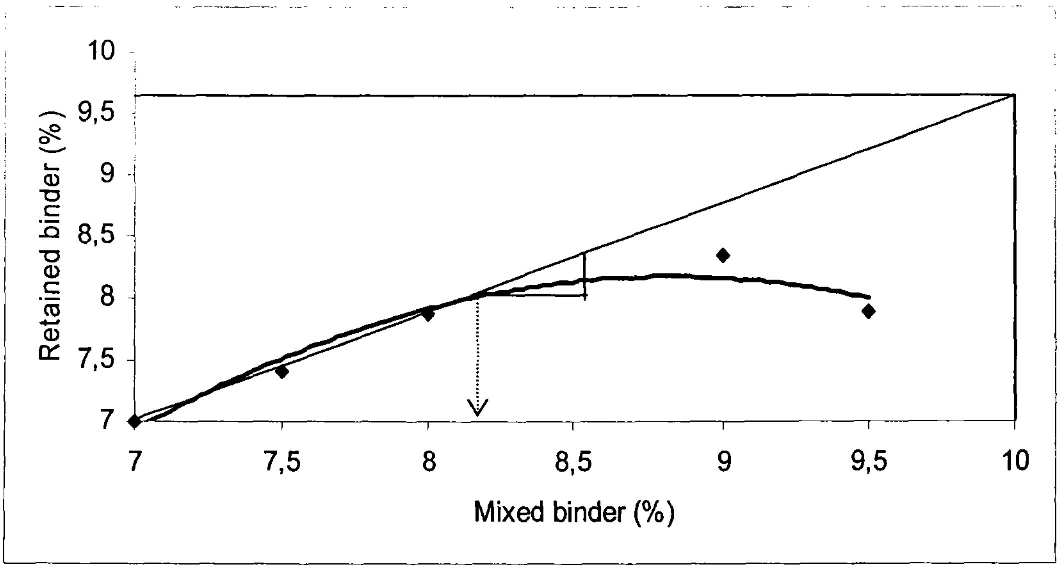

Fig. 12 The result of drainage test for determining the upper limit content of SBS (agg. max. 10)

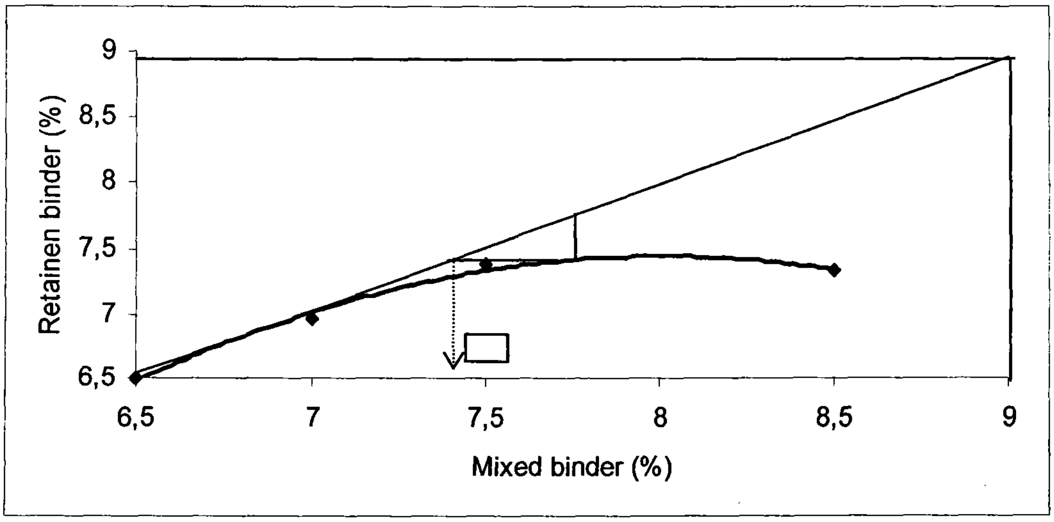

Fig. 13 The result of drainage test for determining the upper limit content of SBS (agg. max.14)

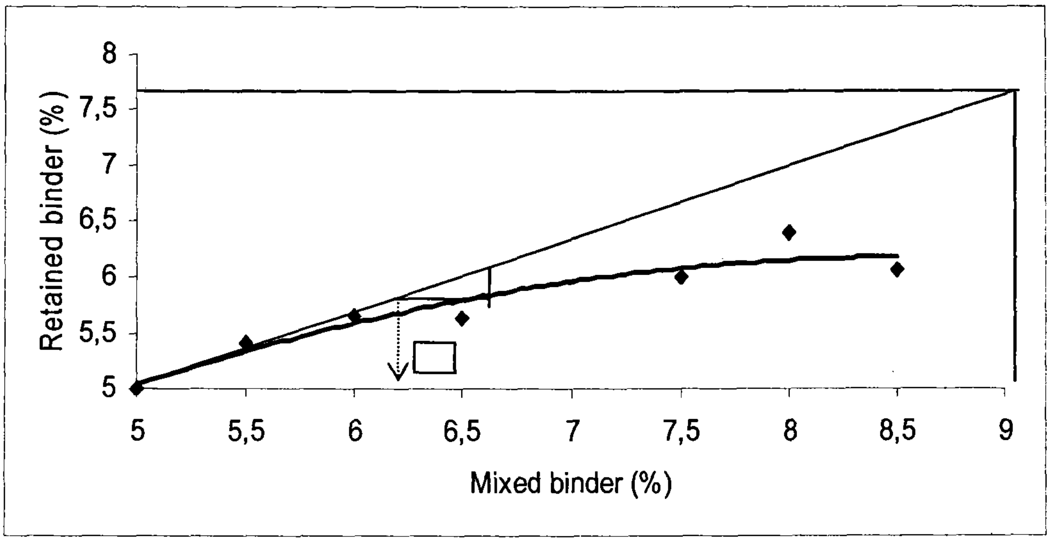

Fig. 14 The result of drainage test for determining the upper limit content of SBS (agg. max.20)

DEVELOPMENT OF BINDER FOR
DENSE GRADED AND
POROUS ASPHALT MIXTURES

REPORT-3

MARCH 2004

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EXECUTIVE SUMMARY

A major concern today afflicting pavement surfacings includes excessive permanent deformation as a result of frequent repetitions of heavy axle loads and other field problems such as placement difficulties (tender mixes), excessive displacement under traffic (low stability). Pavement distress caused by thermal cracking, raveling and stripping result in higher maintenance cost, shorter life service, higher life cycle cost and loss of lives as a consequence of traffic accidents. Bituminous surfacing in Malaysia failed primarily through cracking, more critically some of the bituminous surfacing suffer from surface down cracking much earlier that their normal design life of seven to ten years. The main aim of this report is to summarize the development a new product called CRABit comprising of crumb rubber, additives and antioxidant, which would facilitate mixing of aggregates, binder and crumb rubber in the form of a dry process. Modification of bitumen with rubber has important consequences on the engineering properties of bituminous binders. To achieve this laboratory binder mixer was designed and fabricated to check the mixed binders for compatibility.

1.0 INTRODUCTION

1.1 Problem Statement

Bituminous surfacing was originally introduced in Malaysia around 74 years ago (Mohamed and Zulakmal, 1997), while the other countries like the United States started using bituminous surfacing for about 100 years ago (Krchma and Gagle, 1974). They have been used in Europe since the 1850's (Croney, 1977). In the 1930's, Malaysian road network consisted mainly of semi grout, also known as bitumen grouted stone and double surfacing dressing. In the 1960's the British based bitumen Macadam was accepted as a surfacing material. Bituminous materials primarily used in Malaysia use the conventional 80/100-pen grade which has a softening point of 42–52^{0}C. Malaysia lies in the tropics where the weather condition is different than in the cold regions and pavement temperature in Malaysia ranges from 20^{0}C in the early hours of the day to as high 60^{0}C during the midday. It is not surprising that the bituminous surfacing in Malaysia failed due to cracking or more specifically surface down cracking as early as four years after laying, much earlier that their normal design life of seven to ten years (Mohamed and Zulakmal, 1997).

The total length of road networks in Malaysia had more than doubled during economic boom and the funds allocated for road construction and upgrading have been increasing with each five-year development plan. The demands for road maintenance is partly due to increased road network and probably due to the inability of conventional premix to meet the demands of increased traffic loading and axle loading due to accelerated economic activities. This classical dichotomy and rheological weakness of the conventional binders has generated an increasing interest in the use modified binders in flexible pavements and creates a need to improve the performance of bitumen to minimize the stress cracking that occurs at low temperature and the plastic deformation at high temperature. In view of the traffic and environmental demands on the pavement performance, a conference on the use of rubberized bitumen in road construction in 1997 (Mohamed and Zulakmal, 1997) recommended the use of recycled tire rubber. The use of recycled tire rubber as an additive in various types of bituminous construction not only solve problem of waste disposal but also offers the benefit of resource recovery and the environment.

2.0 OBJECTIVES

1. To develop a new product named CRABit which is combination of 80/100 bitumen, crumb rubber and additive that would facilitate mixing with hot aggregates in the form of a dry process.
2. To study the rheological properties of CRABit using the Dynamic Shear Rheometer.

3.0 SCOPE OF WORK

The scope of the study is limited to the development of CRABit for improving bituminous mixes in terms of short term and long term ageing. The DSR (Dynamic Shear Rheometer) was used to obtain complex shear modulus and phase angle at various temperatures and time of loading. It also enables testing of wide variety of binders over a broad range temperature and this will be useful for predicting the change of behavior as a consequent to the addition of additives.

4.0 LITERATURE REVIEW

4.1 Bitumen

Bitumen is composed of hydrocarbon and their derivatives and is therefore soluble in organic solvent such as carbon disulphide. At ambient temperature it is a viscous liquid but at very low temperatures it is almost a solid. It displays thermoplastic behaviour, that is, it softens gradually when heated to become liquid but returns to its original state when cooled.

4.2 Bitumen Modification

Polymer modified bitumen is a modified binder obtained by incorporation of thermoplastic materials, synthetic thermohardening resins, rubber or elastometers or resins (Fritz et al., 1992). Research done by Choyce (1989) and Khosla and Zahran (1989) have indicated that the addition of polymers especially SBS to bitumen improves resistance to permanent deformation of asphalt mixes. A published state-of-the- art report conducted by SHRP (Coplantz et al 1993) indicates that the addition of modifier slows down the ageing process measured in terms of penetration, viscosity and ductility. This is achieved by the agglomeration of the polystyrene end-blocks into separate domains providing the physical cross-links for a three-dimensional polybutadiene or polyisoprene rubbery matrix. It is the polystyrene end-blocks that impart strength to the polymer and the mid-block that gives the material its exceptional elasticity (Vonk and Gooswilligen, 1989).

4.3 Rheological Property of Bitumen

Rheology has been extensively used to classify and evaluate bituminous binders according to their performance properties. This has led to a better knowledge of bitumen behavior that occurs when subjected to different thermal and mechanical conditions, as seen during road construction and service in the field. The stress and strain characteristics of asphalt and asphalt concrete mixtures are both time and temperature dependent. Under ordinary situation, bitumen exhibits elastic and viscous response simultaneously and hence described as visco-elastic. Test equipments such as rheometers, an example is shown in Figure 1, are normally used for characterisation.

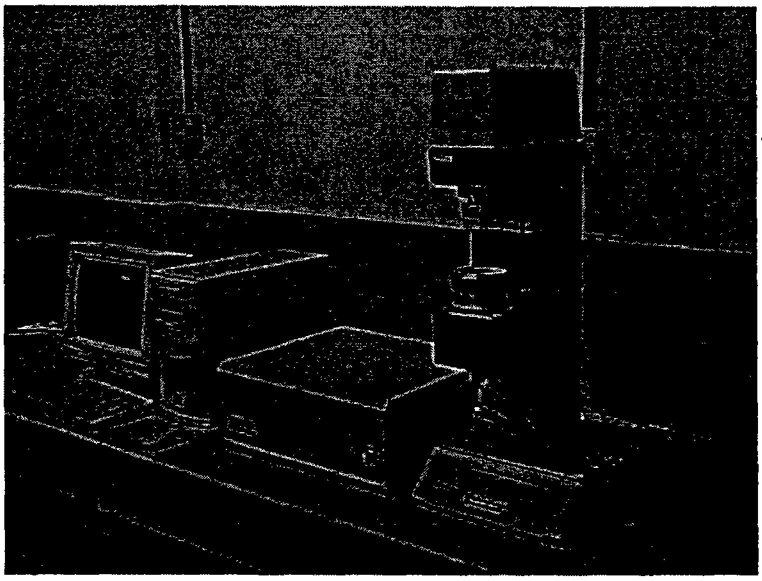


Figure 1 The Dynamic Shear Rheometer Equipment Used in This Investigation

4.4 Dynamic Shear Modulus and Phase Angle

The characteristics of binder can be assessed in terms of fundamental visco elastic properties such as the shear complex modulus, **G*.** This parameter is a measure of the total resistance to deformation, is shown in Figure 2 while the phase angle δ represents the relative distribution of this total response between and in phase component and out phase component. The relative distribution of these components is a function of composition of the material, loading time (frequency) and temperature. These parameters will be useful for this study since it can be used to predict pavement performance such as rutting and fatigue.

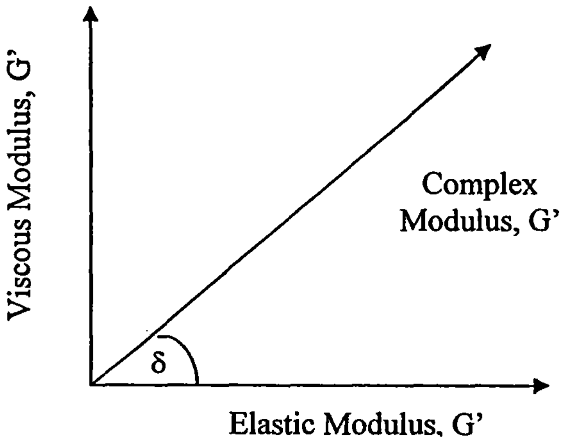


Figure 2 Relationship Among the Various Rrheological properties (Azemi, and Rehan, 1997)

5.0 METHODOLOGY

5.1 Introduction

The study focused on the development of a product comprising rubber, additive and anti oxidant formulated from various curing agents using dynamic vulcanization process. A high shear mixer was fabricated for this purpose. Rheological characteristics were extensively used in order to classify and evaluate bituminous binders according to their performance properties. This has led to a better knowledge of bitumen behavior that occurs when subjected to different thermal and mechanical conditions, as seen during road construction and service in the field. In the present work, rheology has been applied to evaluate the properties of modified and modified bitumen. The property of crumb rubber used is shown in Table 1.

Table 1 Property of Crumb rubber

Property	Value
Rubber Size	40 mesh
Specific gravity	1.08
Softening Point	47^0C

5.2 Tests on Bitumen

Physical tests are necessary to characterise the rheological properties of bitumen. The parameters outlined in Table 2 are used in this this study.

Table 2 Test Parameters

Asphalt Type	80/100, CRABit, SBS and (DAMA)
Modifiers	Crumb Rubber Powder + Additive+ Antioxidant
Blending Temperature	180°C, 2200 rpm
Time of Mixing	60 minutes

5.3 Preparation of Modified Bitumen

To blend bitumen with rubber and additives at elevated temperatures, a high shear mixer capable of mixing at 3000 rpm was fabricated. An 80/100 conventional binder was accurately weighed to 2 kg into containers and 1% and 2% concentration of CRABit was separately blended at the designated temperature 180^0C (356°F) and shearing rate 2200 rpm. Then, additives are blended with the rubberised asphalt and mixing continued for another 60 minutes to obtain a homogeneous mixture.

5.4 Characterisation of Bitumen Using the HAAKE Rheometer

The test variables in the characterisation process is outlined in Table 3. Although other response variable may be considered, the complex shear modulus (G*) was selected because it represents shear stiffness at a frequency simulating traffic loading. The 10Hz frequency also provides the best waveforms from the hydraulic servor with good repeatability.

Table3 Variables Used in the Bitumen Rheological Characterisation

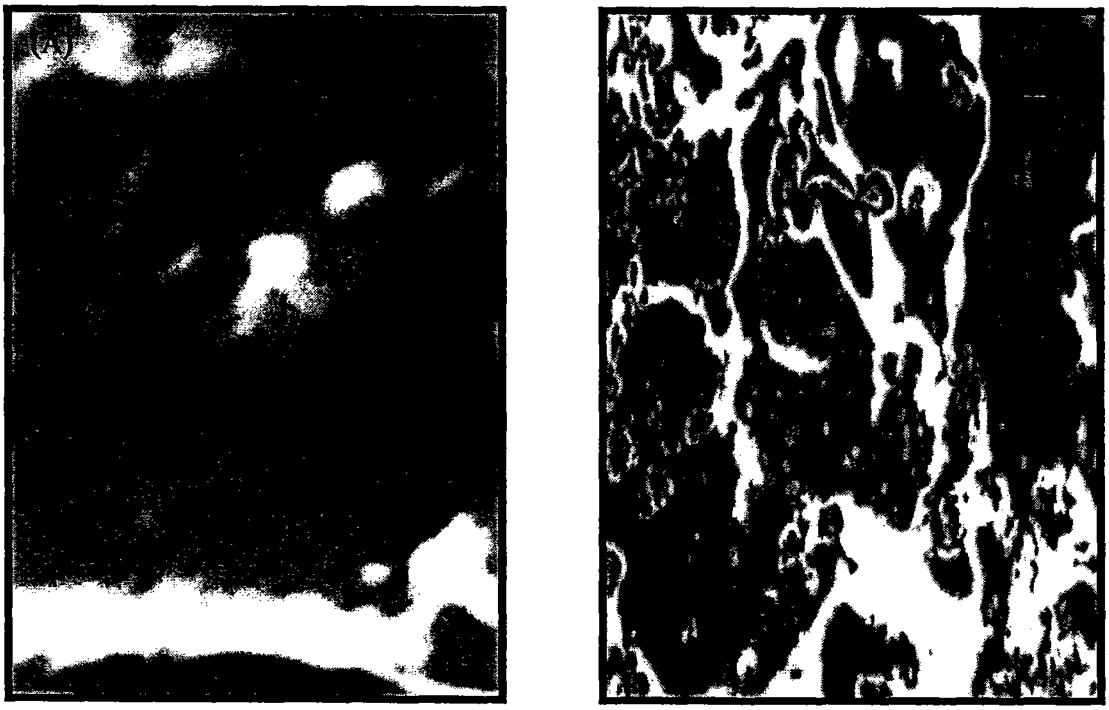
Frequency	Temperature	Deform	Plate/Gap
10 rad/sec	10-70°C	12%	25mm/1mm

In the non-destructive test, the material is subject to a sinusoidal stress. It enables measurements to be made on materials which cannot be sheared due to their three dimensional structure or due to their elastic properties when the material will not stay in the measuring gap. Furthermore, oscillation tests can be helpful to differentiate between two samples which cannot be distinguished by shear experiments because the test can separate the elastic and viscous components while shearing leads to an integrated characterization only.

6.0 RESULTS AND DISCUSSION

6.1 Compatibility Test of Modified Bitumen (CRABit)

The SEM was used to check compatibilty of and bitumen and the results are shown in Figure 4. The addition of 1% CRABit (Figure 4(a)) is more compatible due to homogeneous 'sponge-like' structure whereas a 5% content exhibits some discontinuities as can be seen clearly Figure 4(b).



(a) (b)

Figure 4 Microscopic Structure of (a) 1% and (b) 5% CRABit and bitumen Systems

6.2 Stiffness Modulus and Phase Angle

The stiffness modulus and phase angle results are shown in Tables 4 and 5 respectively. The results reveal information on the softness of the asphalt during application on the pavement and the recovery from deformation at elevated temperatures. The results indicate some behavioural changes that can be expected to affect field performance.

For a purely elastic material, the phase shift is zero hence cos δ equals 1. If so, G' 100% reflects the integral character G*. The loss modulus G" is a representative for the viscous properties of a material. Table 4 shows the viscoelastic behavior of modified and unmodified bitumens. It can be observed that the addition of 1% CRABit increases the modulus by 21% compared to unmodified bitumen while higher temperatures increase the corresponding modulus by 35.6%. An addition of 5% CRAbit increases the value by 45% while higher temperatures caused it to increase by 39.3%.

Table 4 Comparison of Stiffness Modulus of Modified and Unmodified Bitumen

TEMPERATURE (^{0}C)	STIFFNESS MDULUS (G*)				
	80/100	**CRI1%**	**CRI5%**	**DAMA**	**SBS**
10	7184000	6822000	6308000	7151000	902600
20	2283000	2418000	2455000	2525000	445000
30	321100	388400	330400	398500	70110
40	36710	45130	53370	56860	46330
50	5106	5535	5605	6947	18010
60	883.6	1372	1456	1999	9804
70	529.1	618.5	706.2	605.8	2635

The results indicating the complex modulus versus temperature is summarised in Figure 6. The modulus of modified bitumen is higher compared with the unmodified bitumen although the SBS modified bitumen exhibits the best performance. The stiffness modulus of CRAbit is not significantly different to DAMA and other rubber additives. Most of the asphalt met the SHRP criteria which should be greater than 1000 Pa at lower temperature while the crumb rubber and SBS exhibits favourable characteristics at higher temperatures.

The phase angle (δ) indicates the elasticity of a bitumen. The relationship between phase angle and temperature at loading frequency 10 Hz for all binders tested is shown in Table 4 and graphically plotted in Figure 6. All binders with the exception of SBS exhibits similar trend.

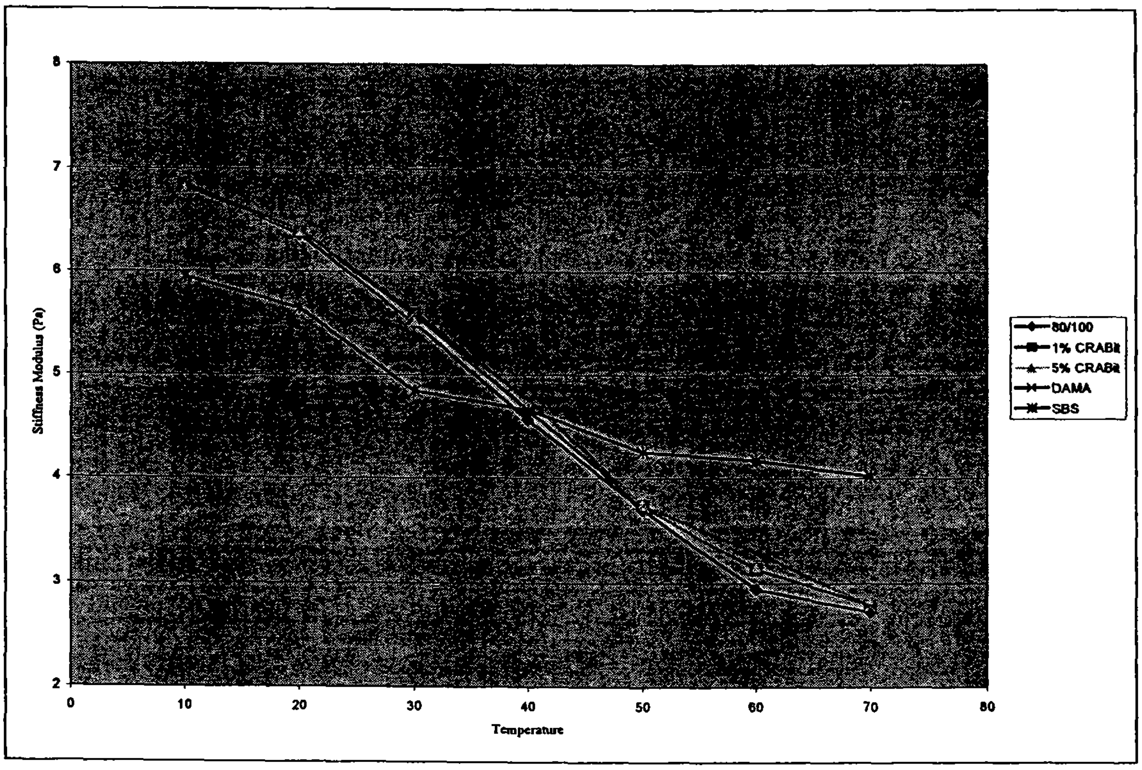


Figure 5 Complex Modulus as a function of Temperature

Table 5. Comparison of Phase Angle of Modified and Unmodified Bitumen

TEMPERATURE (^{0}C)	PHASE ANGLE δ [°]				
	80/100	CRI1%	CRI5%	DAMA	SBS
10	10.57	14.40	11.28	10.82	43.77
20	41.18	38.97	36.34	38.05	44.60
30	70.61	67.28	69.10	67.72	39.87
40	82.07	81.36	79.48	80.41	44.21
50	87.92	87.62	87.08	87.18	33.68
60	89.86	89.36	88.84	89.63	33.16
70	89.96	89.80	89.49	89.96	69.28

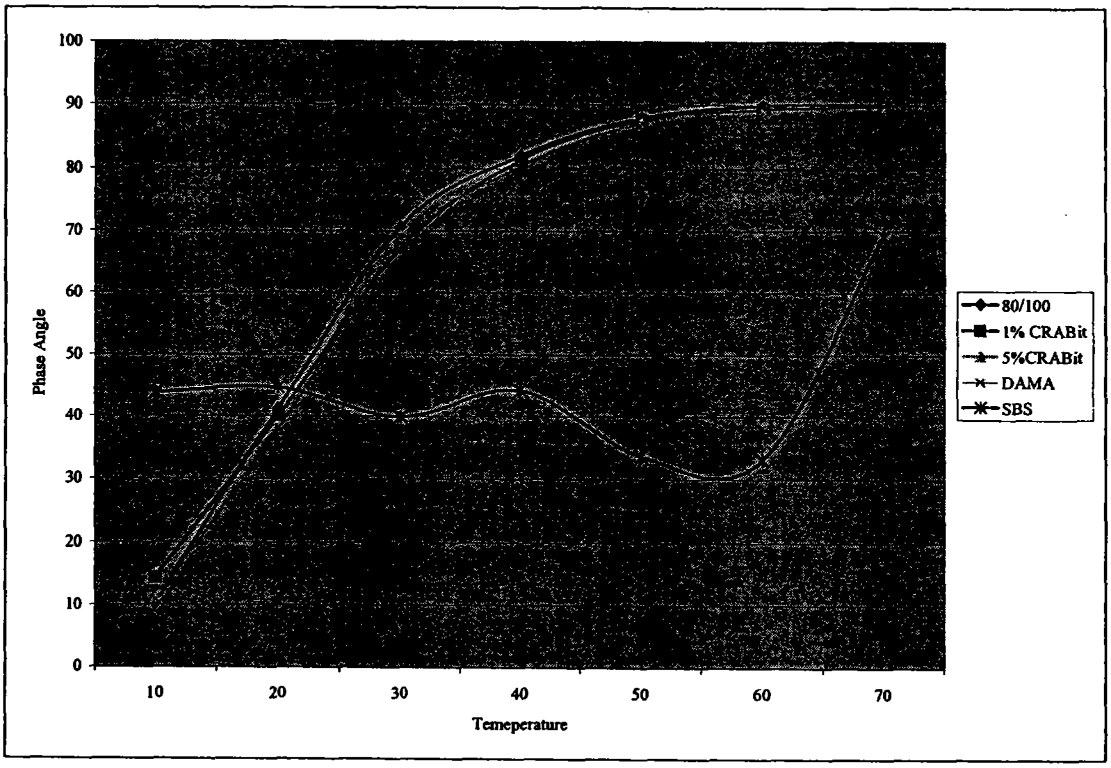


Figure 6. Phase Angle as a function of Temperatures

7.0 CONCLUSION

The effects of crumb rubber with modified additives on the performance related properties has been analyzed by using data collected from a rheometer. The binder rheological analysis indicates a high G* value. Therefore, the modified bitumen has the potential to improve the resistance to permanent deformation at high temperatures under extreme loading conditions. Despite the low CraaBit percentage, the modulus can increase by as much as 39% compared to normal binder

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PERFORMANCE AND CLOGGING BEHAVIOUR OF SINGLE LAYER POROUS ASPHALT

REPORT-4

MARCH 2004

By

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EXECUTIVE SUMMARY

In the 1970's, the fatality index on Malaysian roads exceeded 20. In 1991, a Cabinet Committee on Road Safety was set up to come up with measures to reduce the predicted number of deaths by 30% or translated into a fatality index of 3.14 by the turn of the century. Among the measures suggested included the application of porous asphalt. Up to now, a number of major trunk roads have been paved with porous asphalt. This report summarizes the characteristics of single layer porous asphalt made from maximum aggregate sizes 20, 14 and 10 mm. Their properties are evaluated in terms of permeability, marshall stability, disintegration, creep and wheel tracking test. The effects of clogging and over compaction were also investigated.

1.0 INTRODUCTION

Porous asphalt is an innovative road surfacing technology which allows water to seep into the asphalt mix through its continuous air voids. Porous asphalt is normally used as a wearing course material and always laid on an impervious base course. It can be effective in enhancing traffic safety particularly during rainy weather, reducing hydroplaning potential and having good skid resistance properties at high speed. The use of porous asphalt also reduces traffic noise and glare at night and on wet surfacing. In addition, porous asphalt exhibits superior resistance to permanent deformation. Two prime factors affecting the performance of porous asphalt is permeability loss and poor resistance to disintegration. These factors appear to be affected by the maximum aggregate size, aggregate gradation, binder type and content apart from traffic loading. In general, higher coarse aggregate content implicates higher porosity and permeability but a reduction in strength and lacking in durability. A porous asphalt mixture that becomes clogged up easily by dust and debris is not suitable for porous asphalt pavement construction. Experience in some countries such as USA, indicates that significant loss in permeability of porous pavement was experienced after two to three years because of clogging of voids by deicing materials or other debris (Mallick et. al., 2000). In Singapore, local residual soils deposited from dirty wheels and vehicles carrying earth has been a major source of materials contributing to clogging of porous asphalt layers.

2.0 OBJECTIVE

The objective of the research program is to determine the performance and clogging properties of single layer porous asphalt incorporating conventional binder and polymer modified binder (SBS).

3.0 SCOPE OF WORK

The scope of work during this stage of the research focus on the performance of single layer porous asphalt made with 10 mm, 14 mm and 20 mm maximum aggregate size. Performance is measured in terms of Marshall stability, porosity, density, void in mineral aggregate, void filled with bitumen, permeability, resistance to disintegration, indirect tensile modulus and creep stiffness modulus. A newly developed laboratory simulative test that attempts to simulate clogging in the field is included in the study.

4.0 LITERATURE REVIEW

4.1 Binder for Asphalt Porous

Bitumen is composed of hydrocarbon and their derivatives and is therefore soluble in organic solvent such as carbon disulphide. At ambient temperature is a viscous liquid, although at very low temperatures it is almost a solid. It displays thermoplastic behaviour. By virtue of its water proofing and adhesive properties, bitumen is extensively used as a binder in the road construction industry. Bitumen is also susceptible to time of loading.

Polymer modified bitumen is a modified bitumen obtained by incorporation of thermoplastic materials, synthetic thermohardening resins, powdered rubber or elastometers in bitumen (Fritz et. al., 1992). A polymer is a very large molecule compromising hundred or thousand of atoms formed by successive linking of one or two, occasionally more types of small molecules into chain or network structures (Hall, 1985). Despite large number of products from polymer, there are relatively a few types which are suitable for bitumen modification.

4.2 Previous Investigation Using SBS Modified Binder

A large number of investigations on the relationship between binder properties and mix properties have been published. Research carried out by Choyce (1989) and Khosla and Zahran (1989) have indicated that the addition of polymers, especially SBS, to bitumen improves resistance to permanent deformation of asphalt mixes. The mix resistance to stripping which is based on retained marshal stability resistance indicates SBS modified binder has a positive effect on water resistance of asphaltic mixtures (Beecken, 1992).

The improvement in binder properties with the addition of SBS was found to manifest itself in the field and from laboratory wheel tracking and repeated load tests. In another wheel tracking investigation, the average deformation rate for a 200-pen bitumen modified with 6% SBS is equivalent to that of a 50/60 pen base bitumen. The use of SBS modified bitumen does not require modifications to existing mixing plant. Nevertheless, the mixing temperature needs to be raised by 10-20 °C.

4.3 Porous Asphalt

The common measured properties of porous asphalt include voids in mix, Marshall stability and permeability. These properties are influenced by gradation, maximum aggregate particle and bitumen content.

To a large extent, porosity or void content determines the mix permeability. In porous asphalt, the quantity and gradation of coarse aggregate determines porosity, hence permeability. In general, porosity can be increased by increasing the proportion of the coarse mineral aggregate and reducing the amount of fine aggregate fraction (Cabrera and Hamzah, 1996). Abdullah, et. al., (1998) concluded that increasing void in the mineral aggregates and air voids in the mix by choosing the coarse aggregate gradation for the asphalt mix makes it more porous and water permeable.

The coefficients of permeability (k), in units of cm/sec, on porous asphalt reported by a number of researchers differ greatly. Average k values ranging from 0.16 to 0.41 cm/sec have been recorded. Permeability loss is imminent when voids close up. Increasing the binder content reduces permeability since the excess binder replaced the air voids.

In porous asphalt, the source of stability is from aggregate interlock enhanced by the coarse aggregate matrix. Generally, Marshall stability values for porous asphalt are significantly lower than that of dense asphalt. Marshall Stability increases as the gradation becomes less open by incorporating more fines.

5.0 METHODOLOGY

5.1 Materials

5.1.1 Aggregates

Crushed aggregates supplied by Kuad Quarry Sdn. Bhd. in Penang were used in this investigation. The aggregates were sieved into their respective size range or fractions. Aggregate grading used in this investigation was developed based on the packing theory. The detailed procedure has been described in Report 1.

5.1.2 Filler

The aggregate gradation indicates a 4.0% filler requirement. It is customary to use hydrated lime as filler in porous mixtures to resist stripping but in quantity not exceeding 2.0%. Hence, the additional 2.0% comprised of Ordinary Portland Cement (OPC).

5.1.3 Binder

Binder used in this research was conventional binder (penetration grade 60/70) and polymer modified binder (SBS) supplied by Shell Malaysia. The design binder content used in this investigation was based on the cantabrian and binder drainage tests, as detailed in Report 2.

5.1.4 Material as Clogging Agent

One of the main problems afflicting porous asphalt is loss of permeability due to clogging by road dust and detritus, tyre wear by-products, etc. Material clogging used for this research is clay-soil, where more than 90% passes the 0.075 mm sieve. The soils were taken from Sungai Petani (designated as Soil A) and Sungai Kecil Hilir sites (designated as Soil B) in Kedah.

5.2 Preparation of Porous Asphalt Specimens at DBC

To determine the properties of single layer porous asphalt, specimens were prepared to determine their marshall stability, volumetric properties, coefficient of permeability, resilient modulus, stiffness modulus and clogging of the single layer.

5.2.1 Preparation of Materials

Porous asphalt specimens were prepared using materials whose properties are described in Report 2. Aggregate and filler were batched in metal container. A 1100 g aggregate would suffice to cast one specimen. The batched aggregates were placed in an oven at the desired mixing temperature for a period of at least 4 hours. The base bitumen 60/70 and SBS binders respectively required at least 2 and 4 hours of preheating. The cycle of heating of each binder was kept consistent. The mixing and compaction temperatures for mixes prepared using the two types of binders are outlined in Table 1.

5.2.2 Preparation of Moulds

A 101.6 mm inner diameter and 89 mm height steel moulds were used in conjunction with impact Marshall compaction. Moulds for the Marshall hammer must be fitted with a collar. Both steel moulds and their corresponding base plates were placed in the oven together with the aggregates.

5.2.3 Mixing

An electrically heated vertical paddle mixer was used to blend the aggregates and bitumen. The mixer was first calibrated and then set to the required mixing temperature. The mixing process started with firstly feeding the hot aggregates into the mixer and then mixed dry for 1 minute. This step ensured uniformity and homogeneity of the aggregate blend. The correct quantity of bitumen was added into the blended aggregates and mixing continued. In all experimental work, the amount of bitumen required was calculated as a percentage of the total mix. The mixing process ceased as soon as all of the aggregate was thoroughly coated with bitumen which normally required less than 1 minute of mixing time. Once the temperature of the unsegregated mix had dropped uniformly to the required compaction temperature, an approximately 1100 g of the material was transferred in to the hot mould. Using a metallic rod, the sample was tamped 15 times around the circumference and 5 times in the centre to ensure uniformity of mix. The partially compacted sample was then ready for full compaction at 2x50 blows.

5.3 Tests for Porous Asphalt at DBC

5.3.1 Permeability Test

The equipment used for determining permeability in this investigation is a specially designed water permeameter which was designed based on the Leeds Water Permeameter. The coefficient of permeability of the specimen k was computed from Equation 1.

$$k = 2.3\ [a.L/At]\ [\log (h1/h2)] \qquad (1)$$

Where: k = coefficient of permeability (cm/s)

a = the cross sectional area standpipe (cm2)

t = time taken for water in the standpipe to fall from h1 to h2 (s)

A = cross sectional area specimens (cm2)

L = height of specimens (cm)

h1, h2 =water level at t1 and t2 (cm)

5.3.2 Marshall Stability

Conventionally, the Marshall test is simply a method of measuring stability and flow. In the stability test, the specimen was placed between two split breaking heads in an unconfined manner. Load was applied in a direction perpendicular to the longitudinal axis of the specimen at a constant rate of strain 50.8 mm/minute until failure. The maximum load developed during the test is known as the stability in kN and the amount of deformation that took place up to the moment of failure is term as the flow in units of mm. The reported stability is the average of two specimens.

5.3.3 Resistance to Disintegration

To determine the loss of aggregate from single layer porous asphalt specimen, the cantabrian test was carried out. In this test, a Marshall sample was subjected to 300 drum rotations in the Los Angeles drum. The abrasion loss was expressed in terms of the percentage mass loss compared to the original mass as illustrated in Equation (2).

$$P = \frac{P1 - P2}{P1} \text{x } 100 \qquad \qquad \ldots\ldots\ldots\ldots\ldots (2)$$

Where P is the abrasion loss (%), P_1 and P_2 are respectively the initial and final specimen mass in grams.

5.3.4 Indirect Tensile Resilient Modulus

The indirect tensile resilient modulus test was conducted using the Universal Asphalt Tester, MATTA. The procedure was described in ASTM D4123. Each specimen was tested at 25°C after 4 hours conditioning. The samples were initially subjected to 5 condition pulses. A 1200 N peak load was applied along the vertical diameter of the sample. The pulse period and pulse width were respectively 3000 ms and 100 ms while the rise time was 50 ms. Linear variable differential transducers monitored the resultant indirect tensile strain along the horizontal diameter. Since the test is undestructive, upon completion of this test, the same specimen was tested for Marshall stability.

5.3.5 Creep Test

The creep test is a method of assessing the resistance of bituminous materials to permanent deformation. The total deformation measured during the creep test consist of a reversible part and irreversible part (permanent deformation) as reported by Bolk (1981). In this investigation, the dynamic creep test was carried out using the MATTA.

5.3.6 Clogging Test

The clogging test is a newly test developed at the highway engineering laboratory of the University Sains Malaysia (USM). The preliminary clogging test involves the following steps:

- Determination of particle size distribution of the soil which acts as the clogging agent.
- Determination of the amount of soil required to fill voids in the specimen.
- Determination of a method to remove part of soils trapped or soils that clogged the specimen.

To determine the particle size of soil distribution of the clogging agent, preliminary trials were conducted on different combinations of Soils A and B. The selection of the right combined quantity was based on the time required to attain the minimum permeability which reflects the amount of soil filling up the voids. The minimum coefficient of permeability stipulated in the Korean standard for porous asphalt is 0.01 cm/sec while the corresponding value from TRL is 0.03 cm/sec. For the clogging test, permeability can be analyzed in units of cm/sec or sec. The minimum requirement of permeability adopted in this research is 0.03 cm/sec or about 240 sec. In the initial trial, two permeant concentrations were prepared by dissolving 10 and 25 g in 1 liter of water. The amount of soil particle passing and retained by the specimens were also determined. Next, fresh water was used to determine of permeability of the clogged specimen. To determine the method of removing of trapped soils from single layer porous asphalt required tools such as a brush (k1), water sprayer (k2) and a vacuum cleaner (k3). The selection of method was based on the highest permeability. The permeability tests for clogging begins after the sample has cooled overnight. Then, the specimen was tested for permeability and the value is used as a reference point to indicate the highest achievable permeability for unclogged or virgin specimens. for comparative purposes.

Preliminary tests were carried out to determine a suitable permeant concentration. Soil concentrations ranging from 1.0 to 2.0 g/liter was used. All preliminary clogging tests were carried out on specimens prepared using conventional binder. For this step, the procedure involves removing trapped soil using a vacuum cleaner. Before applying a vacuum cleaner, water was sprayed to loosen up trapped soils in the voids. The duration of spraying and vacuuming for each specimen was 15 seconds.

The procedure of the newly developed clogging test is summarized underneath:

1. Perform the initial permeability on virgin sample (without clogging agent).
2. Mix soil with water (g/liter) water and stirred vigorously for several minutes.
3. Pour soil-water mixes into the permeameter and clog the specimen.
4. Determine sample permeability using fresh water. However, some soil remain trapped in the voids.
5. Repeat steps 3 and 4 until the minimum permeability is attained. Steps 1 to 4 constitute one loading cycle.
6. Leave specimen to dry overnight, then remove the soil from specimen by using a vacuum cleaner.
7. Carry out permeability test by using the fresh water as permeant.
8. Repeat steps 3 to 4 until the terminal permeability is reached.

In the analysis, the coefficient permeability values are not calculated but the permeability is indicated by the value of time taken for water to drop between two designated points on the permeameter.

6.0 RESULTS AND DISCUSSION

6.1 Marshall Stability

The marshall stability results are shown in Figure 2. The stability of single layer porous asphalt made with SBS bitumen is higher than those prepared using conventional bitumen. The increase of marshall stability of single layer PA by using SBS bitumen is about 12% when compared with conventional porous asphalt. However, mixes prepared using 10 mm maximum aggregate size records the highest stability. The results also indicates that the maximum aggregate size, DBC and binder types influenced marshall stability of single layer porous asphalt.

6.2 Volumetric Properties

The density of single layer porous asphalt is shown in Figure 3. Regardless of binder types, the density of porous asphalt made with maximum aggregate size 20 mm is higher compared to other mixtures. However, SBS modified mixes exhibit higher densities compared to conventional mixes. When SBS binder is used, the average density increases about 0.7%. The density appears to be influenced by maximum aggregate size, DBC and binder type.

Figure 4 shows the voids in mix of all porous asphalt single layer investigated. Voids in mix for porous asphalt made with maximum aggregate size 20 mm is higher compared with other mixtures. In general, SBS mixes exhibit higher VIM.

The voids in mineral aggregate (VMA) for mixes of various maximum aggregate sizes is shown in Figure 5. VMA of all mixes made with maximum aggregate size 10 mm is higher compared with other mixtures.

Figure 6 shows the result of voids filled with bitumen (VFB) for single layer porous asphalt. The VFB for single layer porous asphalt made with maximum aggregate size 10 mm is higher compared with other mixtures. However, VFB of porous asphalt made with SBS mixes is higher than the conventional mixes. The maximum aggregate sizes, DBC and binder types appear to influence the VFB of single layer porous asphalt.

6.3 Resistance to Disintegration

The resistance to disintegration of porous asphalt is evaluated in terms of its abrasion loss and the results are shown in Figure 7. SBS mixes are found to be more resistant against disintegration compared to conventional mixes. Incorporation of SBS binder in a porous mixture can cause the resistance to disintegration to increase by an average of 148.6% when compared with the conventional mixes. This may be attributed to more superior inter-particle adhesion of such mixes. Other factors that appear to influence the resistance to disintegration includes maximum aggregate size, design binder content and binder type. An improvement in resistance to disintegration is noticeable by increasing the binder content and decreasing the maximum aggregate size. For instance, the resistance to disintegration increases by up to 155.2% when porous asphalt are made with maximum aggregate size 10 mm compared to maximum aggregate size 20 mm. For 20 mm maximum aggregate size mixes, the DBC is lower but the porosity is higher. This probably explains the cause for its lower abrasion resistance.

6.4 Indirect Tensile Resilient Modulus.

Figure 8 shows the tensile resilient modulus results of all mixes investigated. The resilient modulus of SBS mixes is higher compared to the conventional mix. The resilient modulus of SBS mix is approximately 14.8% higher when compared with single layer porous asphalt by using conventional mix. The corresponding highest increase of resilient modulus is in the region of 22.5%, which is for mixes made of maximum aggregate size 14 mm. It was found that when the maximum aggregate size increases, the resilient modulus of all mixes investigated also increases except for the 20 mm maximum aggregate size SBS mixes. The resilient modulus of mixes made with maximum aggregate size 20 mm increases by about 11.8% when compared to porous asphalt made with maximum aggregate size 10 mm. However, the resilient modulus of the 14 mm maximum aggregate size SBS mixes exhibit the highest modulus compared to all other maximum aggregate sizes.

6.5 Creep Stiffness Modulus

The results of the creep tests are shown in Figure 9. The creep stiffness modulus of single layer porous asphalt made with SBS mix is higher than those prepared using conventional mix. The creep stiffness modulus SBS mixes increases by about 28.1% when compared with conventional mix. However, the highest increase of creep stiffness modulus is about 58% for mixes made with 10 mm maximum aggregate size. The creep stiffness modulus of 14 mm maximum aggregate size mix is the highest.

6.6 Permeability

Figure 10 shows the permeability results of all mixes tested. The coefficient of permeability of SBS mixes is lower compared to conventional mixes. From Figure 10, the permeability of SBS mixes are on average 10.1% lower than conventional mix. However, this value decreases when the maximum aggregate size decreases. The decrease in permeability of porous asphalt made with maximum aggregate size 10 mm is about 36.1% when compared with mixes made with maximum aggregate size of 20 mm. However, in terms of maximum aggregate size, the permeability of SBS and conventional mixes are not significantly different. Increasing the maximum aggregate size and decreasing the DBC increases the porosity. Higher porosity generally leads to higher permeability which is a valuable property of porous asphalt. However, this must be trade-off with specimen strength and durability.

6.6.1 Permeability of Clogged Specimens

In the clogging test, permeability is measured in terms of time rather than the coefficient of permeability. The results of the sieve analysis for soils A and B are shown in Table 2. In Soil A, 100% passes the 2.36 mm sieve while 97.8% passes the 0.075 mm sieve. For Soil B, the corresponding values are 100% and 46.9%. The results indicate that soil A is of the clayey fraction while soil B is in the silty range. Preliminary investigation showed that the specimen clogged in a short time when only Soil B is used as the clogging agent. But it takes very much longer to clog the specimen when Soil B is used because Soil B is much coarser compared to Soil A. It is for this reason that it is necessary to select suitable combinations of both soils so that the rate of clogging is acceptable. Table 4 shows the permeability results as a consequence of clogging specimen using varying proportions of Soils A and B. When both soils are combined in the ratio 57:43, the resultant permeability falls below the minimum value. At this combination the percentage passing 0.075 mm sieve is 75.9% and it easily clogged the specimen. However, the permeability as a consequence of adding 10 g is higher compared with addition of 25 g soil. Next, the proportion was changed to 85:15. This proportion results in higher permeability and the percentage passing the 0.075 mm sieve is 90.1%. It was found that using both proportions, the amount of trapped soil is approximately 10 g as shown in Table 5. Therefore, the maximum mass of soil to be used as permeant shall be a maximum of 10 g when the proportion of Soil A and B is 85:15. From the clogging test results shown in Table 5, the ability of a specimen to trap soils increases when the maximum aggregate size decreases. In this case, porosity plays an important role.

Figure 11 shows the permeability of single layer porous asphalt using various trapped soil removal methods. The methods tried out include brushing, spraying and vacuum cleaning. It appears that the permeability after removing soil from specimen is only 47% of the initial permeability when a vacuum cleaner is used. Nevertheless, when water sprayer and brushing is used, the corresponding values are 33% and 32%

respectively. The results indicate that the residual permeability using vacuum cleaner is the highest compared to the other methods. This indicates that the vacuum cleaner can remove trapped soil from specimen better compared to the rest and is adopted in future investigations. However, the mixes made of maximum aggregate size 20 mm has the higher permeability for all methods used.

To determine the cycle time, preliminary trials were again conducted using 10 g soil. The soil was mixed in 5 liters of water which is the volume of permeameter tube or 2.0 g/liter of concentration. The preliminary tests were conducted only on mixes with maximum aggregate size 20 mm prepared with conventional binder. The results are displayed in Table 6. It shows that the mixes made with maximum aggregate size 20 mm clogged after 2 loading times. The test was then continued with a lower 1.0 g/litre concentration. The specimen was not clogged after 7 loading cycles. For this reason, a permeant concentration of 1.5 g/liter water was adopted in future clogging tests.

The cycles of reduction and increase in permeability upon vacuuming is shown in Figures 12 through 17. For all maximum aggregate sizes, SBS mixes clogged faster compared to conventional mixes. For cycle 1, it requires an average of 3 and 5 loading numbers to attain minimum permeability for SBS and conventional mixes. In terms of number of cycles, mixes made with maximum aggregate sizes 14 and 10 mm attain the minimum permeability after 4 cycles but for maximum aggregate size 20 the corresponding value is 5 and 4 respectively for conventional and SBS mixes. The number of cycles and time for loading are important indicators in the drainage ability of porous asphalt. The resultant permeability just upon vacuuming differs significantly from the terminal permeability. After cleansing, the average permeability, measured in units of seconds of mixes made of conventional binder and SBS mixes is about 100 s and 108.3 s respectively. However, mixes made of maximum aggregate size 20 mm exhibits the highest permeability when drainage time is about 90 sec.

7.0 CONCLUSIONS

1. Addition of SBS modified binder improves properties of porous asphalt the stability, resistance to disintegration, indirect tensile resilient modulus and creep stiffness modulus of porous asphalt.
2. The properties of single layer porous asphalt such as permeability, resistance to abrasion loss and indirect tensile resilient modulus decreases when the maximum aggregate sizes in single PA decreases.
3. The permeability of SBS mix is lower compared to conventional mix.
4. The duration and number of cycle for clogging porous asphalt made with SBS mix is slightly shorter compared to conventional mix.
5. The maximum aggregate size increases, the clogging duration also increases.

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Table 1 Mixing and Compaction Temperatures

Binder Type	Temperature (°C)	
	Mixing	Compaction
Conventional Binder Pen 60/70	140	130
SBS Binder	180	160

Table 2 Sieve Analysis of Soils as Clogging Material

Sieve (mm)	Soil A	Soil B
	% Passing	% Passing
2.36	100	100
1.18	99.52	94.23
0.6	99.21	83.96
0.3	98.80	67.36
0.15	98.19	52.45
0.075	97.75	46.94

Table 3 Soil Combination Used for Clogging Material

Trial I

# Sieve mm	Agg. A % passing	57%	Agg. B % passing	43%	A+B (%)
2.36	100	57	100	43	100
1.18	99.5	56.715	94.2	40.506	97.221
0.6	99.2	56.544	84	36.12	92.664
0.3	98.8	56.316	67.4	28.982	85.298
0.15	98.2	55.974	52.4	22.532	78.506
0.075	97.7	55.689	46.9	20.167	75.856

Trial II

# Sieve (mm)	Agg. A % passing	85%	Agg. B % passing	15%	A+B (%)
2.36	100	85	100	15	100
1.18	99.5	84.575	94.2	14.13	98.705
0.6	99.2	84.32	84	12.6	96.92
0.3	98.8	83.98	67.4	10.11	94.09
0.15	98.2	83.47	52.4	7.86	91.33
0.075	97.7	83.045	46.9	7.035	90.08

Table 4 Permeability for Various Amount and Soil Proportion

Max. Aggregate Size (mm)	Permeability (cm/sec)			Soil Proportion (%)
	Without soil	at 10 g soil	at 25 g soil	
20	0.157	0.069	0.010	Soils A 57 Soils B 43
14	0.130	0.033	0.008	
10	0.127	0.018	0.005	
20	0.162	0.083	0.018	Soils A 85 Soils B 15
14	0.129	0.052	0.018	
10	0.124	0.046	0.022	

Table 5 Amount of Soil in Specimen Voids at Various Soil Proportion

Max. Aggregate Sizes (mm)	Soils (g)				Soil Proportion (%)
	Total	Retained	Passing	In voids	
20	25	4.81	8.60	11.59	Soils A 57 Soils B 43
14	25	7.4	7.59	10.01	
10	25	13.13	2.80	9.07	
20	25	0.52	16.97	7.51	Soils A 85 Soils B 15
14	25	2.23	12.09	10.68	
10	25	3.36	10.03	11.61	

Table 6 Time of Soil Loading of Specimen at Two Soil Concentrations

Soil Concentration (g/liter)	Time of Soil Loading (s)							
	to	t1	t2	t3	t4	t5	t6	t7
1.0	58.28	74.25	86.3	106.9	118.3	134.1	116.2	167.0
2.0	58.28	191.4	239.8					

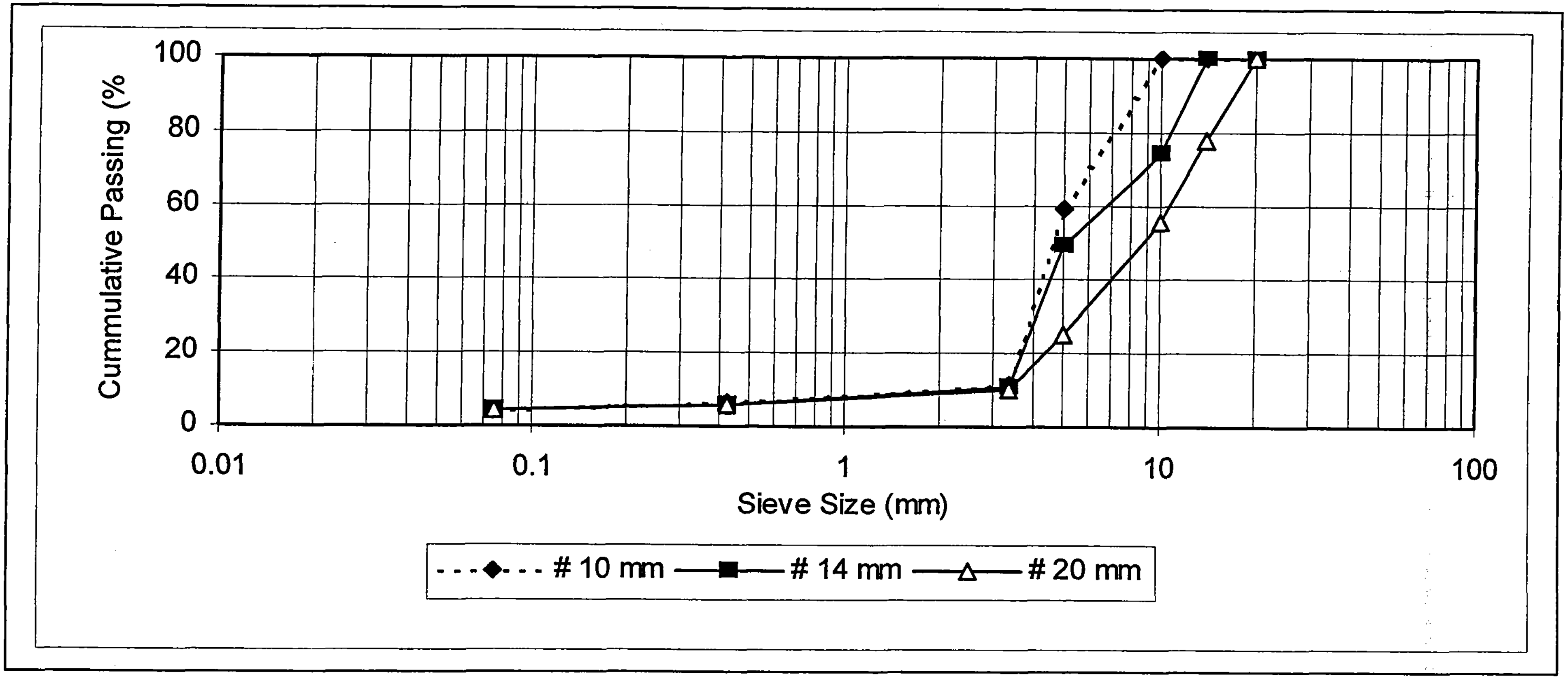

Figure 1 Aggregate Grading Used in This Investigation

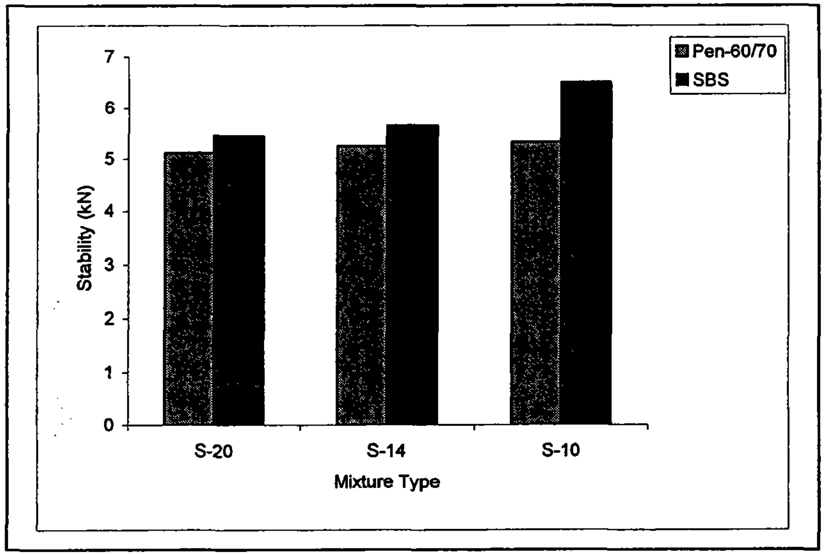

Figure 2 Marshall Stability of Single Layer Porous Asphalt

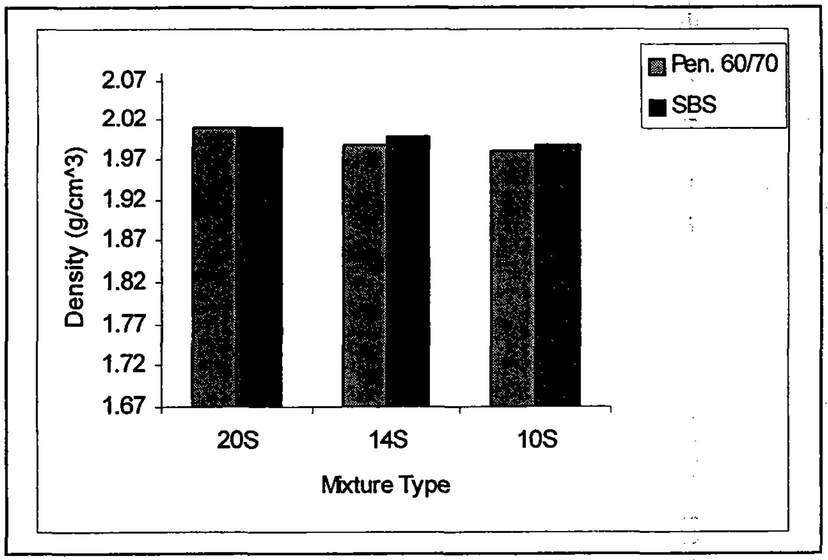

Figure 3 Density of Single Layer Porous Asphalt

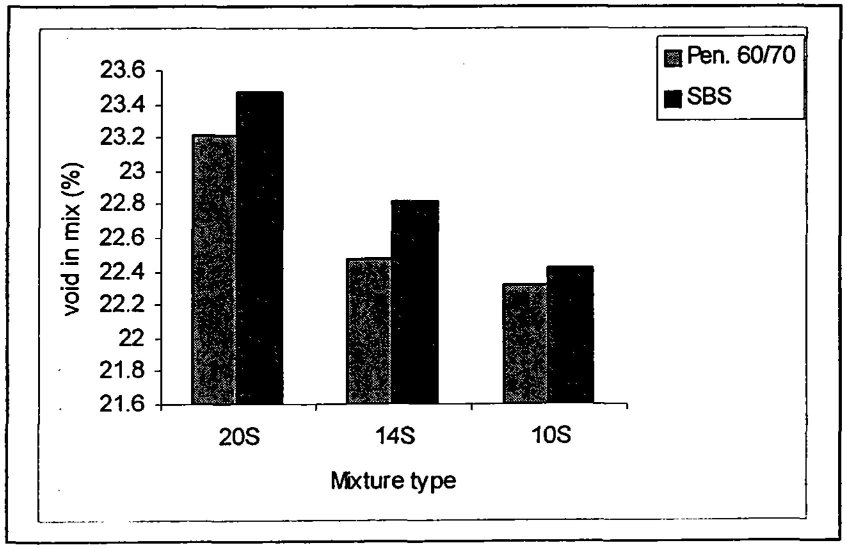

Figure 4 Voids in Mix of Single Layer Porous Asphalt

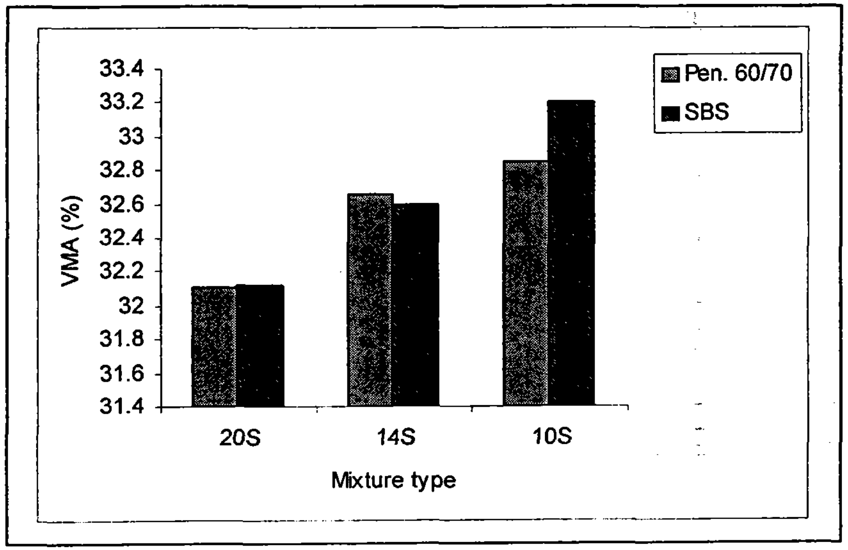

Figure 5 Voids in Mineral Aggregate of Single Layer PA

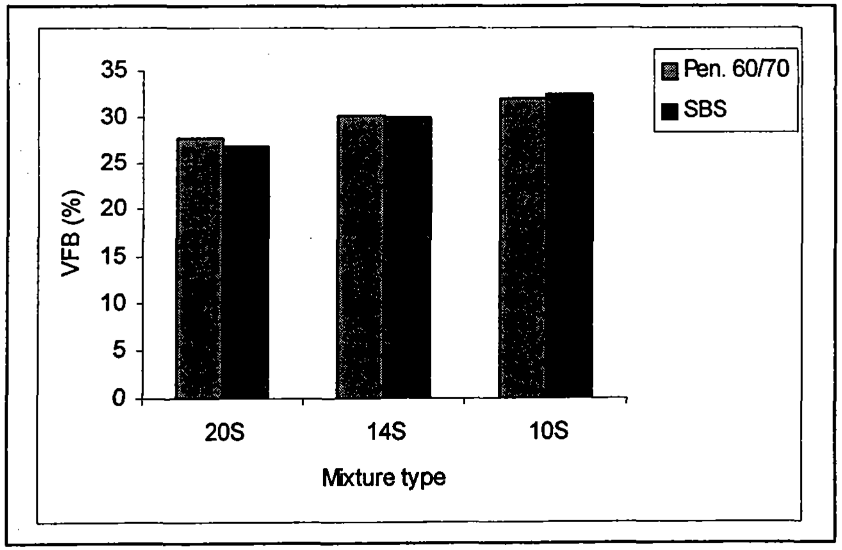

Figure 6 Void Filled with Bitumen of Single Layer PA

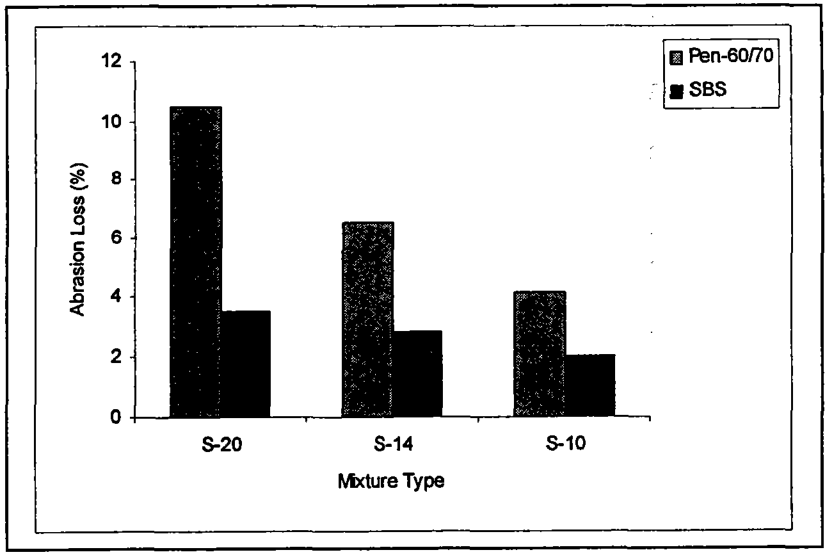

Figure 7 Abrasion Loss of Single Layer Porous Asphalt

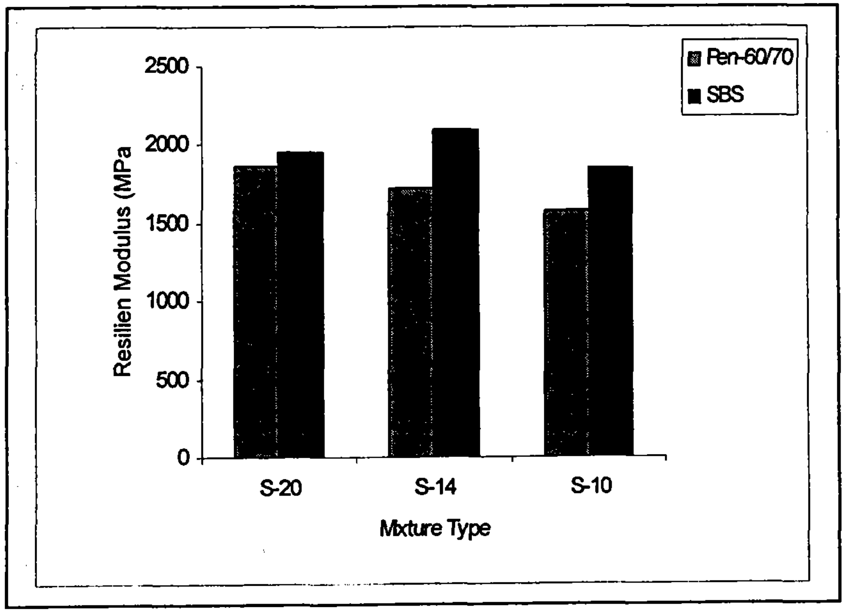

Figure 8 Resilient Modulus of Single Layer Porous Asphalt

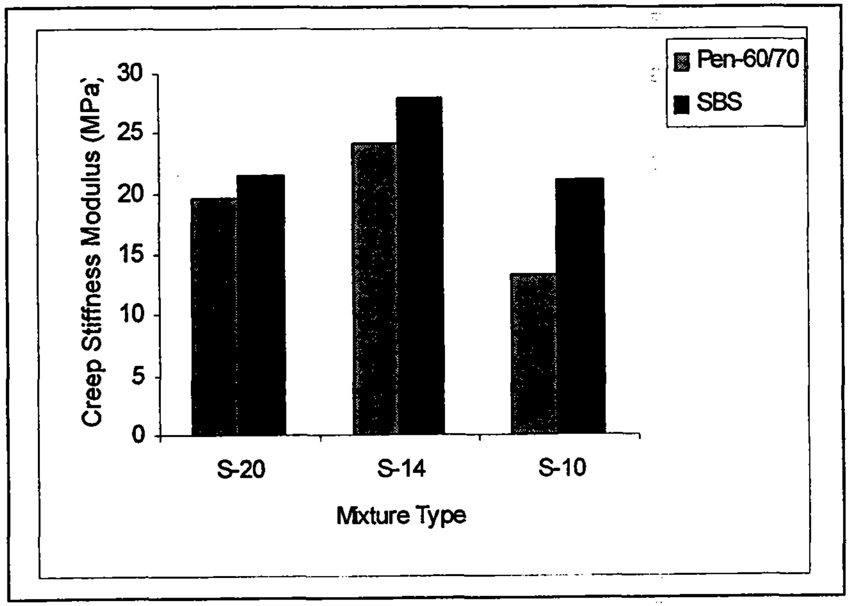

Figure 9 Creep Stiffness Modulus of Single Layer PA

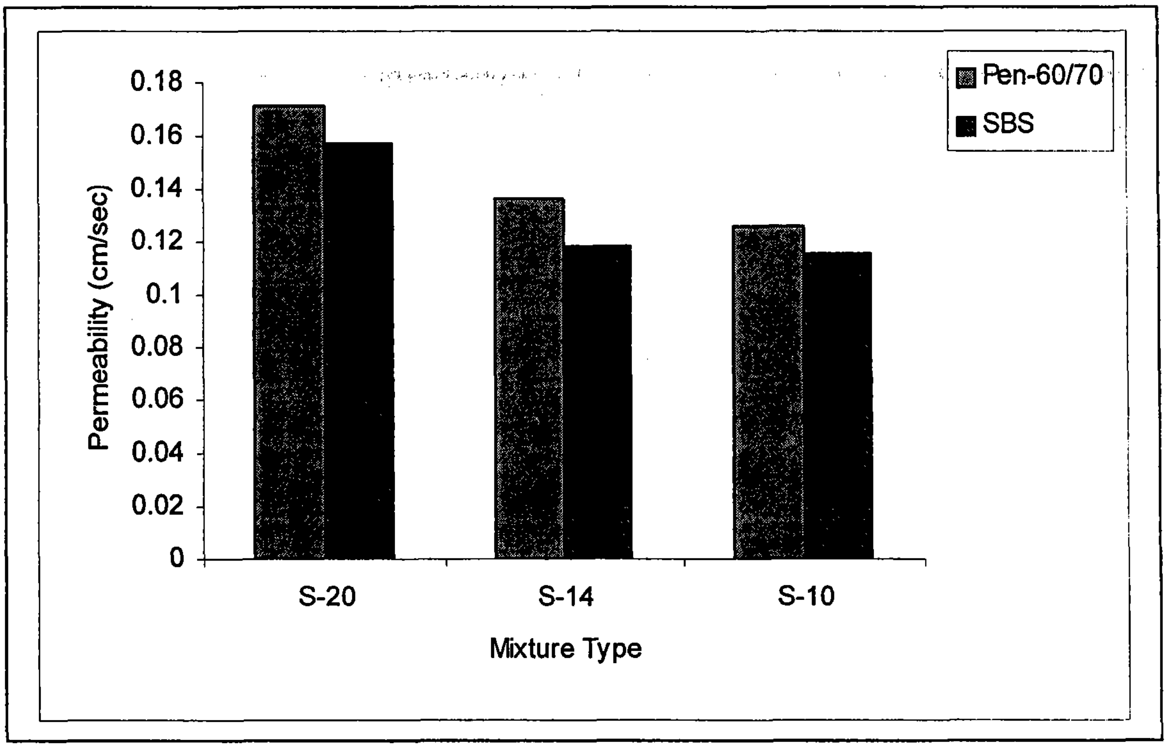

Figure 10 Permeability of Single Layer Porous Asphalt

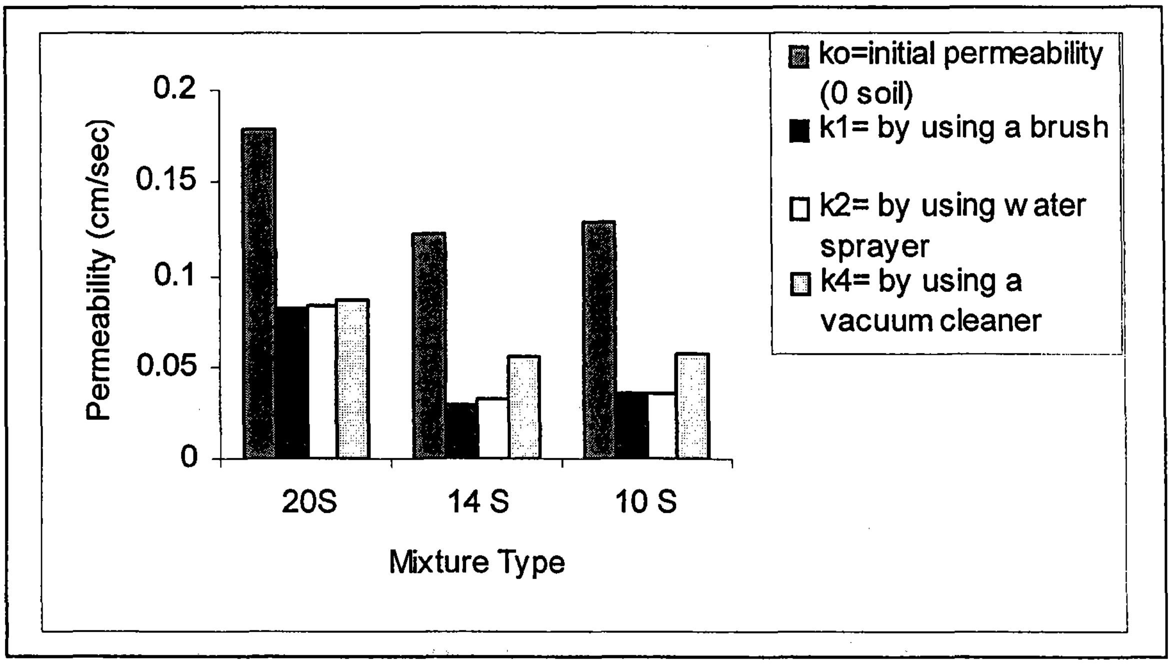

Figure 11 Permeability after Removal Soil from Specimen by Various Methods

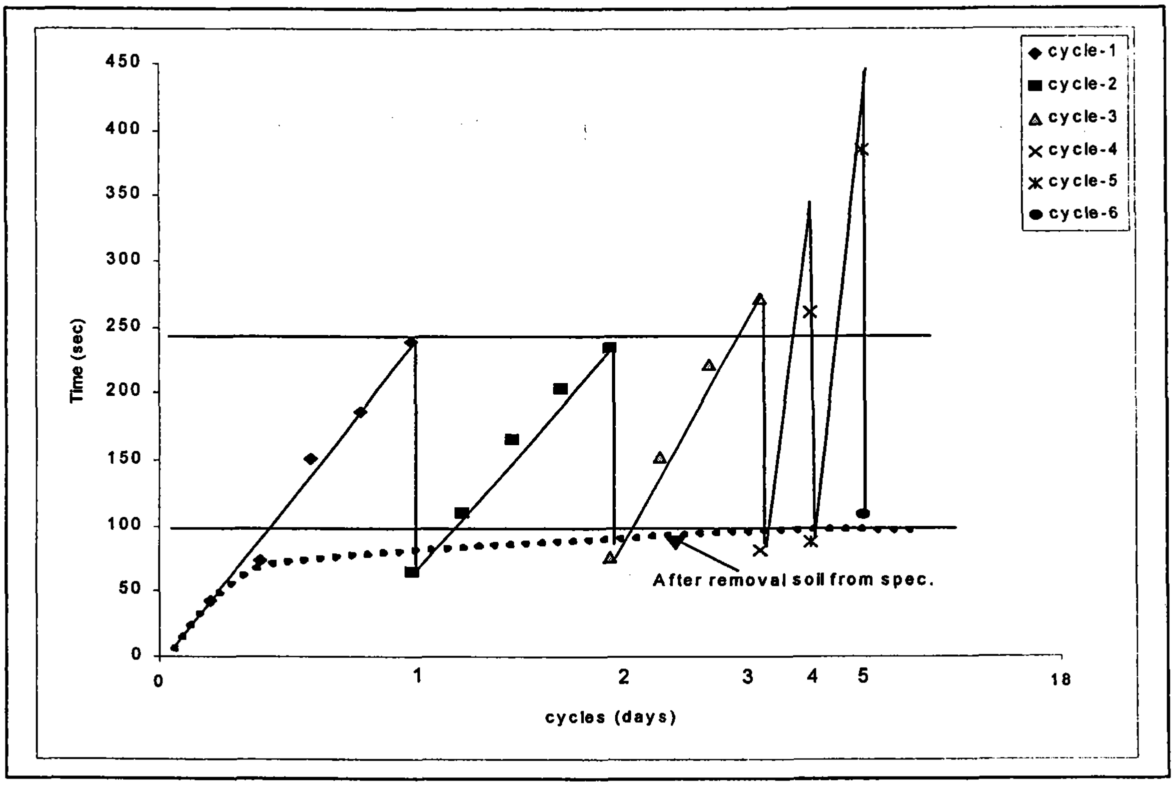

Figure 12 Cycle Time for Clogging of Single Layer PA Made of Maximum Aggregate Size 20 mm and Conventional Binder

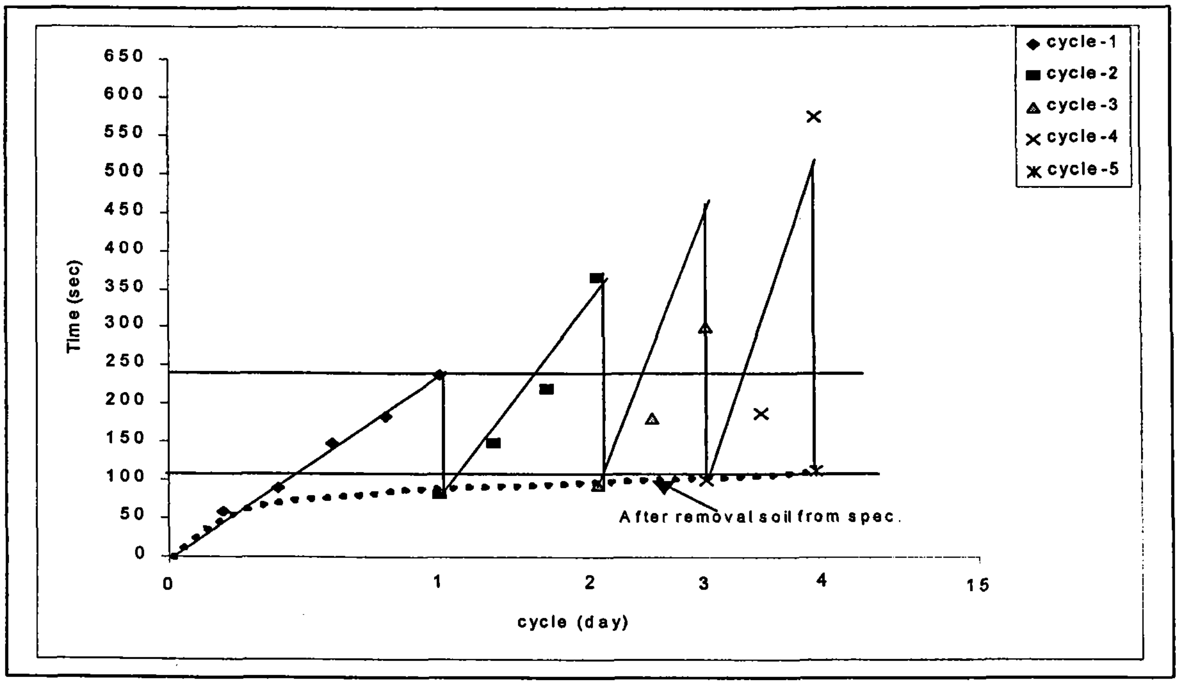

Figure 13 Cycle Time for Clogging of Single Layer PA Made of Maximum Aggregate Size 14 mm and Conventional Binder

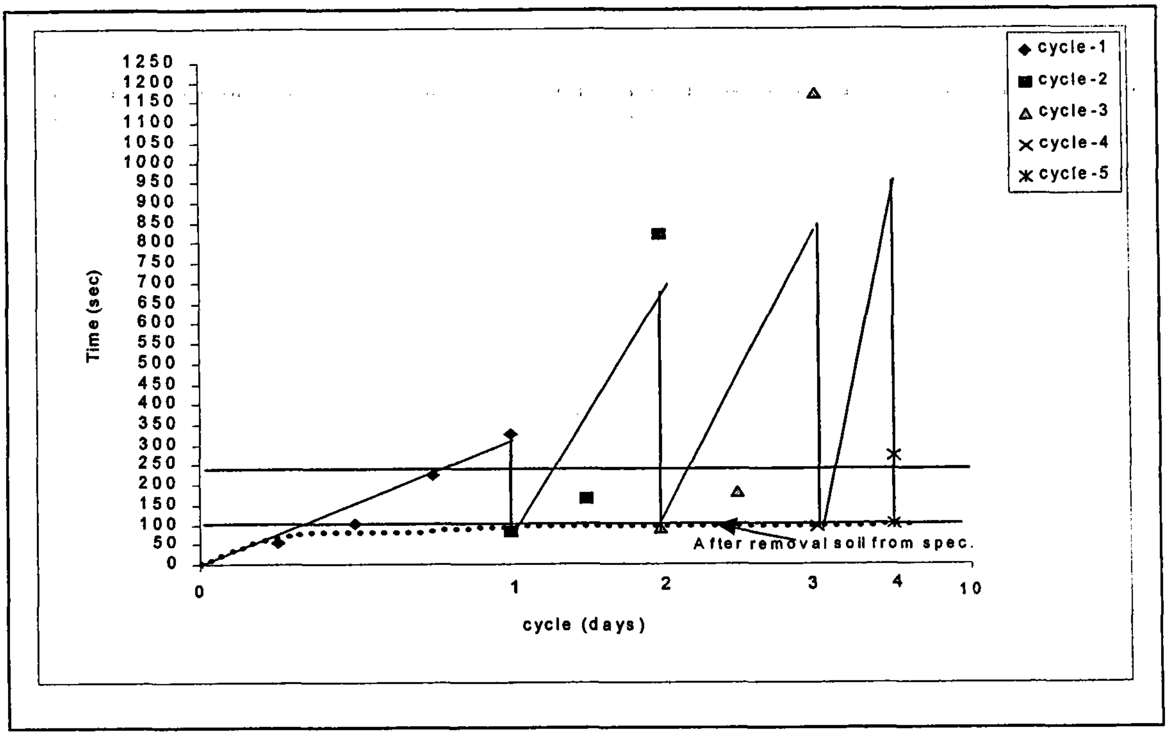

Figure 14 Cycle Time for Clogging of Single Layer PA Made of Maximum Aggregate Size 10 mm and Conventional Binder

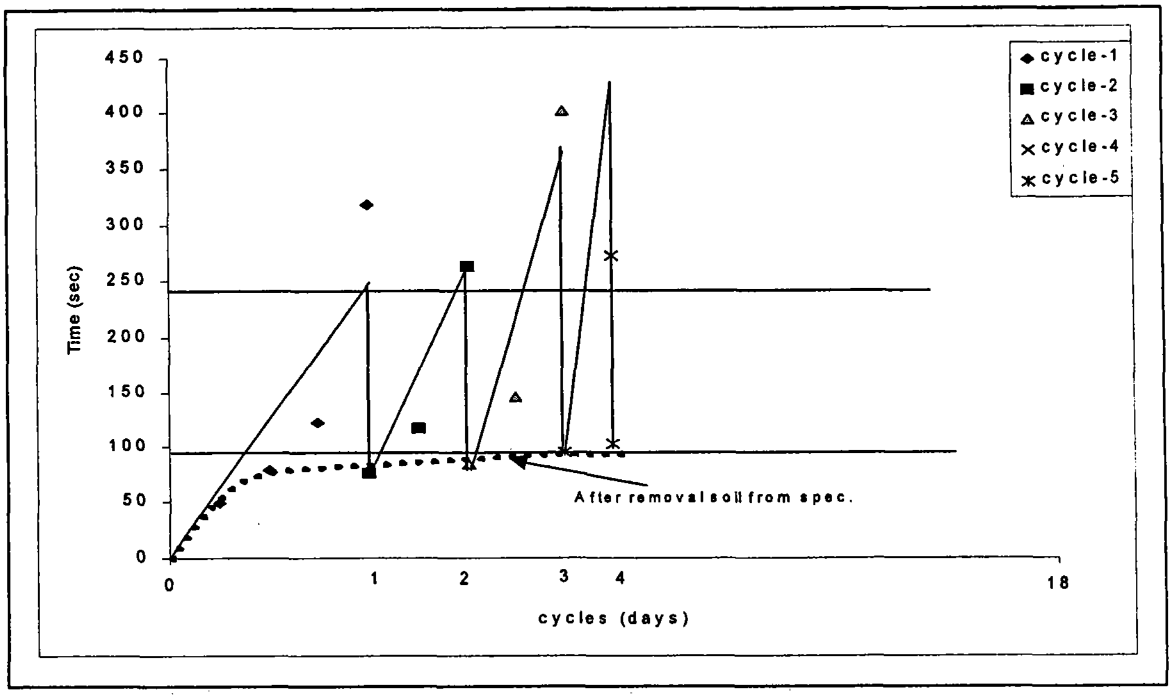

Figure 15 Cycle Time for Clogging of Single Layer PA Made of Maximum Aggregate Size 20 mm and SBS Binder

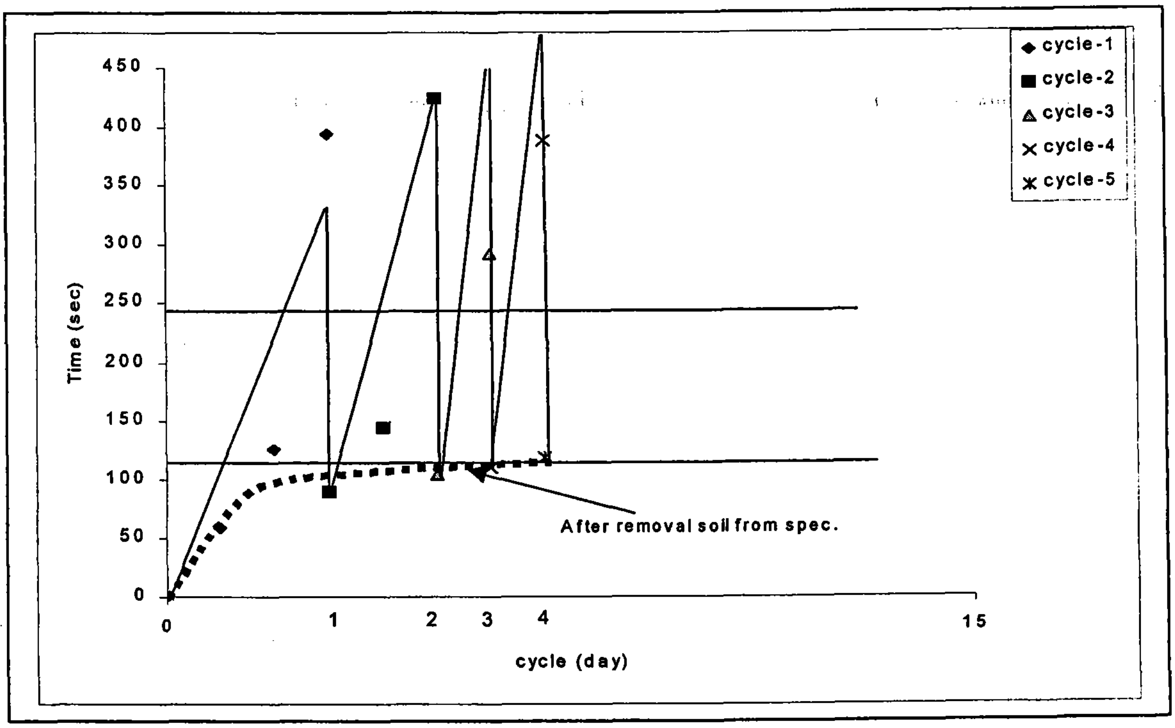

Figure 16 Cycle Time for Clogging of Single Layer PA Made of Maximum Aggregate Size 14 mm and SBS Binder

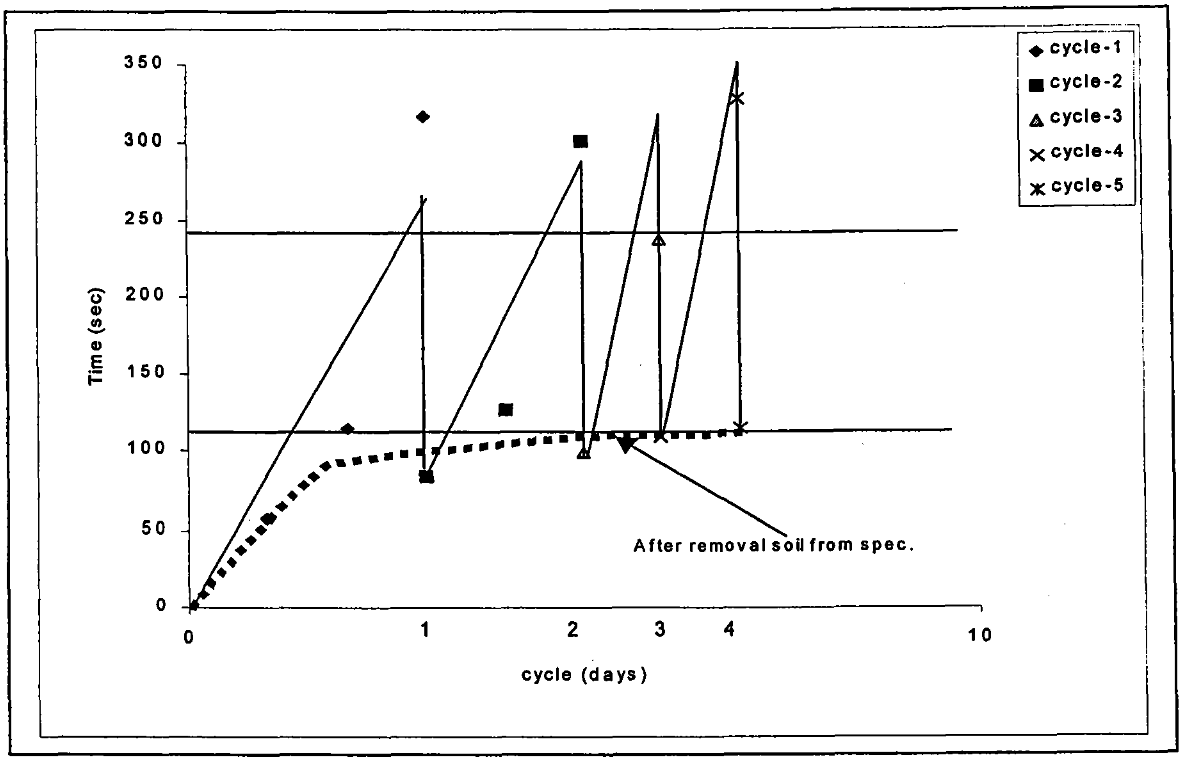

Figure 17 Cycle Time for Clogging of Single Layer PA Made of Maximum Aggregate Size 10 mm and SBS Binder

ENGINEERING PROPERTIES AND PERFORMANCE OF DOUBLE LAYER POROUS ASPHALT

REPORT-5

JULY 2004

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1.0 INTRODUCTION

Over time, the permeability of porous asphalt reduces due to clogging. Porous surfacings are normally vacuumed twiĉe a year to unclog the pores. Despite this maintenance effort, permeability remains low and permeability loss remains.

The search for a solution to mitigate permeability loss has led to the development of double layer porous asphalt. In the Netherlands, it is known as Twinlay. The concepts underlying the design of Twinlay was first reported at the 1st Eurasphalt and Eurobitume Congress in 1996 held at Strasbourg. From the perspective of resistance to permeability loss, the two-layered porous asphalt construction has the potential to better resist clogging compared to conventional single layer porous asphalt. Several roads in Netherlands are paved with this type of construction. The two-layered concept proved to be effective in reducing traffic noise. Additional benefits over and above those from conventional porous asphalt layer were also observed. This includes the water discharge capacity and the ease of cleaning maintenance.

2.0 OBJECTIVE

The objective of the research program is to determine the engineering properties and performance of double layer porous asphalt incorporating conventional binder and polymer modified binder (SBS).

3.0 SCOPE OF WORK

The scope of work during this stage of the research focus on the performance of double layer porous asphalt, each layer made with top layer 10 mm, 14 mm and base layer 20 mm maximum aggregate size. Performance is measured in terms of Marshall stability, permeability, resistance to disintegration, indirect tensile modulus and creep stiffness modulus.

4.0 LITERATURE REVIEW

4.1 Double Layer (Twinlay) Porous Asphalt (PA)

According to Van Borchove (1996), the two-layered concept proved to be effective not only in reducing traffic noise, but also additional benefits over and above those of conventional PA layers. Double-layer PA has a large drainage capacity, not so quickly contaminated as single layer PA and if clogged double layer PA are relatively easier to clean.

In the Netherlands, the top layer of PA 2/4 is a refined version of the standard twinlay. The percentage of modified binder in the top layer is somewhat higher than for normal twinlay. The base layer of twinlay M consists of PA 11/16 and has a thickness of 4.5 cm, similar to ordinary twinlay (Van Bochove, 2000).

Next, twinlay has in comparison with conventional dense asphalt concrete, the following advantages:

1. The fine top layer offers acoustic advantages through the fine surface texture (reduction tire noise). The coarse bottom layer provides in combination with the fine top layer also required damping of sound.
2. The top layer prevents coarse dirt or temporary a large a mount of dirt from entering in to the construction.
3. Dirt, which nevertheless penetrates in the wearing course, can be removed with less effort. The dirt is absorbed at the top of the thin top layer, from where it can easily be removed with existing cleaning techniques.
4. The difference in airflow resistance between the top and bottom layer has a positive effect on the self-cleaning capacity caused by traffic.
5. The bottom layer is more permeable compared to conventional porous asphalt through which the sideways discharge of water improves considerably.

An investigation by Battiato (1996) on the properties of double layer draining (DDL) mix made of 1.5 cm thick for upper layer and 3 cm for bottom layer can be seen in Table 1.

Table 1 The characteristics of Upper and Lower Layer for PA

No.	Test Type	Standard	Unit	Values	
				Upper	Lower
1.	Marshall stability	CNR 30/1973	da N	475	460
2.	Marshall flow	CNR 30/1973	mm	2.35	2.08
3.	Marshall stiffness	CNR 30/1973	da N/mm	202	222
4.	Indirect tensile strength at 25 C	CNR 134/1991	da N/cm^2	6	5
5.	Indentation test at 60 C	DIN 1996	mm	1.08	-
6.	% of mix voids	CNR 39/1973	%	25	22
7.	% of bitumen on agg.	CNR 38/1973	%	5	4.5

5.0 METHODOLOGY

5.1 Materials

Materials used namely aggregate, filler and binders were as described in Report 1.

5.2 Preparation of Porous Asphalt Specimens at DBC

To determine the properties of double layer porous asphalt, specimens were prepared in a similar manner to that described in Report 1.

Twinlay is made up of the top and base layer. The maximum aggregate size for the base layer is 20 mm while two maximum aggregate sizes 14 and 10 mm will be used for the top layer. The development of their gradations has been described in Report 1. The relative thickness of top and bottom layers (T/B) in a specimen are 0/70(S); 30/40

(D1); 20/50 (D2) and 15/50 mm (D3) or schematically shown in Figure 1. The binder contents used for all mixes are based on their respective design binder content.

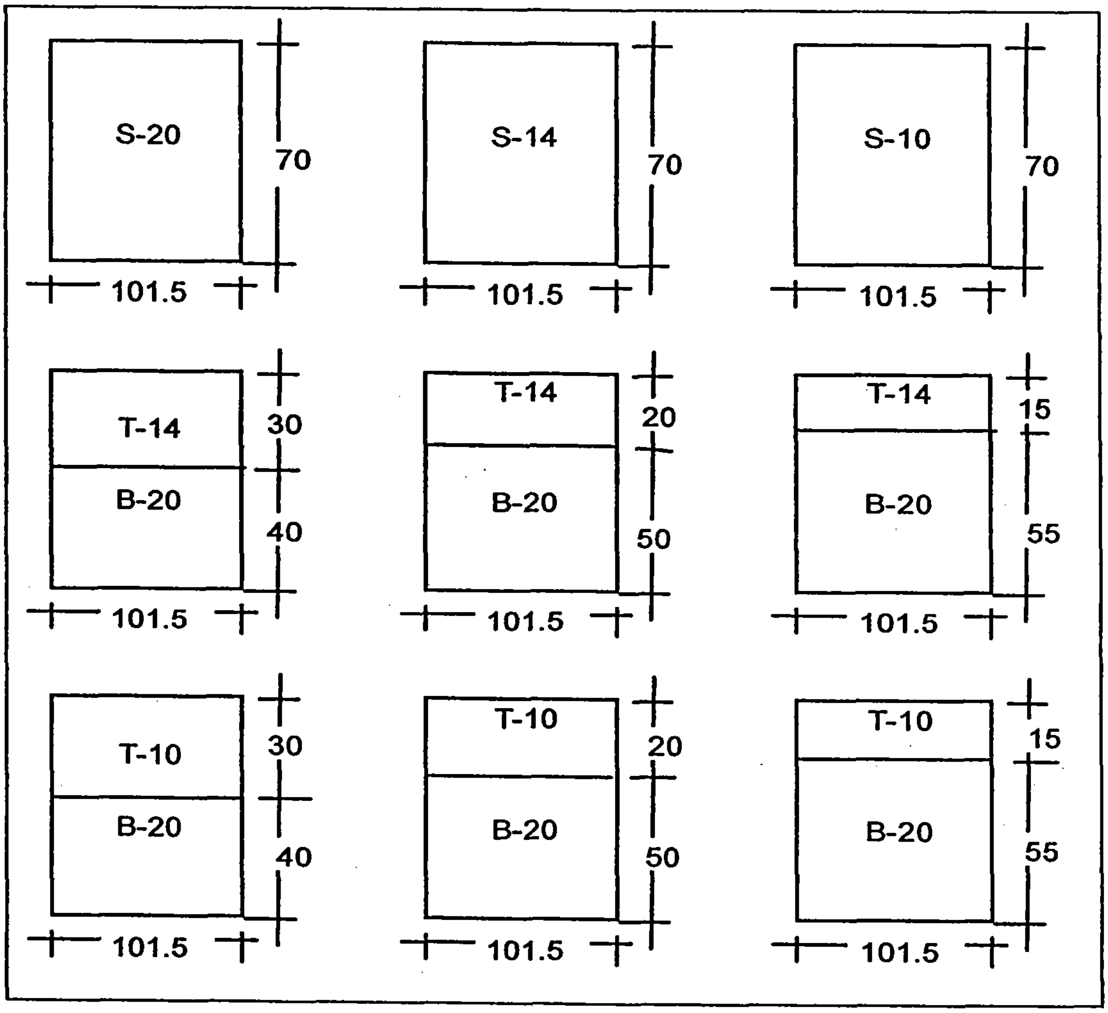

Figure 1 Schematic Diagram Showing Relative Thickness of Double Layer Porous Asphalt

The preparation of 70 mm thick for double layer porous Marshall specimen is similar with the preparation of Marshall specimen for single layer porous asphalt. But, the amount of aggregate for top and base layer are calculated based on the Marshall density of single layer. Knowing the density and volume, the actual amount of mix required can be calculated. The relationship between density and binder content used for this estimation for conventional and SBS binder are shown Figures 2 and 3. By interpolation, the densities at DBC for maximum aggregate sizes 20, 14 and 10 mm are 2.0; 1.98; 1.97 g/cm^3 and 2.01; 1.98; 1.96 g/cm^3 respectively for conventional and SBS mixes.

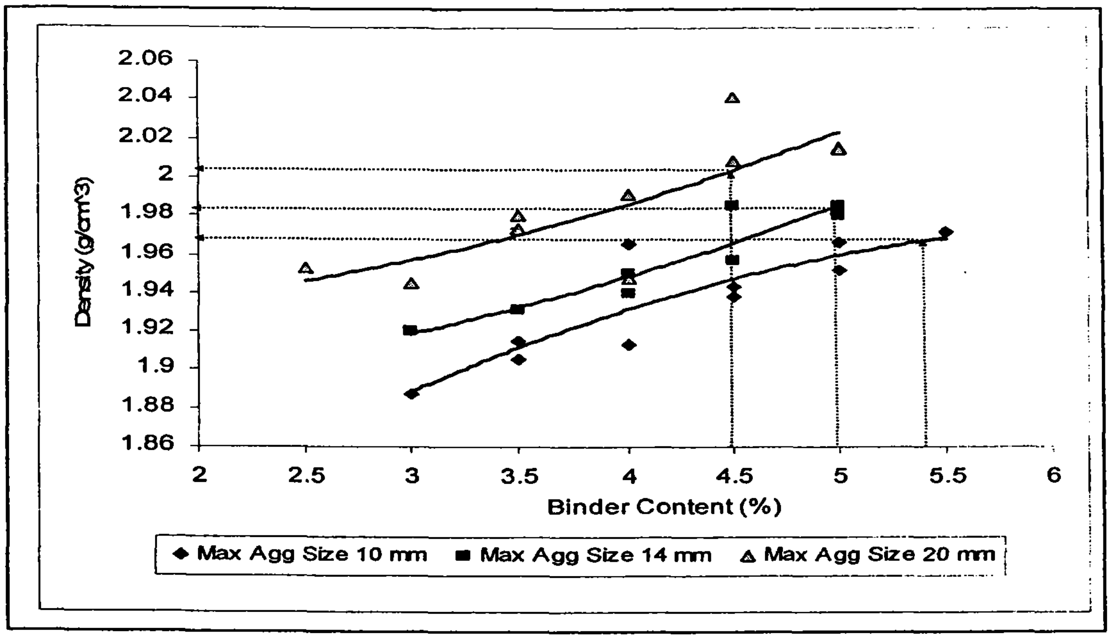

Figure 2 Density of Single Layer Porous Asphalt at Various Binder Contents (Pen 60/70)

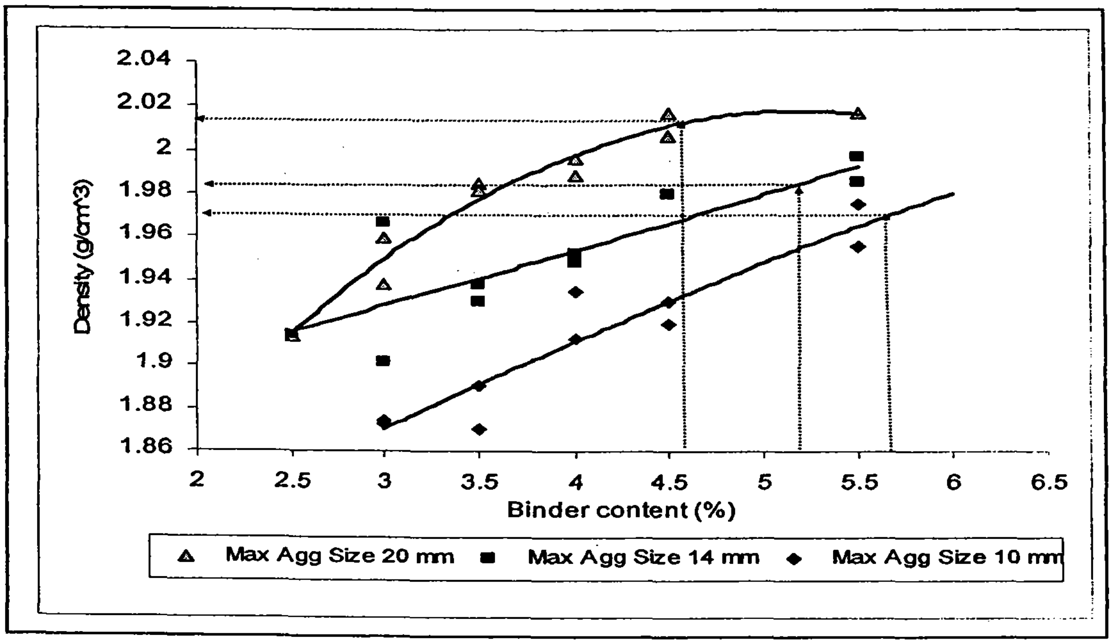

Figure 3 Density of Single Layer Porous Asphalt at Various Binder Contents (SBS)

Two methods were explored to assess the suitability of compacting cylindrical double layer porous asphalt specimen in the Marshall mould.

In Method A, compaction was carried out in 2 steps at 2x50 blows. The compaction procedure is as follows:

- The base layer mix ingredients were prepared for mixing. The required quantity of mix was placed inside the Marshall mould and the whole assembly was then transferred into the oven which was earlier set at the destined compaction temperature.

- For the next step, the porous mix for the top layer was prepared. Once ready, the calculated amount of mix was then transferred into the Marshall mould and placed on top of the base layer mix.
- The mix was then compacted at 1x50 blows each face.

The steps involved in compaction Method A is described in Figure 4.

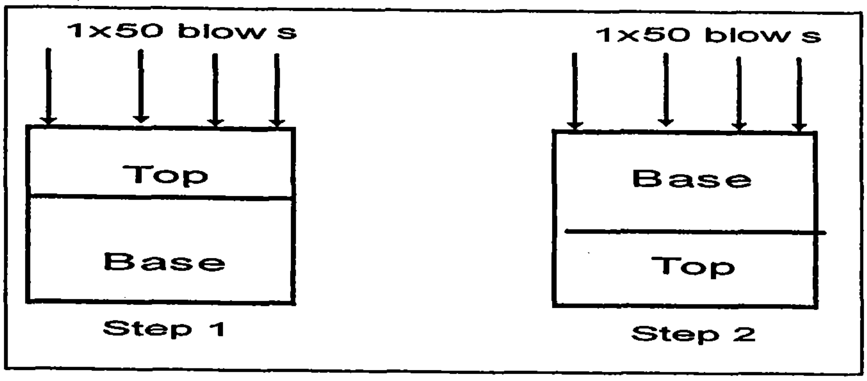

Figure 4 Compaction Procedure in Accordance With Method A

In Method B, compaction was accomplished in 3 steps involving (1x25+1x25+1x50) blows. The detailed compaction procedure is outlined underneath:

- The required quantity of base layer mix was prepared, mixed, placed inside the Marshal mould and then subjected to 1x25 blows. As in Method A, the whole assembly was then kept inside an oven that has been set at the compaction temperature.
- Then, top layer mix was prepared, mixed and a known quantity was placed on top of the partially compacted base layer inside the Marshall mould.
- The top layer mix overlying the base layer mix was then subjected to 1x25 blows of the Marshall hammer. Upon completion, the mould was turned upside down and compacted at 1x50 blows.

The steps of the compaction procedure described in Method B is shown in Figure 5.

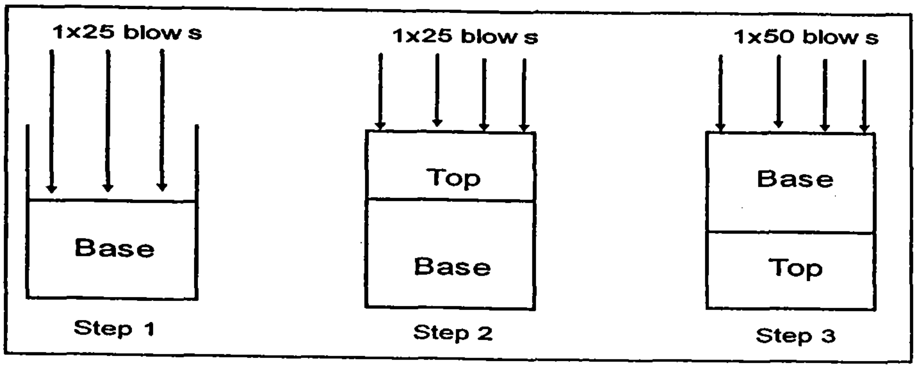

Figure 5 Compaction Procedure in Accordance With Method B

Compacted specimens were tested for permeability and stability.

5.3 Tests for Double Layer Porous Asphalt at DBC

5.3.1 Permeability Test
Permeability was measured using a water permeameter as described in Report 4.

5.3.2 Marshall Stability
Marshall stability was measured using the Marshall testing machine as described in Report 4.

5.3.3 Resistance to Disintegration
The resistance to disintegration was evaluated using the Los Angles drum as described in Report 4.

5.3.4 Indirect Tensile Resilient Modulus
The indirect tensile resilient modulus was evaluated using the MATTA machine as described in Report 4.

5.3.5 Creep Test
The creep test was evaluated using the MATTA machine as described in Report 4.

6.0 RESULTS AND DISCUSSION

6.1 Compaction Method
The Marshall stability and permeability of specimens compacted using Method A and B is shown in Table 2. The compaction procedure adopted shall be based on the results shown in Table 2.

Table 2 Marshall Stability and Permeability for Double Layer PA at Various Compaction

Mixture Type	Marshall Stability (kN)		Permeability (cm/s)	
	A*	B*	A*	B*
Top Layer 30 cm (Ag 14)	4.97	5.48	0.127	0.126
Top Layer 20 cm (Ag 14)	5.41	6.25	0.133	0.134
Top Layer 15 cm (Ag 14)	5.98	5.83	0.145	0.142
Top Layer 30 cm (Ag 10)	4.71	4.48	0.135	0.133
Top Layer 20 cm (Ag 10)	5.95	5.88	0.135	0.133
Top Layer 15 cm (Ag 10)	5.5	5.44	0.138	0.139
Single Layer (Ag 10)	5.34	-	0.126	-
Single Layer (Ag 14)	5.26	-	0.136	-
Single Layer (Ag 20)	5.12	-	0.171	-

Method A*= 2x50 blows
Method B*=1x25+1x25+1x50 blows

Table 2 shows that the Marshall stability and permeability values for double layer porous asphalt compacted using Methods A and B do not differ significantly. The maximum stability difference between both compaction methods are approximately 7.3% and 2.2% respectively for stability of top layer made with maximum aggregate sizes 14 and 10. The corresponding difference in permeability is about 0.74% and 0.73%. Based on these results, method A was adopted to compact double layer porous asphalt. Compaction Method A at 2x50 blows was thought to result in uniform of density between top and bottom layers. More importantly, compaction Method A eliminates the requirement for minimum top layer thickness in relation to maximum aggregate size. However, double layer prepared using 10 and 14 mm maximum aggregate sizes and 12 and 30 cm top layer thickness exhibit higher stability than the corresponding single layer. However, the 20 mm maximum aggregate size single layer records the highest permeability.

6.2 Permeability

Figures 6 and 7 show the permeability results of all mixes tested. The coefficient of permeability of SBS mixes is lower compared to conventional mixes. From Figures 6 and 7, the permeability of SBS mixes are on average 2.2% and 6.0% lower than conventional mix respectively for top layer made of maximum aggregate sizes 14 and 10 mm. However, this value increases when the top layer thickness decreases. The highest permeability is 0.147 cm/s was measured for mix made of 15 cm top layer thickness and 10 mm maximum aggregate size. The average permeability of double layer mix is higher compared to 14 and 10 mm single layer mix but not single layer 20 mm. However, in terms of maximum aggregate size, the permeability of SBS and conventional mixes are not significantly different. Increasing the maximum aggregate size and decreasing the DBC increases the porosity. Higher porosity generally leads to higher permeability which is a valuable property of porous asphalt. However, this must be trade-off with specimen strength and durability.

6.3 Volumetric Properties

The density, porosity, voids filled with bitumen (VFB) and voids in mineral aggregate (VMA) of double layer porous asphalt are not calculated because of the difficulty to determine the geometry and weight of each layer. However, the properties of each layer have been discussed in Report 4.

6.4 Marshall Stability

The Marshall stability results are shown in Figures 8 and 9. The stability of double layer porous asphalt made with SBS bitumen is higher than those prepared using conventional bitumen. The increase of marshal stability of double layer PA by using SBS and maximum aggregate size 14 and 10 mm is about 14.7% and 15.3% when compared with conventional porous asphalt. However, mixes prepared using 10 mm maximum aggregate size, SBS binder and 30 cm top layer thickness records the highest stability. The average stability value of double layer mix is higher compared to all single layer 10, 14 and 20 mm except for 10 mm SBS single layer mix. The SBS double layer mix has the highest increase in stability by as much as 14.1% when compared to single layer made of maximum aggregate size 20 mm. All mixes incorporating SBS binder and 14 mm top layer maximum aggregate size results in stability above 6.0 kN. The results also indicate that the maximum aggregate size, the relative thickness of each layer, DBC and binder types influenced Marshall stability of double layer porous asphalt.

6.5 Resistance to Disintegration

The resistance to disintegration of porous asphalt is evaluated in terms of its abrasion loss and the results are shown in Figures 10 and 11. SBS mixes are found to be more resistant to disintegration compared to conventional mixes. Incorporation of SBS binder in a porous mixture can cause the resistance to disintegration to increase by an average of 51.1% and 55.3% when compared with the conventional mixes respectively for top layer made of maximum aggregate size 14 and 10 mm. This may be attributed to more superior inter-particle adhesion of such mixes. However, mixes prepared using 10 mm maximum aggregate size, SBS binder and 30 cm top layer thickness records the highest resistance to disintegration. The abrasion loss of all mixes prepared using SBS and base binders respectively experienced 1.4 to 4.5% and 5.7 to 7.76% abrasion loss. An improvement in resistance to disintegration is noticeable by increasing the binder content and decreasing the maximum aggregate size. For instance, the resistance to disintegration increases by up to 75.9% when porous asphalt are made with SBS binder and compared to base bitumen at top thickness layer 30 cm. The resistance to disintegration decreases when the thickness of top layer decreases. According to Report 4, the resistance to disintegration for porous asphalt made of maximum aggregate size 10 mm is better than 14 and 20 mm. Double layer prepared using 10 and 14 mm maximum aggregate sizes exhibits abrasion higher than all 14 and 10 mm single layer with the exception of 20 mm single layer mix. The 20 mm single layer is the least resistant to disintegration.

6.6 Indirect Tensile Resilient Modulus.

Figures 12 and 13 show the resilient modulus results of all mixes investigated. The resilient modulus of SBS mixes is higher compared to the conventional mix. The resilient modulus of SBS mix is approximately 3.5% and 6.2 % higher when compared with double layer porous asphalt by using conventional mix respectively for top layer made of maximum aggregate sizes 14 and 10. The corresponding highest increase of resilient modulus is in the region of 7.9% which is for mixes made of top layer thickness 15 cm and maximum aggregate size 10 mm. It was found that when the top layer thickness decreases, the resilient modulus of all mixes investigated also increases. The resilient modulus of mixes made with top layer thickness 15 cm and maximum aggregate size 10 mm is 2076.5 MPa and 1965 MPa respectively for mixes made with SBS binder and base bitumen. However, the resilient modulus of 15 cm top layer thickness and 10 mm top layer maximum aggregate size is the highest. The resilient modulus for double layer made of 10 and 14 mm top layer aggregate sizes and 30 and 20 cm top layer thickness is generally higher compared to the single layer made of maximum aggregate sizes 14 and 10 mm, but this value is generally lower from the single layer made of maximum aggregate size 20 mm.

6.7 Creep Stiffness Modulus

The results of the creep tests are shown in Figures 14 and 15. The creep stiffness modulus of double layer porous asphalt made with SBS mix is higher than those prepared using conventional mix. The creep stiffness modulus of SBS mixes increases by about 19.6% and 43.2% when compared with conventional mix respectively for top layer made of maximum aggregate sizes 14 and 10 mm. However, the highest increase of creep stiffness modulus is about 67.7% for mixes made with 30 mm top layer thickness and 10 mm maximum aggregate size. The creep stiffness modulus of 30 cm top layer and 14 mm maximum aggregate size mix is the highest. The average creep stiffness of double layer mix made of top layer maximum aggregate sizes 14 mm is higher compared to the 20 mm single layer except for single layer made of maximum aggregate sizes 14 mm. The top layer made of maximum aggregate sizes 10 mm has higher creep stiffness from single made of maximum aggregate size 10 mm, but for SBS mix made of top layer maximum aggregate size 10 mm is higher from the single layer made of maximum aggregate sizes 20 and 10 mm.

7.0 CONCLUSIONS

1. Addition of SBS modified binder improves properties of double layer porous asphalt in terms of stability, resistance to disintegration, resilient and creep stiffness modulus.
2. The properties of double layer porous asphalt such as abrasion loss, indirect tensile resilient modulus and permeability increases when the porous asphalt top layer thickness decreases.
3. The stability, permeability, abrasion loss of double layer porous asphalt is generally higher in comparison with 14 and 10 mm single layer porous asphalt. However, the 20 mm single layer exhibits higher permeability and abrasion loss compared to all double layer mixes.

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APPENDIX

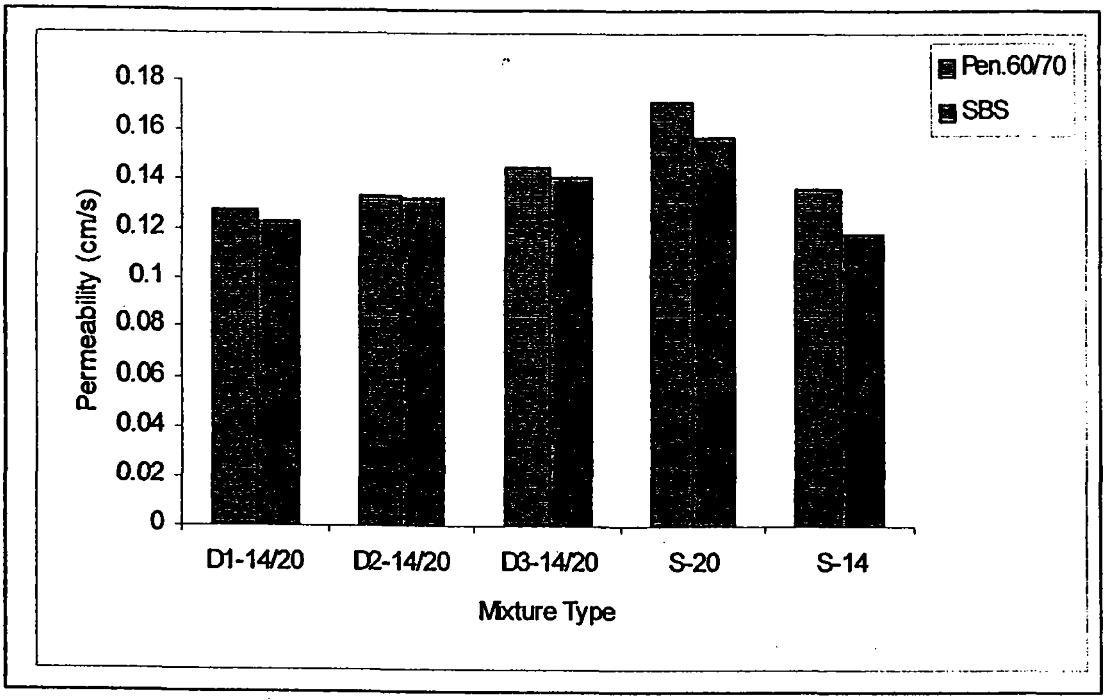

Figure 6 Permeability of Double Layer PA Made with 14 mm Top Layer Maximum Aggregate Size

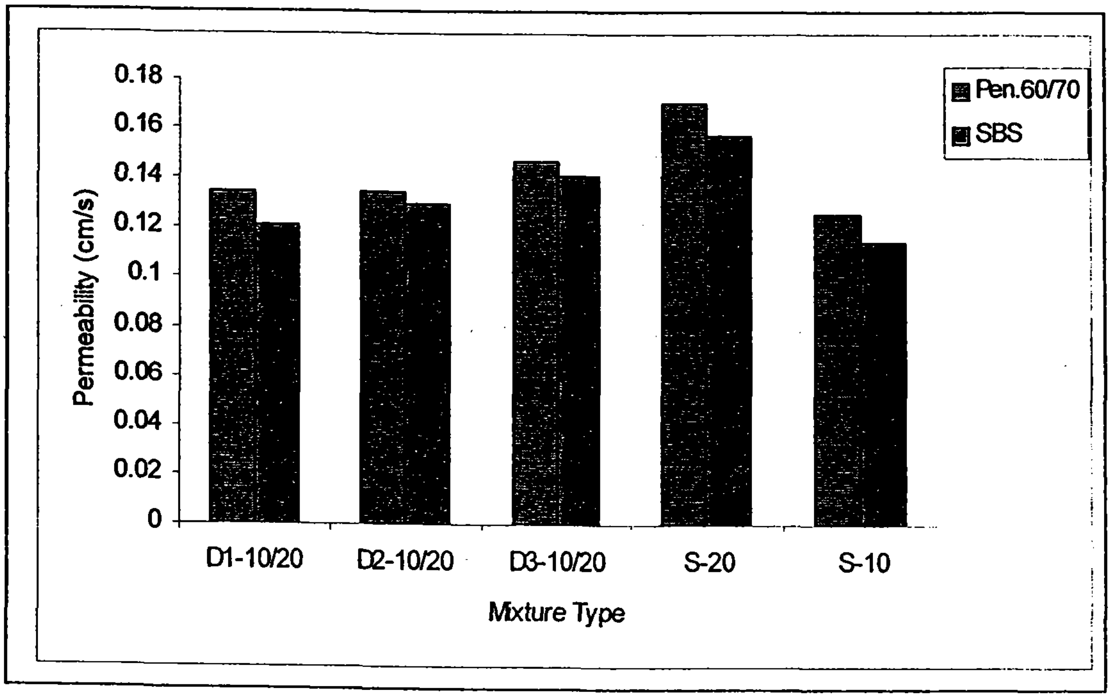

Figure 7 Permeability of Double Layer PA Made with 10 mm Top Layer Maximum Aggregate Size

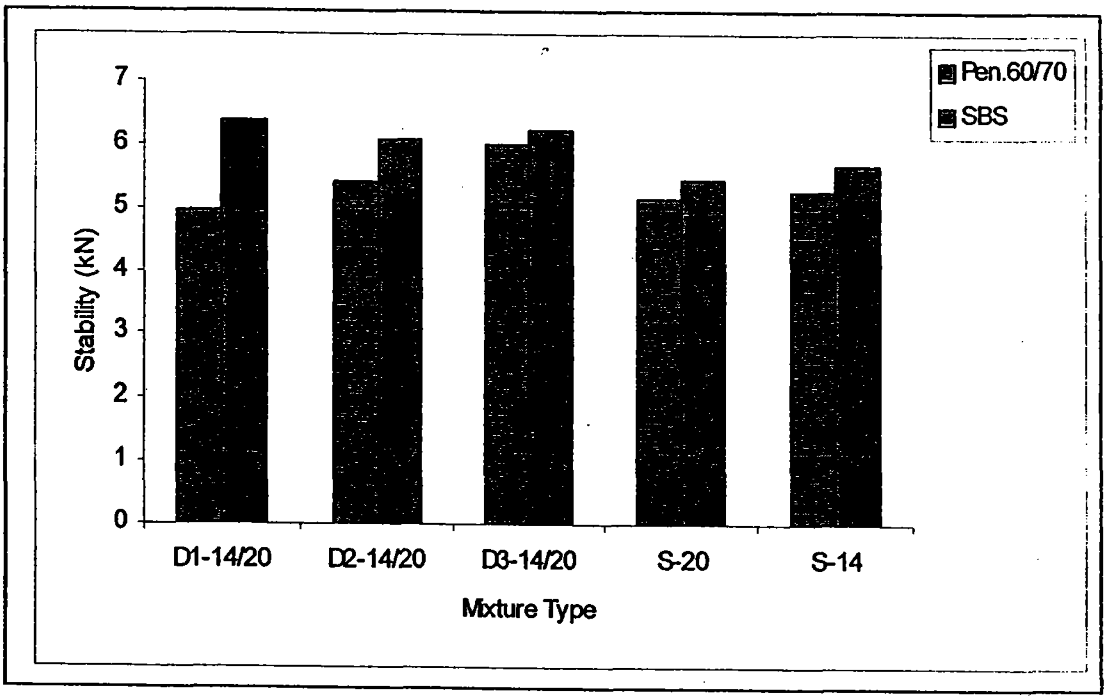

Figure 8 Marshall Stability of Double Layer PA Made with 14 mm Top Layer Maximum Aggregate Size

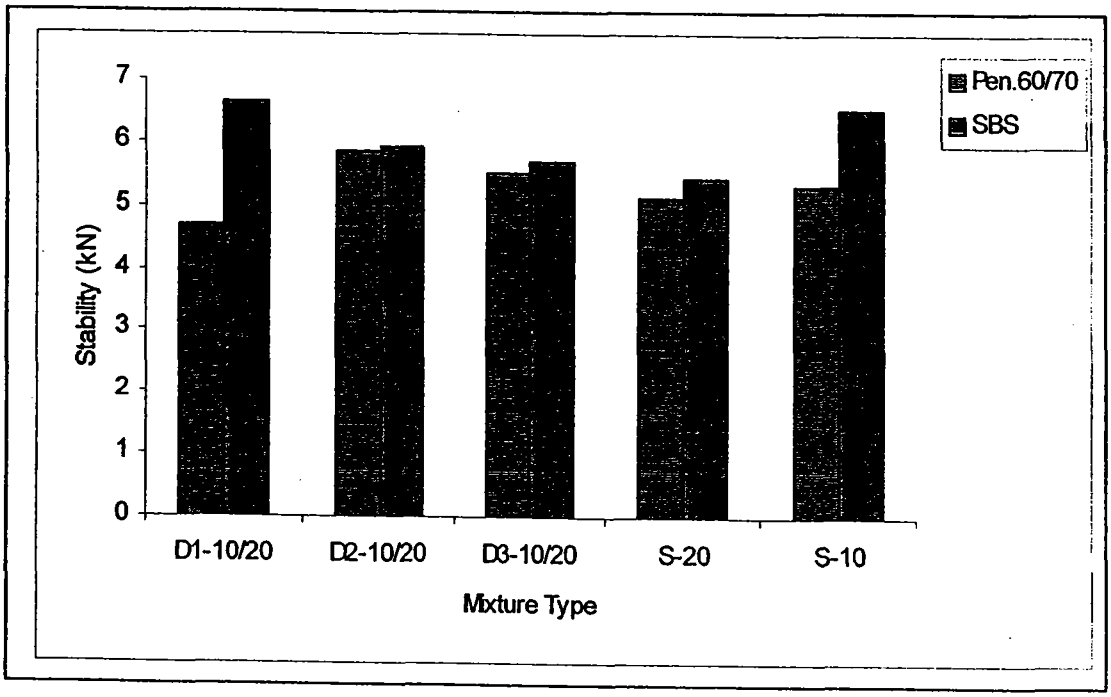

Figure 9 Marshall Stability of Double Layer PA Made with 10 mm Top Layer Maximum Aggregate Size

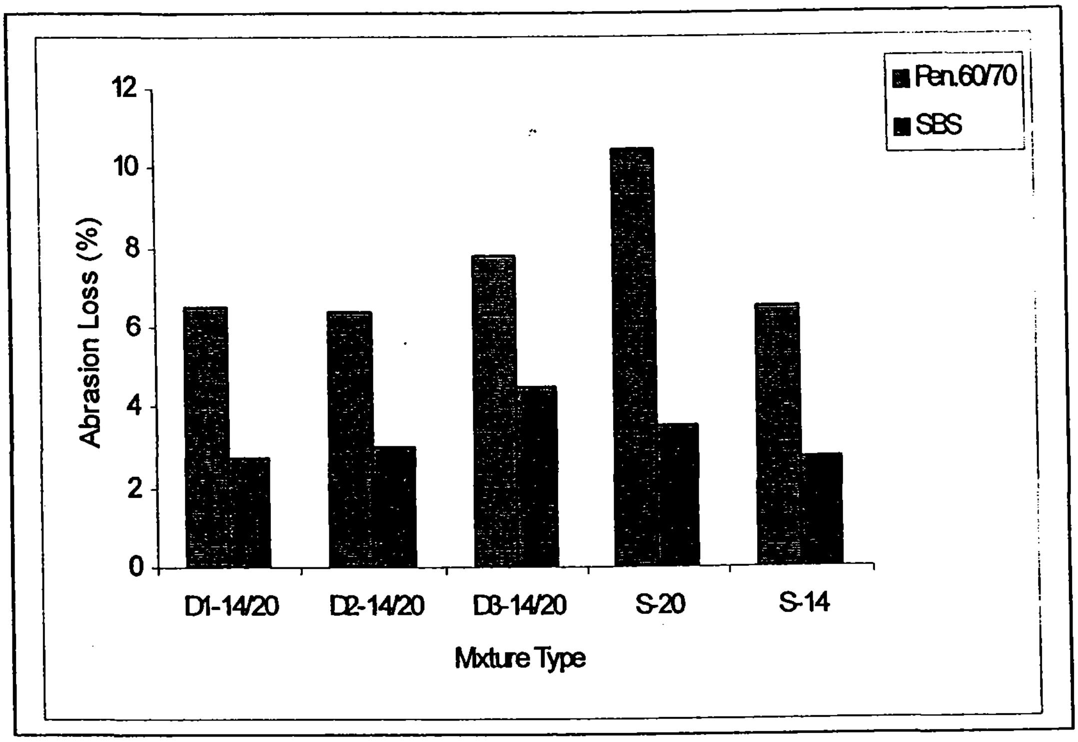

Figure 10 Abrasion Loss of Double Layer PA Made with 14 mm Top Layer Maximum Aggregate Size

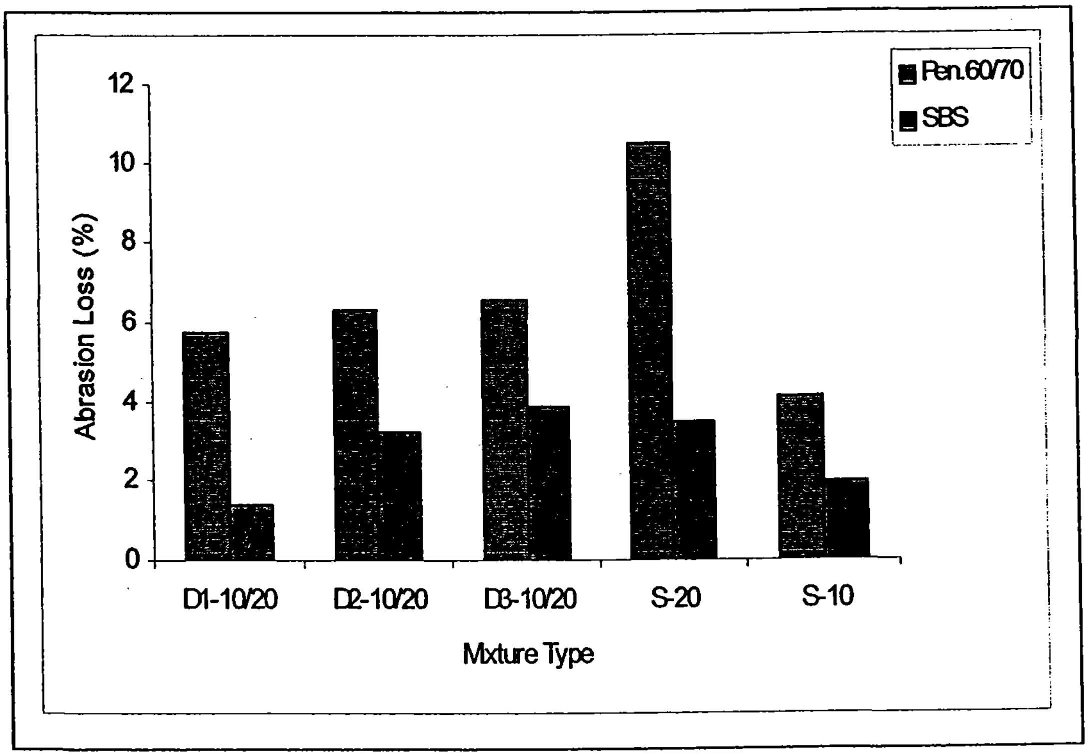

Figure 11 Abrasion Loss of Double Layer PA Made with 10 mm Top Layer Maximum Aggregate Size

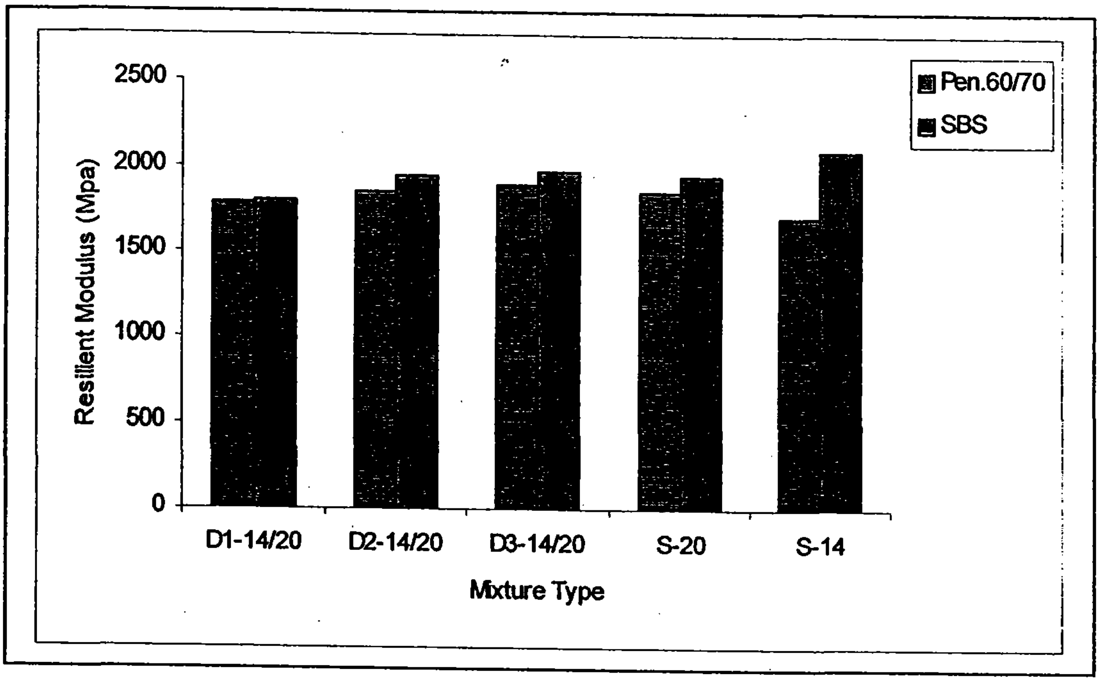

Figure 12 Resilient Modulus of Double Layer PA Made with 14 mm Top Layer Maximum Aggregate Size

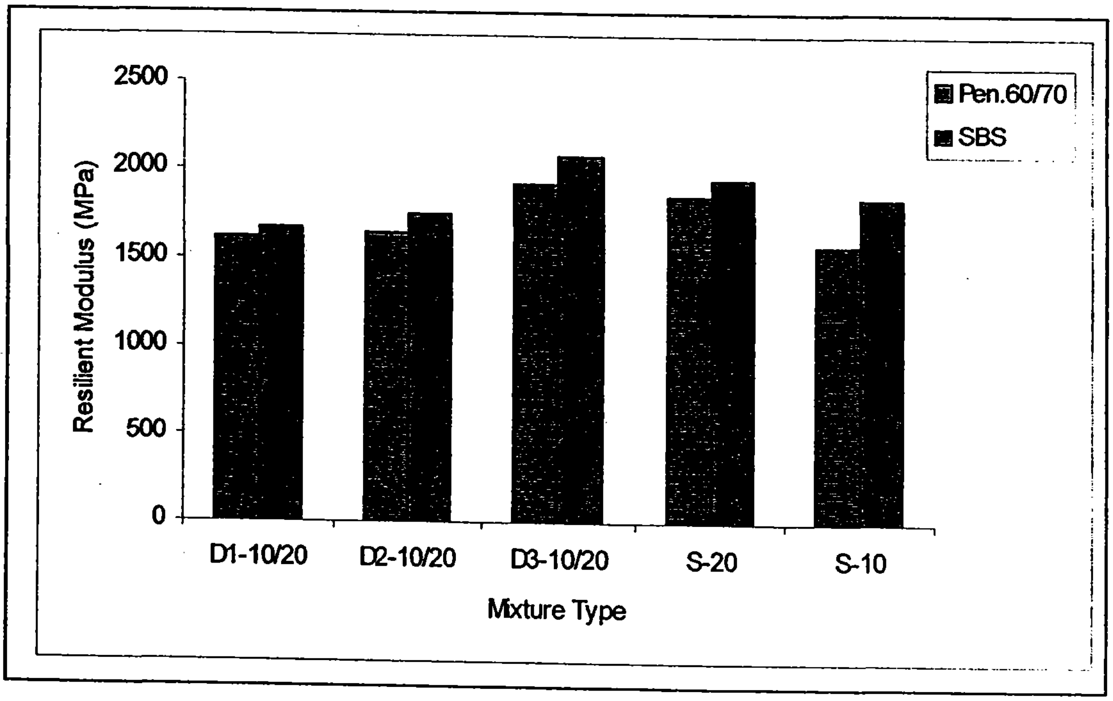

Figure 13 Resilient Modulus of Double Layer PA Made with 10 mm Top Layer Maximum Aggregate Size

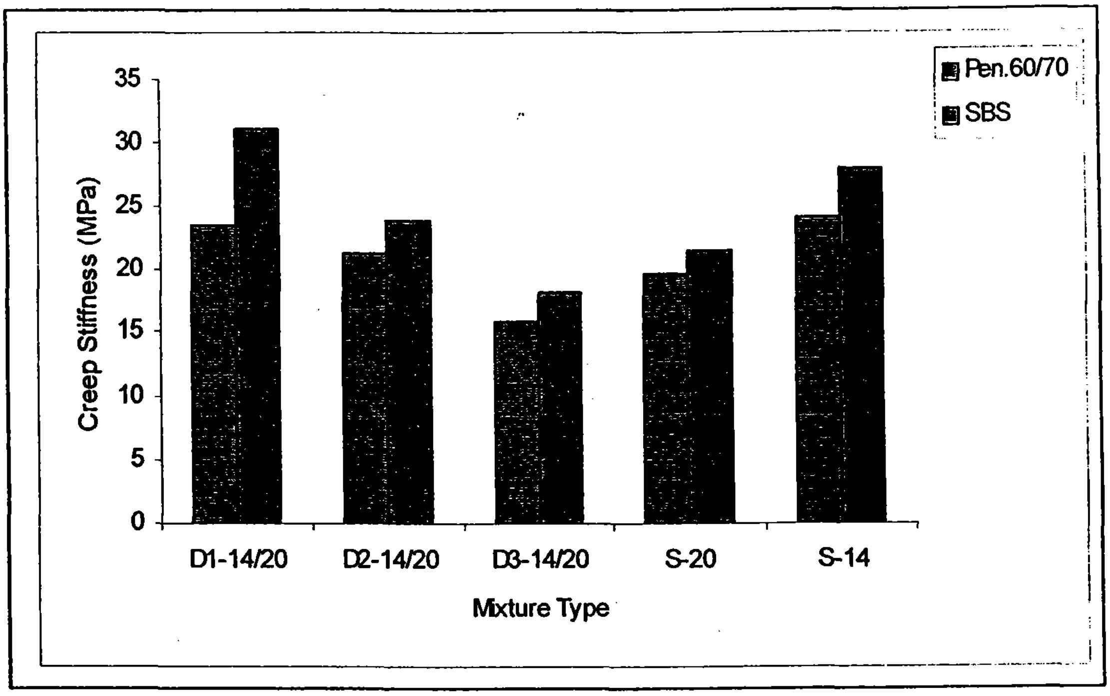

Figure 14 Creep Stiffness of Double Layer PA Made with 14 mm Top Layer Maximum Aggregate Size

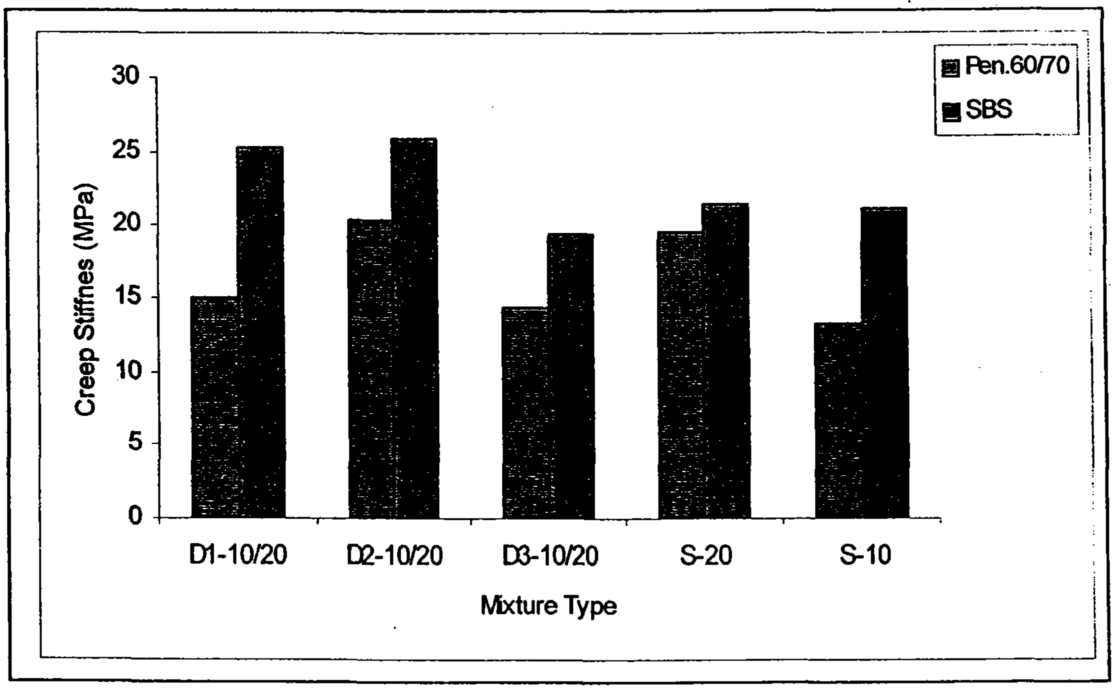

Figure 15 Creep Stiffness of Double Layer PA Made with 10 mm Top Layer Maximum Aggregate Size

CLOGGING BEHAVIOUR OF DOUBLE LAYER POROUS ASPHALT AND RESISTANCE TO OVERCOMPACTION OF POROUS ASPHALT

REPORT-6

SEPTEMBER 2004

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Preparation of 70 mm total thick for double layer porous Marshall specimen was similar to that of single layer porous asphalt. However, in a double layer system, the amount of aggregate for top and base layers were calculated based on the Marshall density of single layer. Knowing the density and height hence volume, the actual amount of mix required can be calculated, as described in Report 5. The compaction procedure adopted for single and double layer is 2x50 blows as detailed in Reports 1 and 5.

5.3 Preparation of Porous Asphalt Specimens at DBC for Overcompaction Test

There are two stages of specimen preparation for overcompaction test:

- Specimen preparation
- Overcompaction Test Procedure and Configuration

5.3.1 Specimen preparation

Preparation of specimen in this step was carried out as in Report 4 for single layer and Report 5 for double layer PA. Prior to extrusion, permeability was measured using a water permeameter. Having done this, each specimen was extruded and the weight of the dry mould recorded. Specimen mass was calculated by difference. The porosity was calculated based on the measurements of specimen heights and diameters. The next step was to slide the specimen into their moulds. This was made by heating the mould. Specimens, confined inside the mould, were then conditioned in an oven at 60°C for 3 hours. The oven was left at this temperature for at least 2 hours prior to specimen conditioning.

Overcompaction has to be carried out using the gyropac. Hence, specimens compacted via Marshall hammer has to be pushed inside the gyropac mould. It was found that the diameters of the two moulds were not truly compatible. Hence, specimen has to be compacted using the gyropac and overcompacted inside the gyropac mould itself. However, the density of gyropac compacted specimen has to be equivalent to the density of Marshall specimens compacted via 2x50 blows. To determine the pressure and number of gyrations to achieve the required density of single layer porous asphalt, a preliminary compaction investigation was carried out. Mixes used in this investigation have maximum aggregate size 14 mm and a 60/70 bitumen as binder. The target density was 1.98 g/cm^3. The compaction was carried out under various pressures or axial load, namely 240, 300 and 400 kPa and each axial load was subjected to 600 gyrations. From the results, a graphical relationship was drawn between density and gyrations at various axial loadings. The relationships indicated that a 1.98 g/cm^3 density is achievable either at combinations of 300 gyrations and 240 kPa or 170 gyrations and 300 kPa or 75 gyrations and 400 kPa. The complete results are shown in Figure 1.

However, in subsequent investigation, an axial pressure of under this configuration, it is possible to compact specimens to achieve a 1.98 g/cm^3 density as shown in Figure 2.

5.3.2 Overcompaction Test Procedure and Configuration

A preliminary investigation was carried out to determine the axial load and number of gyrations for overcompacting specimens. The axial loads tried were 300, 400, 500 and 600 kPa and each subjected to 600 gyrations. The test temperature of 60°C was adopted to accelerate the overcompaction process. The results of preliminary

1.0 INTRODUCTION

Two major problems afflicting porous asphalt performance is its poor resistance to disintegration and permeability loss. Poor resistance to disintegration leads to a variety of surfacing problems, such as potholes and raveling. Permeability loss can be due to voids closure a consequence of traffic overcompaction, binder creep and more importantly, clogging. Voids clogging have the net effect of diminishing mix permeability. When this happens, all benefits associated with an open mixture will be lost. Typical clogging agents include, dust, mud, tyre wear by-products and detritus or generally described as dirt.

To mitigate clogging, a double layer porous asphalt system has been proposed. A number of such constructions have been tried in the Netherlands, Italy and Denmark. In the Netherlands, the double layer system is described as Twinlay while the Italians termed it as Double Draining Layer. The concepts underlying the design of Twinlay was first reported to the 1st Eurasphalt and Eurobitume Congress in 1996 held at Strasbourg by Van Borchove (1996). From the perspective of resistance to permeability loss, the two-layered construction has the potential to better resist clogging compared to conventional single layer porous asphalt. In addition, the two-layered concept proved to be effective in reducing traffic noise at low and high speeds. Additional benefits over and above those of conventional porous asphalt layer were also observed. This primarily includes ease of cleaning maintenance.

2.0 OBJECTIVE

1. To assess the clogging behaviour of double layer porous asphalt.
2. To evaluate the resistance to overcompaction of single and double layer porous asphalt incorporating conventional and SBS modified binders.

3.0 SCOPE OF WORK

The scope of work during this stage of the research focus on the clogging behaviour of double layer porous asphalt, each layer made with top layer 10 mm and 14 mm maximum aggregate size while the base layer maximum aggregate size was of 20 mm. Resistance to overcompaction of porous asphalt (single and double layer) was measured using the gyropac. Specimens tested for overcompaction were tested for permeability before and after while its workability index determined during the process of mix compaction.

4.0 LITERATURE REVIEW

4.1 Reduction of Air Voids

It is a well known fact that porous asphalt suffers from voids closure due to the kneading action of traffic and silting up of the pores but the proportions of each these phenomena in the process cannot be assessed. In one field trial (Colwill and Daines, 1989), the average porosity of the material had fallen to 15.5% from 20.2% after 3 years. Apart from reducing the amount of dirt, provision of good drainage, an initial high porosity and the cleansing action of fast moving traffic, are some favourable conditions that could retard clogging. An initial porosity of at least 25% (Khandal et. al. 1977), 22% (Van Heystraten et. al. 1990), 20% (Jimenez and Gordillo 1990) have been suggested to cope with subsequent voids closure.

According to Koester (1985), the rate of voids closure was found to be most rapid after one year in service and subsequently took place at a much slower pace. He also found that voids clogged up more easily if the maximum aggregate size in the mix was 10 mm rather than 16 mm.

4.2 Permeability Loss

In general, permeability was measured by creating a known pressure differential across the specimen and then either measuring the amount of permeant collected over a known period of time, or noting the time taken for a permeant to fall between two pre-defined levels on the permeameter tube.

The coefficient of permeability (k), in units of cm/s, reported by a number of investigators differs greatly. Average k values of 0.16-0.17 cm/s (Gemayel and Mamlouk, 1998), 0.38 cm/s (Woelfl et al. 1981) and 0.71 cm/s (Daines 1986) have been reported.

When voids close up, permeability loss is imminent. From field trials at the A38 Burton Bypass, Colwill et. al. (1992) reported a hydraulic conductivity loss of 16 to 22% of the 'as laid' values after 3 years in service. After 6 years, the corresponding levels were 10 to 15%. Some road sections in Switzerland (Isenring et. al. 1990) showed satisfactory permeability values after 5 years of sustaining traffic, others had become almost completely dense within one year. At another site in Spain, the initial drainage times of 25-75 sec increased to 80-100 s after 3 years and to 160-400 s after 9 years (Kraemer 1990).

4.3 Workability

The Leeds workability method developed by Cabrera (1991) was used in this investigation. It was based on the relationship between mix porosity and the associated compaction energy input applied by the gyropac.

Factors affecting workability have been studied by a number of researchers. To summarize, workability was found to be influenced by aggregate type and texture; aggregate grading; type of filler, and content and type of binder.

5.0 METHODOLOGY

5.1 Materials

Materials used namely aggregate, filler and binders were as described in Report 1.

5.2 Preparation of Porous Asphalt Specimens at DBC for Clogging Test

Twinlay comprises of top and base layers. The maximum aggregate size for the base layer is 20 mm while two maximum aggregate sizes 14 and 10 mm were used for the top layer. The development of their gradations has been described in Report 1. The relative thickness of top and bottom layers (T/B) in a specimen are 0/70(S); 30/40 (D1); 20/50 (D2) and 15/50 mm (D3) as schematically shown in Figure 1 of Report 5. The binder contents used for all mixes are based on their respective design binder content as described in Report 2.

6.0 RESULTS AND DISCUSSION

6.1 Permeability of Clogged Specimens

Figure 5 shows the permeability of double layer porous asphalt using various trapped soil removal methods. The methods tried out include brushing, spraying and vacuum cleaning. It appears that the permeability after removing soil from specimen is only 63% and 67.3% of the initial permeability when a vacuum cleaner is used respectively for top layer aggregate sizes 14 and 10 mm. Nevertheless, when water sprayer and brushing is used, the corresponding values are 48.6%, 33.9% and 51%, 35.8% respectively. The results indicate that the residual permeability using vacuum cleaner is the highest compared to the other methods. This indicates that the vacuum cleaner can remove trapped soil from specimen better compared to the rest and is adopted in future investigations. However, the mixes made with 10 mm top layer maximum aggregate size and 15 cm top layer thickness has the higher permeability for all methods used.

In a double layer system, the top layer functions as a filter to trap dirt. Figure 6 shows the amount of retained dirt on the top layer which decreases as the top layer thickness decrease. However, the amount of retained dirt for double layer made with top layer maximum aggregate sizes 10 mm is higher compared with top layer maximum aggregate size 14 mm.

The cycles of reduction and increase in drainage time upon vacuuming is shown in Figures 7 through 18. For all maximum aggregate sizes, SBS mixes clogged faster compared to conventional mixes. For cycle 1 to complete, it requires an average of 4 and 5 loadings to attain the maximum drainage time for SBS and conventional mixes respectively. In terms of the number of cycles, top layer made with maximum aggregate sizes 14 and 10 mm attain the minimum permeability after 4 cycles but for top layer thickness 15 cm the corresponding value is 5 and 4 respectively for conventional and SBS mixes. The number of cycles and time of loading are important indicators in the drainage ability of porous asphalt. The resultant drainage time just upon vacuuming differs significantly from the terminal drainage time. After cleansing, the drainage time of mixes made of conventional and SBS binders is about 90s, 115s and 84s, 99s respectively for top layer maximum aggregate sizes 14 and 10 mm. However, top layer made of maximum aggregate size 10 mm and thickness 15 cm exhibits the lowest drainage time when drainage time is about 80 sec. From the perspective of resistance to clogging, the two-layered porous asphalt construction has the potential to better resist clogging compared to conventional single layer porous asphalt.

6.2 Resistance to overcompaction

6.2.1 Permeability

The results of permeability tests of porous asphalt before and after overcompaction are shown in Figures 19 and 20. Figures show that the drainage time of porous asphalt mixes increase after overcompaction test. Drainage time for mixes compacted at high axial load 600 kPa is higher compared with porous asphalt compacted at low axial load 400 kPa. The increase of drainage time for mixes made of base binders are 113% to 582% and 274% to 826% respectively for low and high axial loads when compared with the drainage time before overcompaction. For SBS mixes the drainage time are 35% to 251% and 85% to 647% respectively for low and high axial loads. The results indicate the severe effect in terms of permeability loss upon overcompaction and are more pronounced at higher axial load. However, mixes made of base binder and maximum aggregate sizes 10 mm recorded the highest drainage time. It was found, that the drainage time of SBS mixes is higher compared to the conventional mixes. But, drainage time for double layer porous asphalt is lower when the top layer thickness decreases.

6.2.2 Workability Index

The workability index (WI) is calculated based on the correlation between voids and gyropac gyrations. The values of WI for the porous asphalt mixes studied are shown in Table 1 and Figure 21. The porosity at zero gyration was determined from the extrapolation of lines shown in Figures 22 through 27. Figure 21 shows that the conventional mixes have higher WI compared with SBS mixes. The WI also increases as the maximum aggregate size decreases. For both unmodified and modified mixes studied, the reduction in WI of double layer PA is insignificant when top layer thickness decreases. However, the WI of conventional double layer is slightly higher compared with SBS mixes. The results indicate that porous asphalt made of smaller aggregate and base binder are relatively easier to compact when compared with other mixes. It was also found that the workability index decreases when initial compacted void in mix increase.

6.2.3 Porosity and Porosity Reduction

The initial void in a porous mix is very important for porous asphalt. While in service, subsequent reduction of voids can decrease the service life of porous asphalt. Voids in double layer mixes were calculated based on the assumption that the density specimen of top and base layers was homogonous. The difference of voids at initial and after overcompaction is as voids reduction in porous mixes. Table 2, Figures 28 and 29 show the void in mixes before and after overcompaction while Figures 30 through 35 show the porosity reduction during the overcompaction process. The initial voids in porous asphalt made of base bitumen is 21.43%. After overcompaction at low and high pressures, the porosity reduces to 18.26% and 17.15% respectively. With SBS mixes, the corresponding values are 21.91%, 19.37% and 18.37%. Mixes made of base bitumen and smaller maximum aggregate size record the highest reduction of voids. For double layer porous asphalt mixes, the loss of void is lower as the top layer thickness decreases. The results show that the percentage void reduction for mixes made of base bitumen is 17.35% and 24.24% of initial voids before overcompaction respectively for low and high loads. SBS mixes has 13.14% and 19.36% of initial void before overcompaction respectively for low and high loads. Hence, the porosity loss of double layer porous asphalt is lower compared with single layer. An exception is the single layer made of maximum aggregate size 20 mm. It can be concluded that the double layer suffers from less porosity reduction compared with the single layer.

7.0 CONCLUSIONS

1. The duration and number of cycles to clogging of porous asphalt made with SBS mix is slightly shorter compared to conventional mix.
2. As the top layer thickness increases, it takes a longer time to clog the specimen.
3. Double layer PA has the potential to better resist clogging compared to conventional single layer PA.

4. The effect of permeability loss as a consequent of overcompaction is very severe and is more pronounced at higher overcompaction load.
5. The drainage time after overcompaction for porous asphalt prepared using SBS binder is higher compared with conventional mixes, but the loss of voids in SBS mixes is lower compared with conventional mix.
6. The workability index of single layer increases when the maximum aggregate size reduces.
7. The workability index of double layer porous asphalt decreases slightly when the top layer thickness decreases.

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APPENDIX

Table 1 Workability index for porous asphalt

Mixture	Voids at 0 Radian		Work. Index (WI)		Remarks
Type	Pen. 60/70	SBS	Pen. 60/70	SBS	
20 S	40	40	2.50	2.50	Single layer
14 S	37	38	2.70	2.63	
10 S	36	37	2.78	2.70	
14D1	35	37	2.86	2.70	Double Layer
14D2	36	37	2.78	2.70	
14D3	37	37	2.70	2.70	
10 D1	36	36	2.78	2.78	Double Layer
10 D2	36	37	2.78	2.70	
10 D3	36	38	2.78	2.63	

Table 2 Voids in mix for porous asphalt before and after overcompaction

Mixture	Initial Voids (%)		Voids at LP (%)		Voids at HP (%)		Remarks
Type	Pen 60/70	SBS	Pen 60/70	SBS	Pen 60/70	SBS	
20 S	20.35	21.03	18.45	19.58	17.25	18.82	Single layer
14 S	21.13	21.52	13.38	19.53	17.38	18.08	
10 S	22.79	23.09	18.19	19.28	16.82	18.12	
14 D1	21.11	21.27	17.27	18.27	16.58	17.37	Double layer
14 D2	21.66	21.48	18.10	18.68	17.09	17.63	
14 D3	21.99	22.71	18.61	20.11	17.95	18.81	
10D1	21.63	22.49	18.56	19.77	17.07	18.77	Double layer
10 D2	21.09	22.05	18.23	19.67	17.13	18.64	
10 D3	21.16	21.55	18.59	19.41	18.01	18.97	

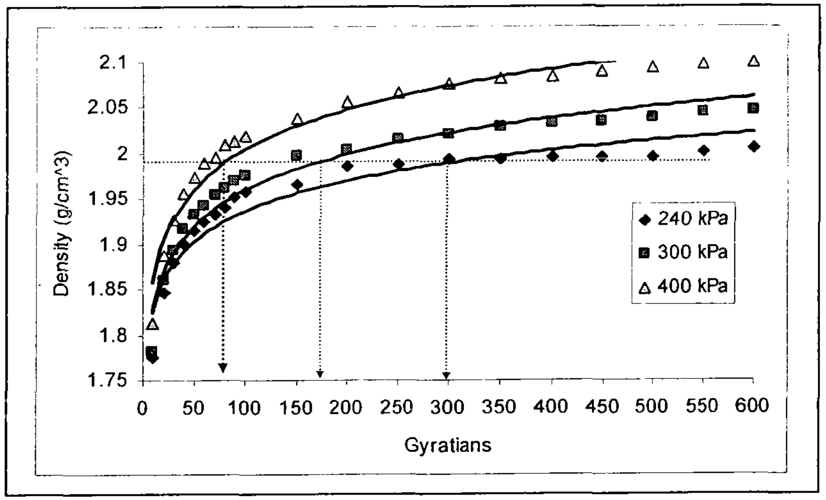

Figure 1 Densities of porous asphalt specimens compacted under various axial loads and gyrations

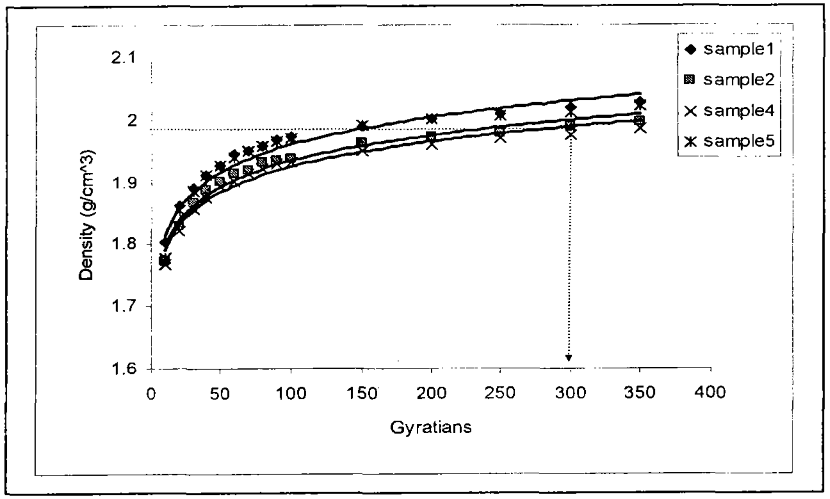

Figure 2 Densities of porous asphalt specimens compacted under 240 kPa axial load and various gyrations

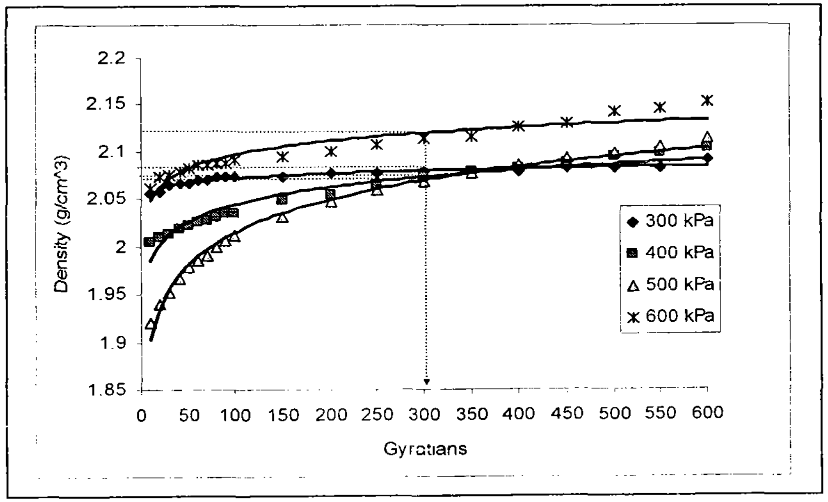

Figure 3 Densities of porous asphalt specimens over-compacted under various axial loads and gyrations

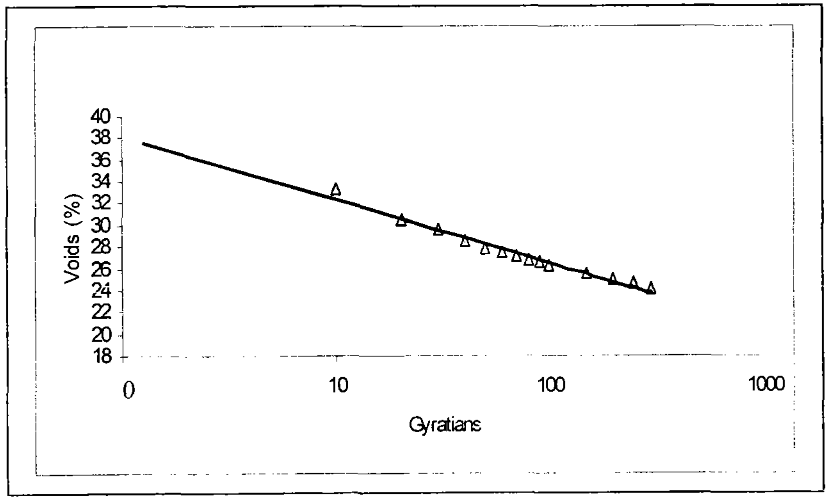

Figure 4 Relationship between voids and logarithm of number of Gyratians

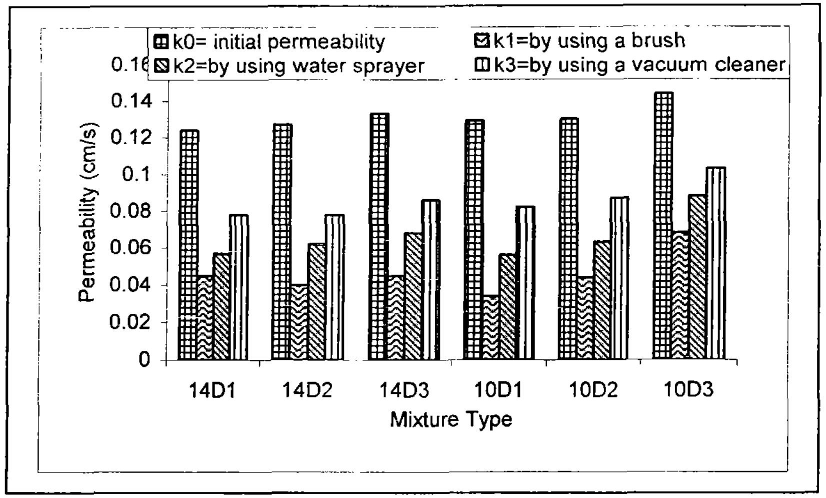

Figure 5 Permeability after removal soil by using various methods

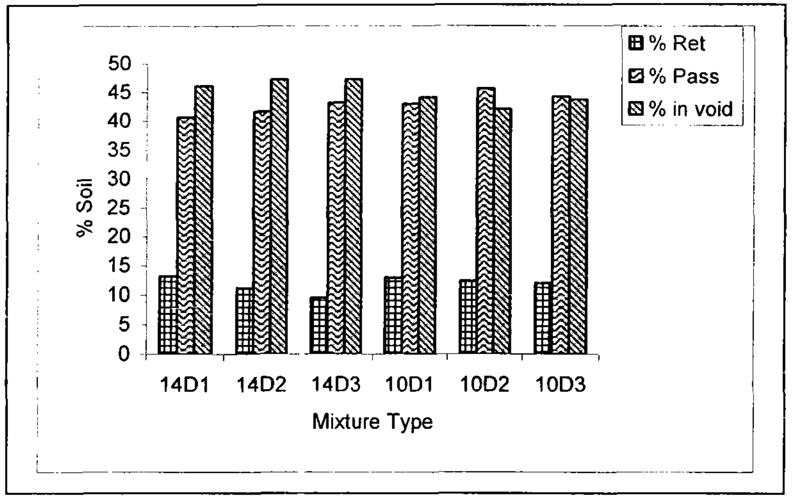

Figure 6 Percentage of trapped soil in double layer PA

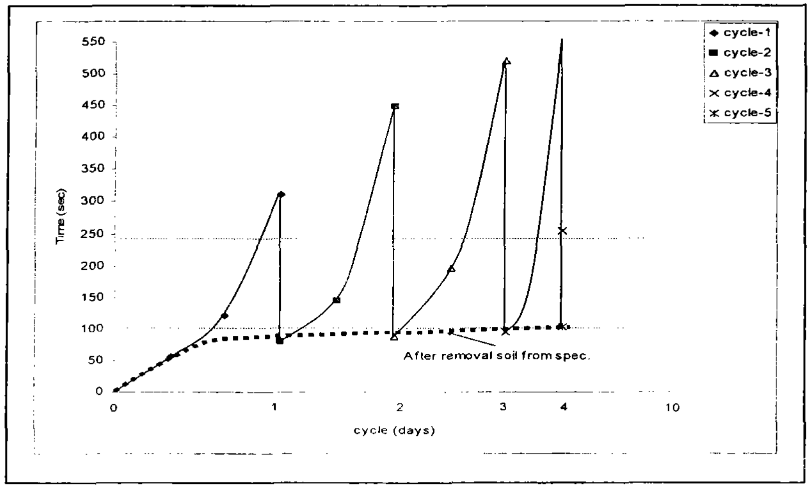

Figure 7 Cycle time for clogging of double layer PA made of 30 cm top layer thickness and 14 mm max. aggregate size (pen 60/70)

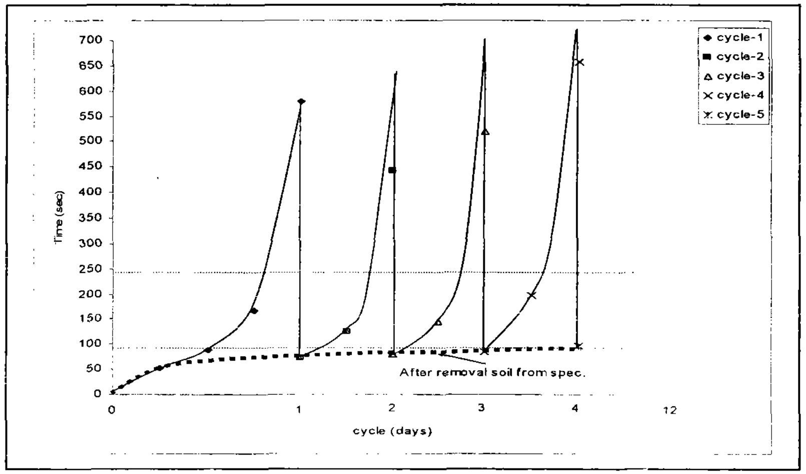

Figure 8 Cycle Time for clogging of double layer PA made of 20 cm top layer thickness and 14 mm max. aggregate size (pen 60/70)

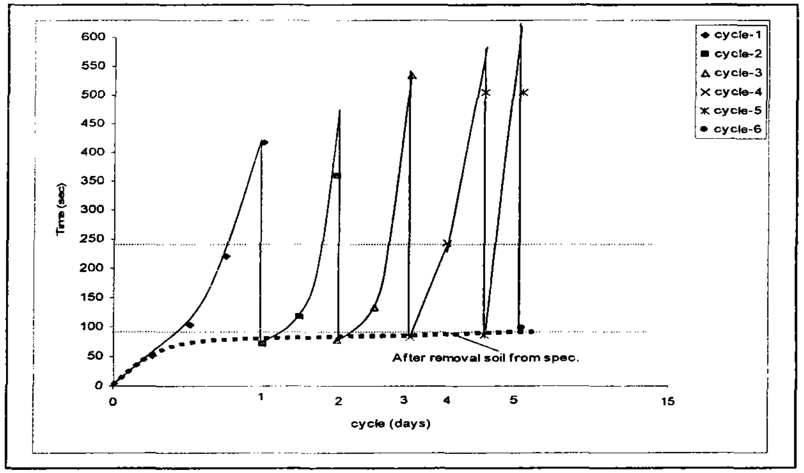

Figure 9 Cycle Time for clogging of double layer PA made of 15 cm top layer thickness and 14 mm max. aggregate size (pen 60/70)

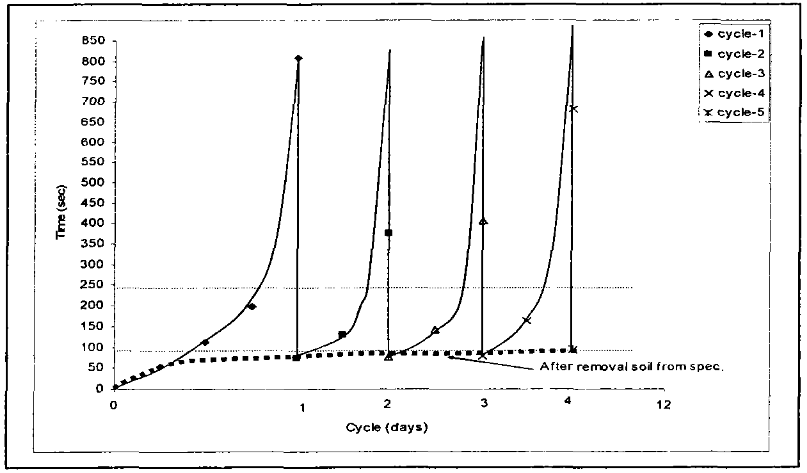

Figure 10 Cycle time for clogging of double layer PA made of 30 cm top layer thickness and 10 mm max. aggregate size (pen 60/70)

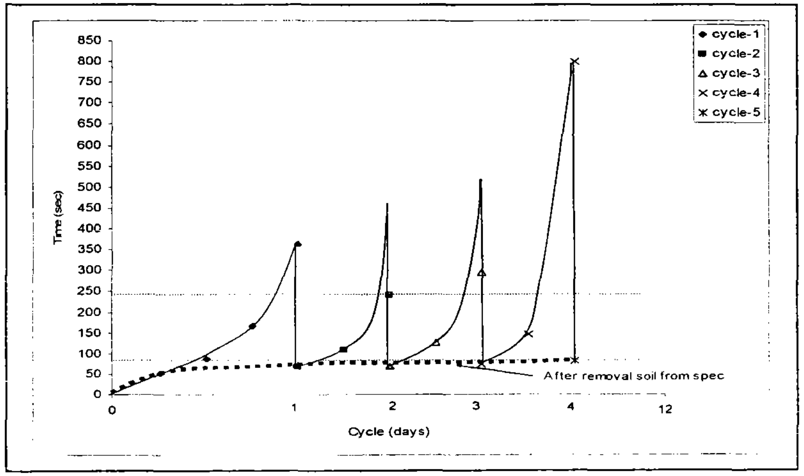

Figure 11 Cycle time for clogging of double layer PA made of 20 cm top layer thickness and 10 mm max. aggregate size (pen 60/70)

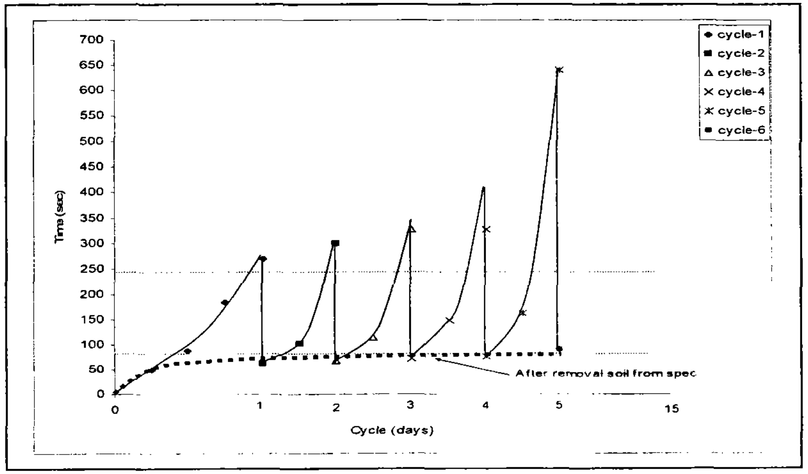

Figure 12 Cycle time for clogging of double layer PA made of 15 cm top layer thickness and 10 mm max. aggregate size (pen 60/70)

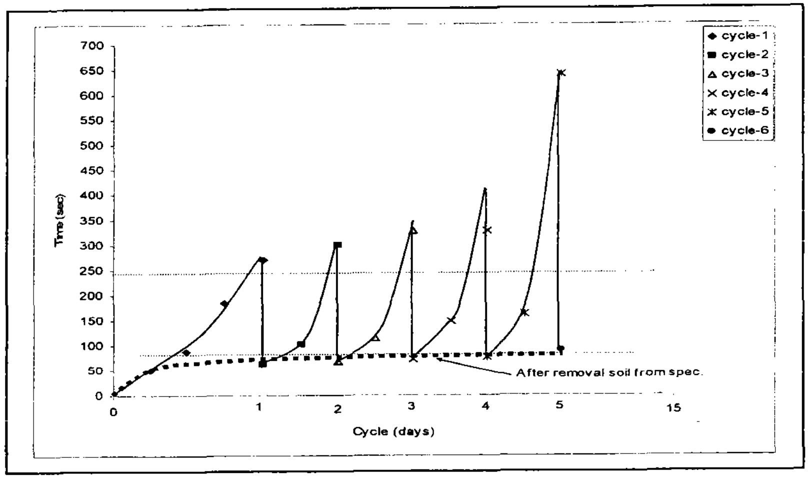

Figure 13 Cycle time for clogging of double layer PA made of 30 cm top layer thickness and 14 mm max. aggregate size (SBS)

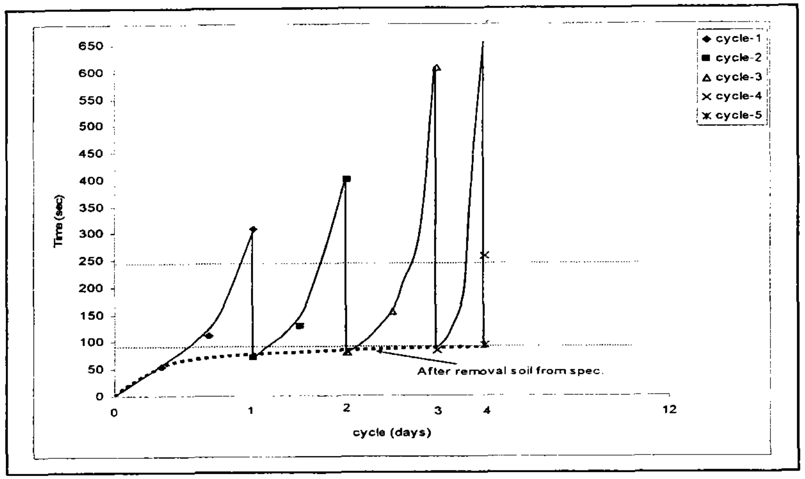

Figure 14 Cycle time for clogging of double layer PA made of 20 cm top layer thickness and 14 mm max. aggregate size (SBS)

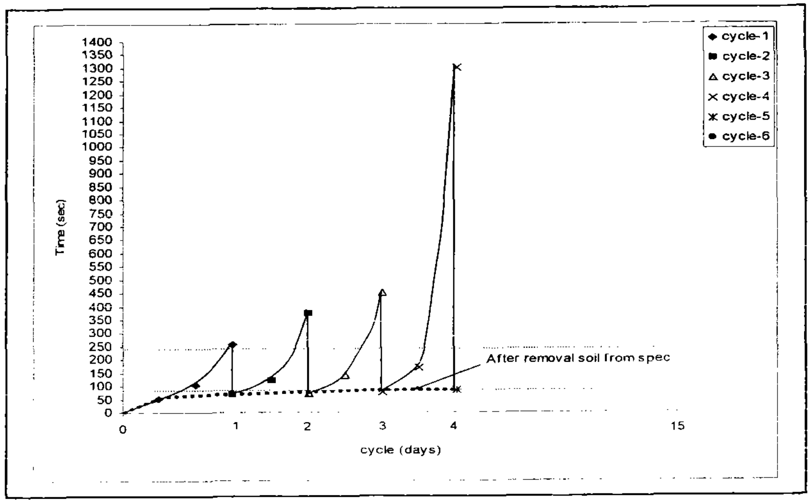

Figure 15 Cycle time for clogging of double layer PA made of 15 cm top layer thickness and 14 mm max. aggregate size (SBS)

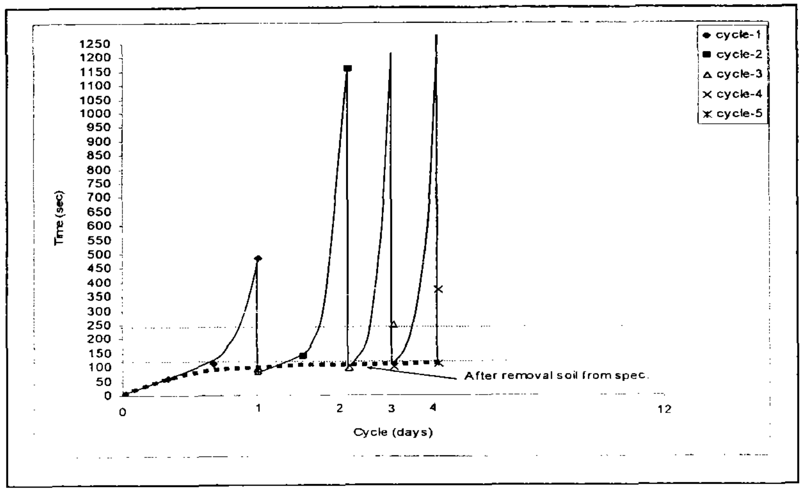

Figure 16 Cycle Time for clogging of double layer PA made of 30 cm top layer thickness and 10 mm max. aggregate size (SBS)

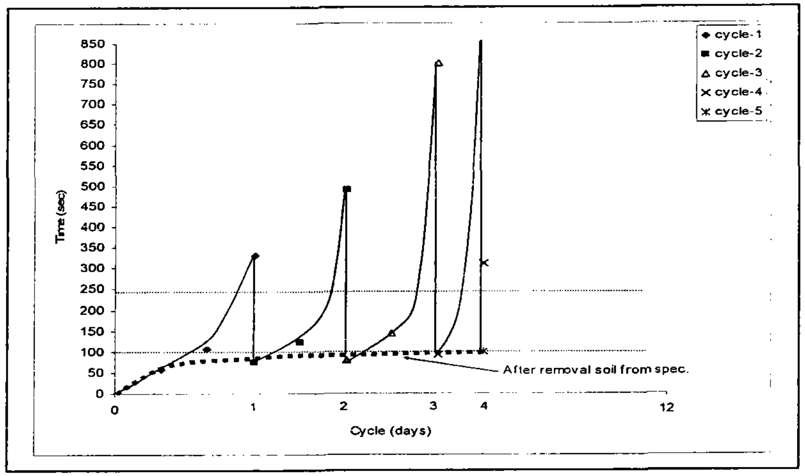

Figure 17 Cycle time for clogging of double layer PA made of 20 cm top layer thickness and 10 mm max. aggregate size (SBS)

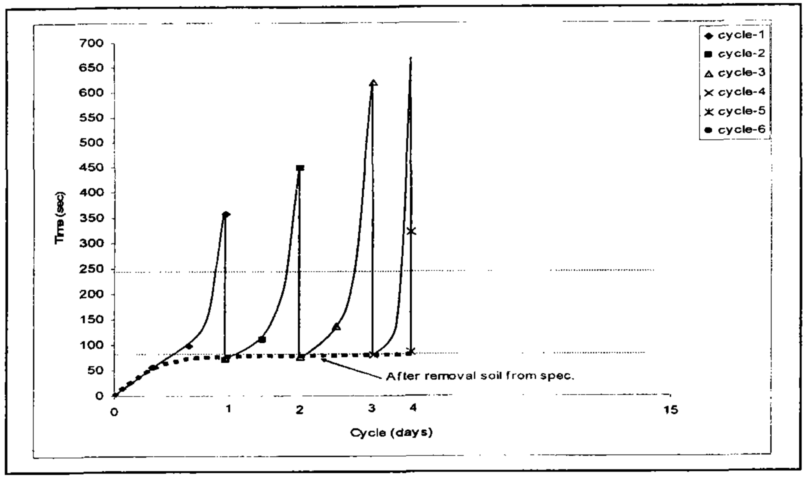

Figure 18 Cycle time for clogging of double layer PA made of 15 cm top layer thickness and 10 mm max. aggregate size (SBS)

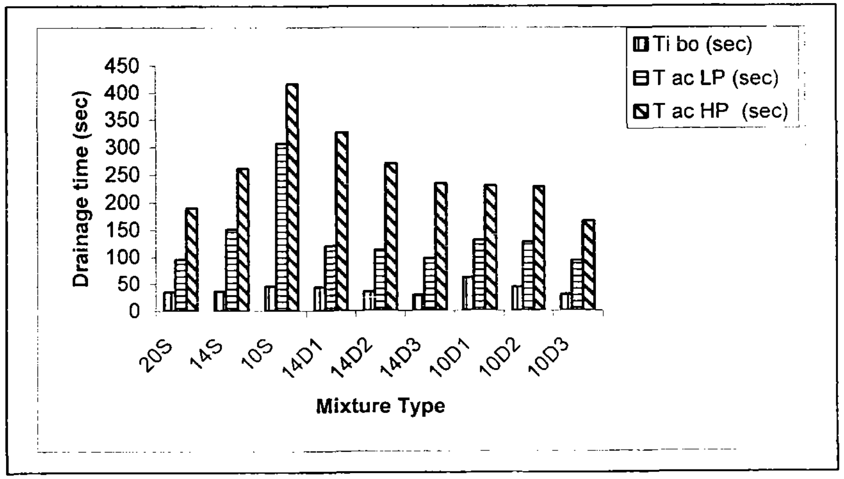

Figure 19 Drainage time for porous asphalt Made of base binder at low and high axial loads

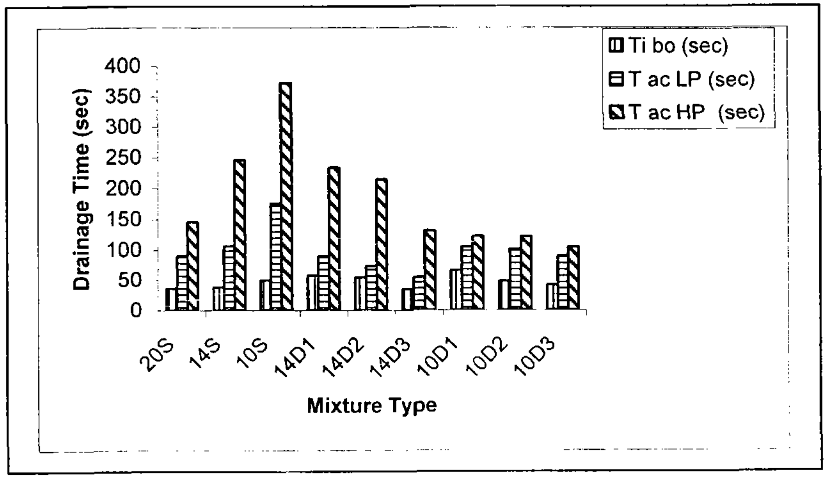

Figure 20 Drainage time for porous asphalt made of SBS at low and high axial loads

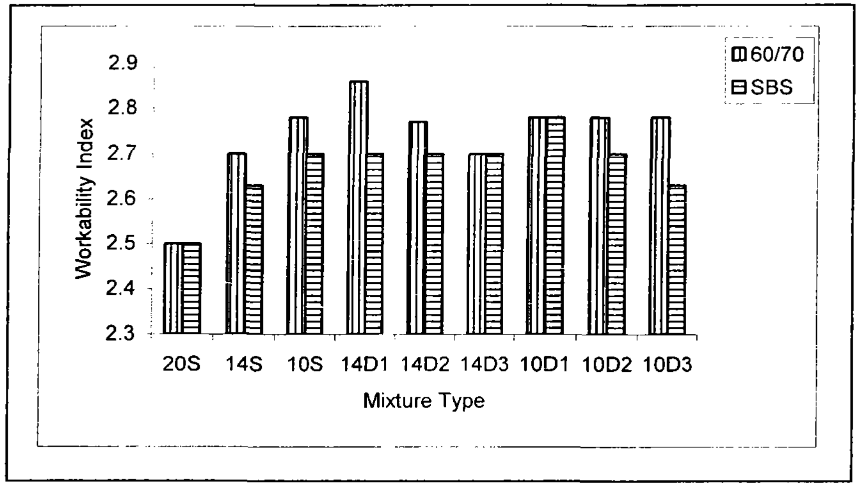

Figure 21 Workability index of porous asphalt made of base and SBS binders

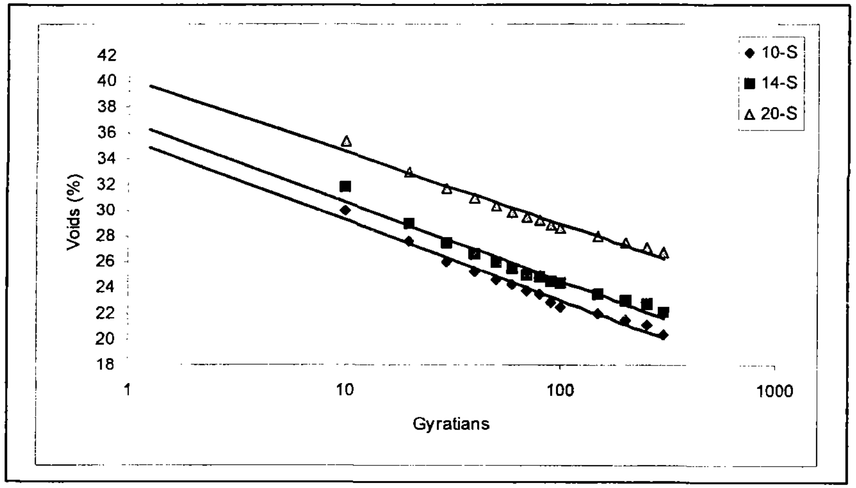

Figure 22 Relationship between void and gyratians for single layer PA made of base binder

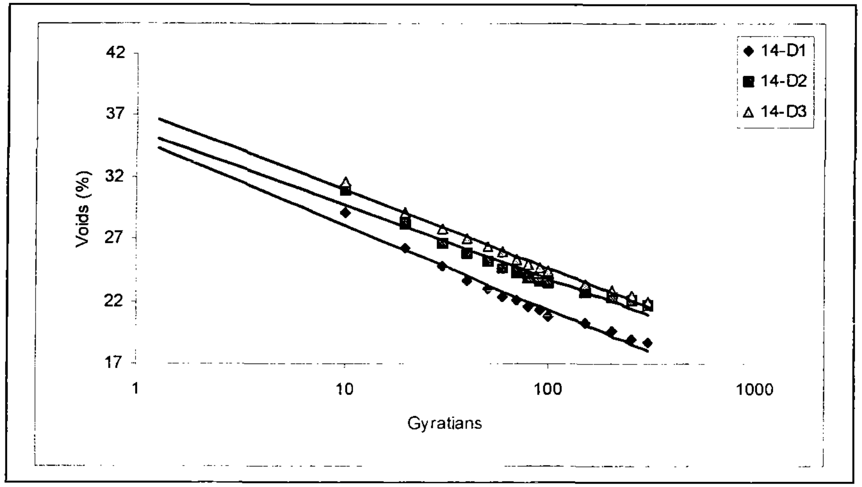

Figure 23 Relationship between void and gyratians for double layer PA made of base binder and 14 mm top aggregate size

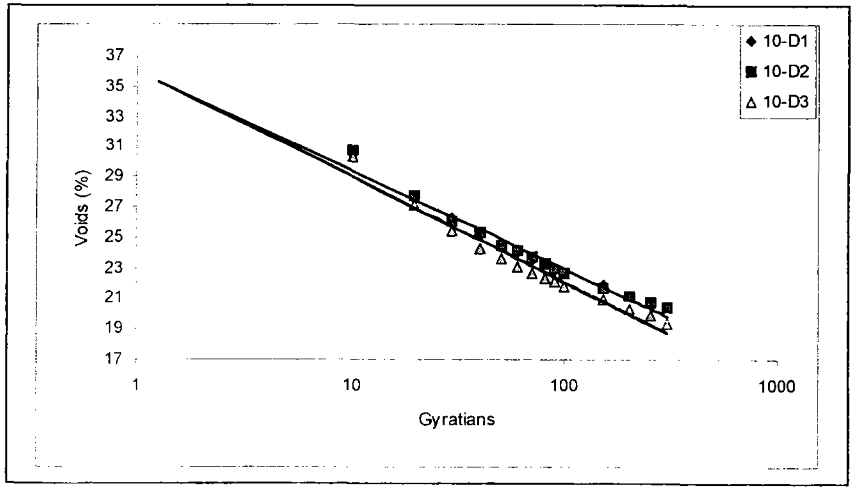

Figure 24 Relationship between void and gyrations for double layer PA made of base binder and 10 mm top aggregate size

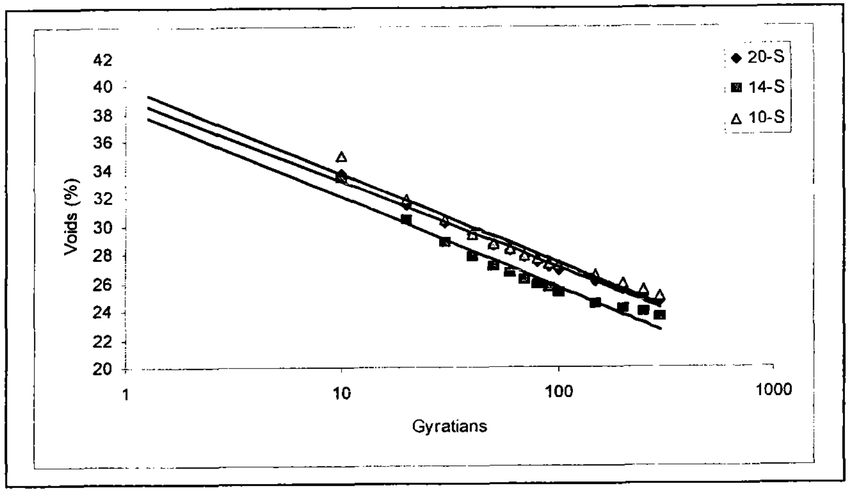

Figure 25 Relationship between void and gyratians for single layer PA made of SBS

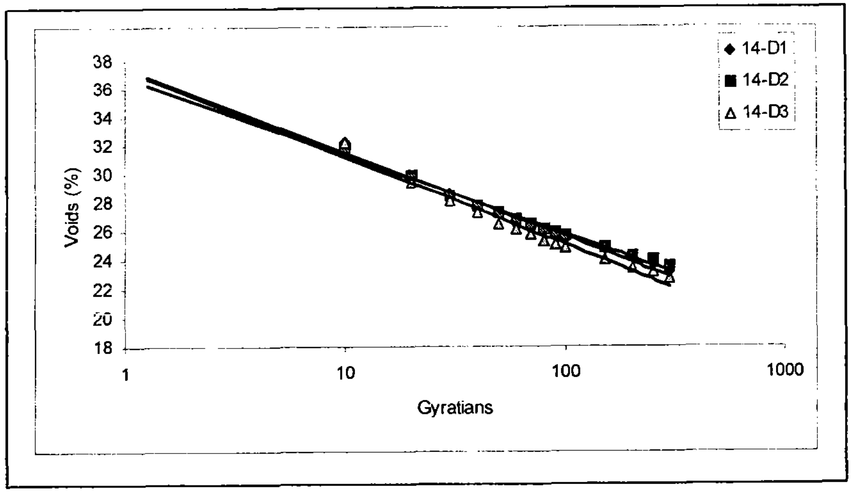

Figure 26 Relationship between void and gyratians for double layer PA made of SBS and 14 mm top aggregate size

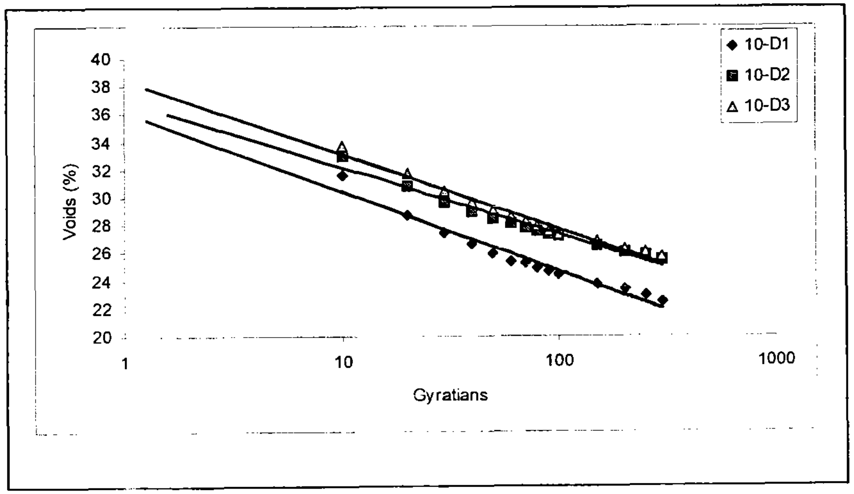

Figure 27 Relationship between void and gyrations for double layer PA made of SBS and 10 mm top aggregate size

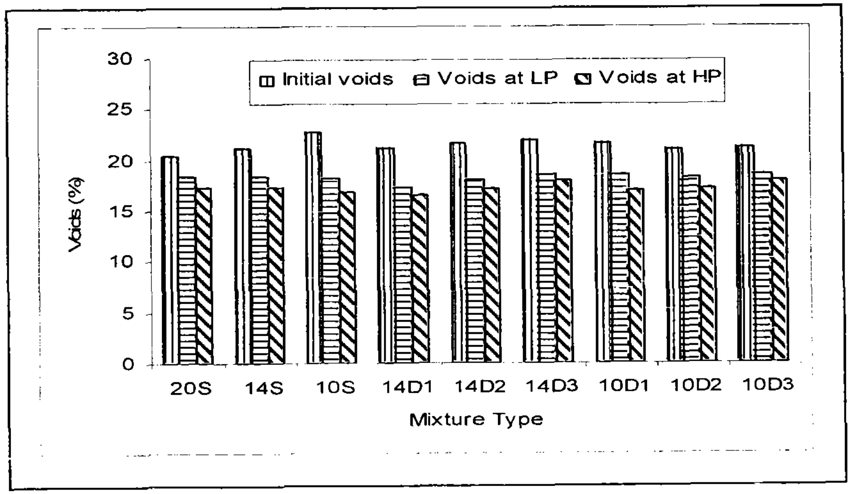

Figure 28 Voids in mix of porous asphalt made of base binder at low and high axial loads

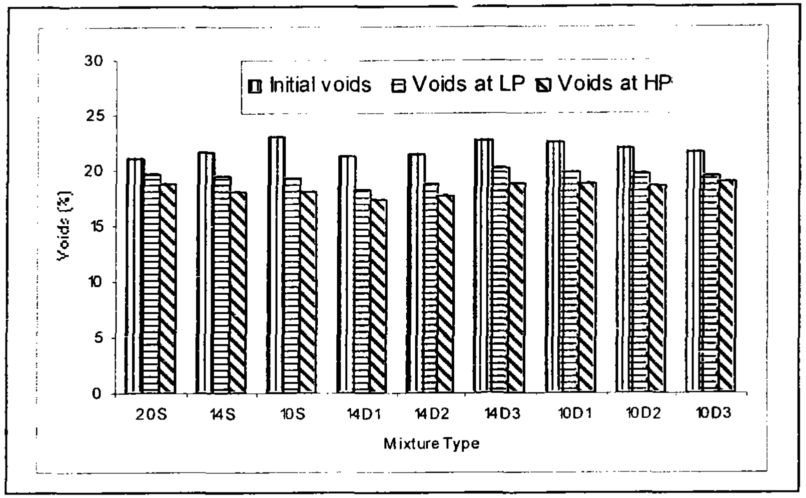

Figure 29 Voids in mix of porous asphalt made of SBS at low and high axial loads

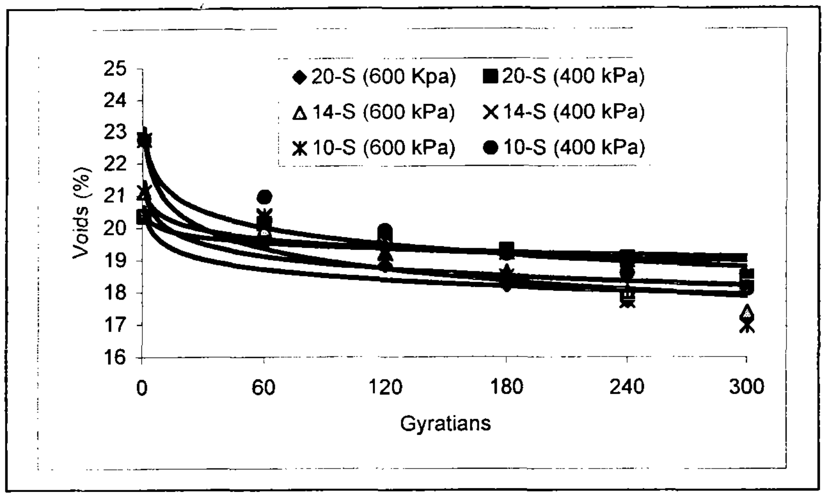

Figure 30 Relationship between voids and gyrations of single layer PA made of base binder at low and high axial loads (pen 60/70)

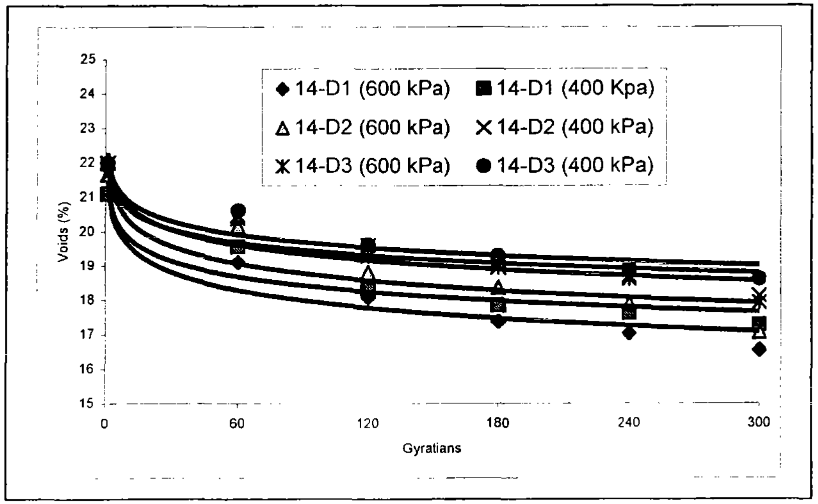

Figure 31 Relationship between voids and gyrations of double layer PA made of 14 mm top agg. size at low and high axial loads (pen. 60/70)

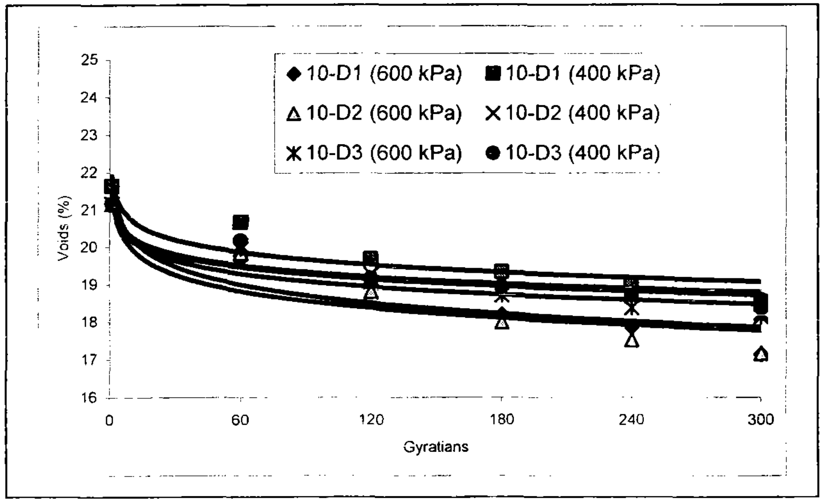

Figure 32 Relationship between voids and gyrations of double layer PA made of 10 mm top agg. size at low and high axial loads (pen. 60/70)

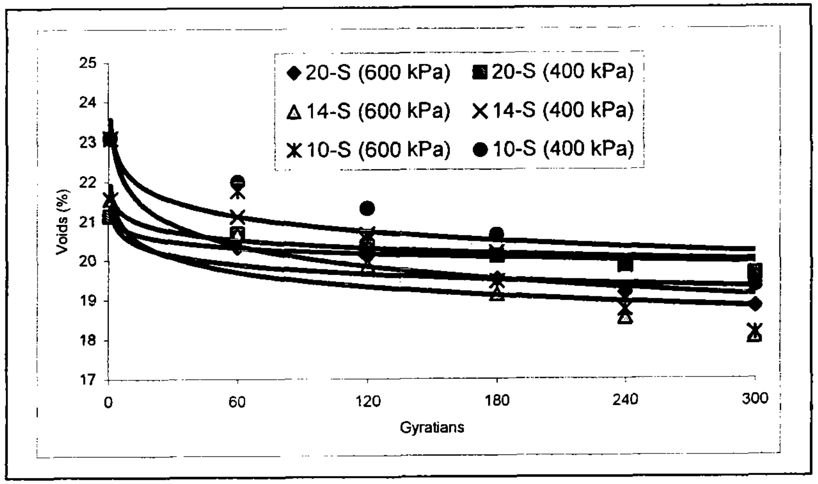

Figure 33 Relationship between voids and gyrations of single layer PA made of SBS at low and high axial loads

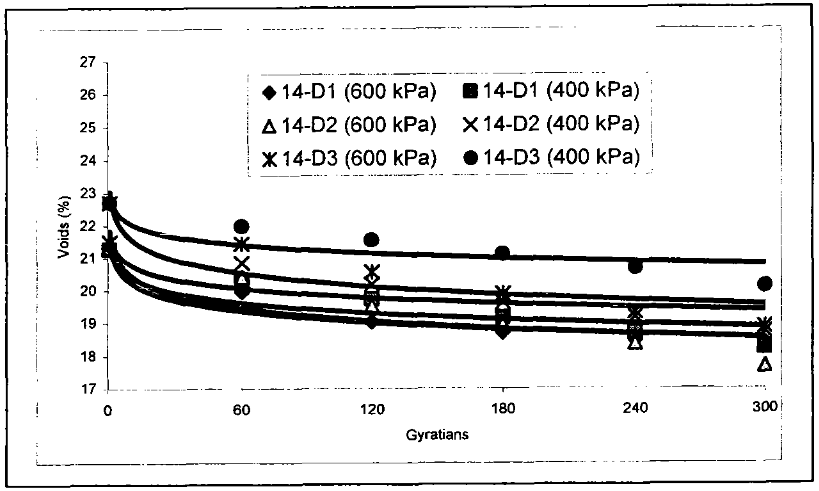

Figure 34 Relationship between voids and gyrations of double layer PA made of 14 mm top agg. size at low and high axial loads (SBS)

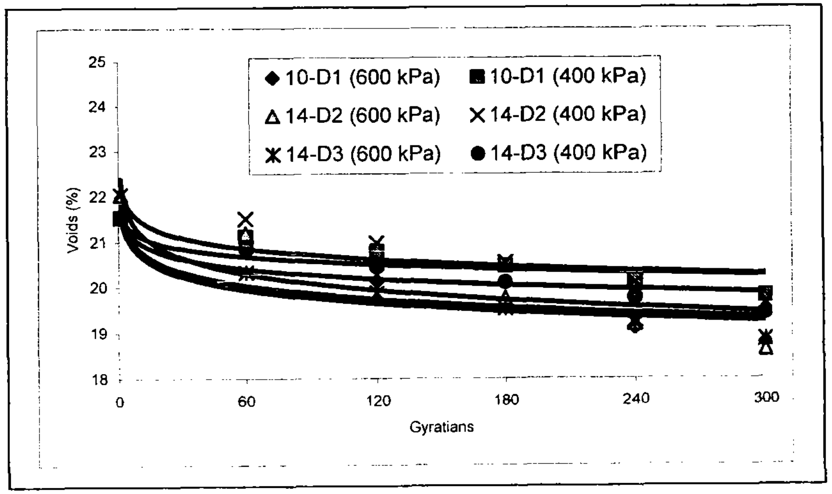

Figure 35 Relationship between voids and gyrations of double layer PA made of 10 mm top agg. size at low and high axial loads (SBS)

RESISTANCE TO PERMANENT DEFORMATION OF POROUS ASPHALT

REPORT-7

APRIL 2005

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1.0 INTRODUCTION

Increased truck traffic, heavier axle loads and higher tire pressure have contributed to the demand for rut-resistant hot mix asphalt (HMA). Rutting or permanent deformation pavement has been and continues to be a major problem on Malaysian roads. Rutting is defined as accumulation of small amounts of unrecoverable strain resulting from applied wheel loads to HMA pavements. This deformation is caused by consolidation or lateral movement or both of the HMA under traffic. Rutting not only decreases the useful life of a pavement but also creates a safety hazard for the traveling public. Porous asphalt is known to be a rut resistant material. Hence, use of porous asphalt as wearing course can helps to prolong life of a pavement.

2.0 OBJECTIVES

The objectives of the research program are as follows:

1. To explore method of slab compaction for the wheel tracking test.
2. To investigate the resistance to rutting of porous asphalt.
3. To determine the horizontal permeability and permeability loss of porous asphalt after trafficking.

3.0 SCOPE OF WORK

The scope of work during this stage of the research focuses on the preparation of specimens and e valuation of the horizontal permeability of porous asphalt made of conventional and modified binder. All slabs prepared measured 300 x 300 x 50 mm.

4.0 LITERATURE REVIEW

4.1 Resistance to Permanent Deformation

More than 30 years ago, the wheel tracking test was developed in order to test and forecast the deformation and adhesive behaviour of asphalt in laboratories. It was described for the first time in the English literature. The basic principles of this test procedure has later been modified and introduced in several countries (Harders, et. al., 2004)

One of the characteristics which are of prime importance in ensuring longer design life and adequate usability of asphalt wearing courses is the resistance to permanent deformation or rutting. Resistance to permanent deformation refers to the resistance to plastic, irreversible deformation under high temperatures and is very dependent on the asphalt mix composition. Use of highly resistant aggregates, crushed stone particles instead of natural stones, modified binders and skeleton structure of asphalt mixes all contribute to reduction of plastic deformations (Ljubic, 2004). It was concluded that the changes in the asphalt mix composition, the resistance to permanent deformation was always lower than with optimum composition, for both types of mixes SMA (stone mastic asphalt) and AC (asphalt concrete). For SMA mixes the content of bituminous mortar is of primary importance for resistance to permanent deformations. Too low content or too high content has a great effect on that resistance.

According to Little et al. (1993), the resistance to permanent deformation of an asphalt pavement layer is not only related to the stiffness of asphalt binder but also to the volumetric of the mixture and the bonding interaction between asphalt binder and aggregate. Permanent deformation of asphalt concrete is influenced by the nature and amount of polymer, fines or mineral filler (particles smaller than 75 mm in the mix).

5.0 METHODOLOGY

5.1 Materials

Materials used namely aggregate, filler and binders were as described in Report 1.

5.2 Preparation of Slab Specimens at DBC

Two types of porous asphalt specimens will be prepared, namely single and double layer PA. the double layer is made up of the top and base layer. The maximum aggregate size for the base layer is 20 mm while two maximum aggregate sizes 14 and 10 mm will be used for the top layer. The development of their gradations has been described in Report 1. The relative thickness of top and bottom layers (T/B) for Marshall specimens were 0/70(S); 30/40 (D1); 20/50 (D2) and 15/50 mm (D3), as reported in Report 5.

The preparation of 300x300x50 mm specimen slab of porous asphalt was based on the density of single layer Marshall specimens. However, the amount of aggregate for top and base layer of double layer was calculated based on the Marshall density of single layer, as shown in the Appendix. Knowing the density and volume, the actual amount of mix required can be calculated. The relationship between density and binder content used for this estimation for conventional and SBS binder are shown Figures 1 and 2. By interpolation, the densities at DBC for maximum aggregate sizes 20, 14 and 10 mm are 2.0; 1.98; 1.97 g/cm^3 and 2.01; 1.98; 1.96 g/cm^3 respectively for conventional and SBS mixes. Relative thickness for slab specimens (T/B) were calculated based on the relative thickness of Marshall specimens, and the values for single layer PA 0/50 (SL), double layer PA 21.4/28.6 (DL1); 14.3/35.7 (DL2); 10.7/39.3 mm (DL3). The total double layer slab thickness was 50 mm. The binder contents used for all mixes were based on their respective design binder content.

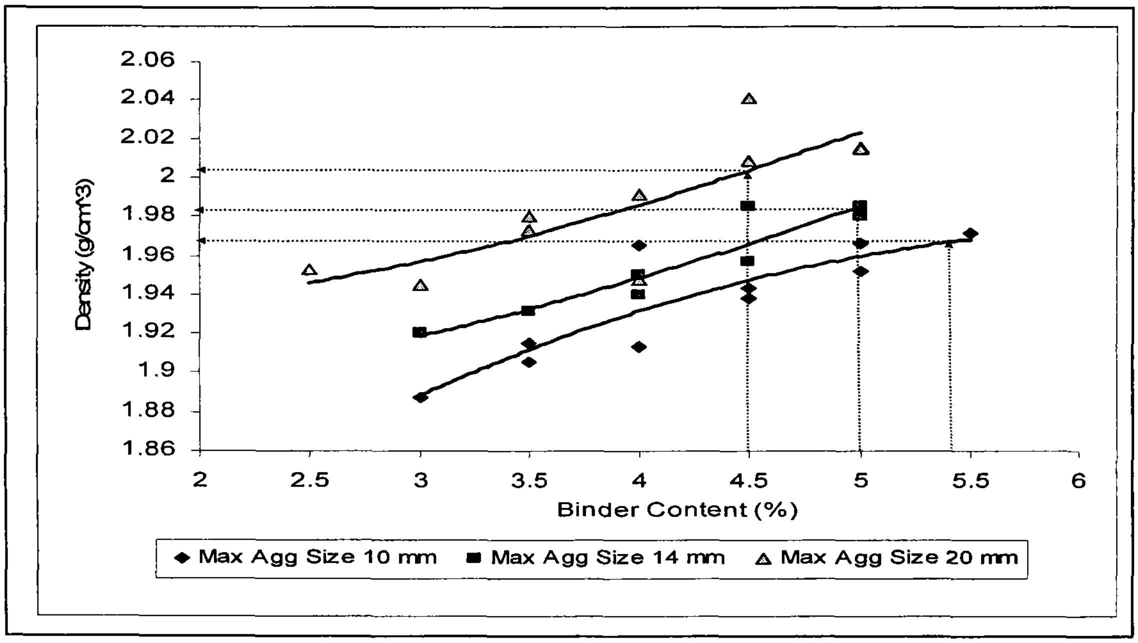

Figure 1 Density of Single Layer Porous Asphalt at Various Binder Contents (Bitumen Pen 60/70)

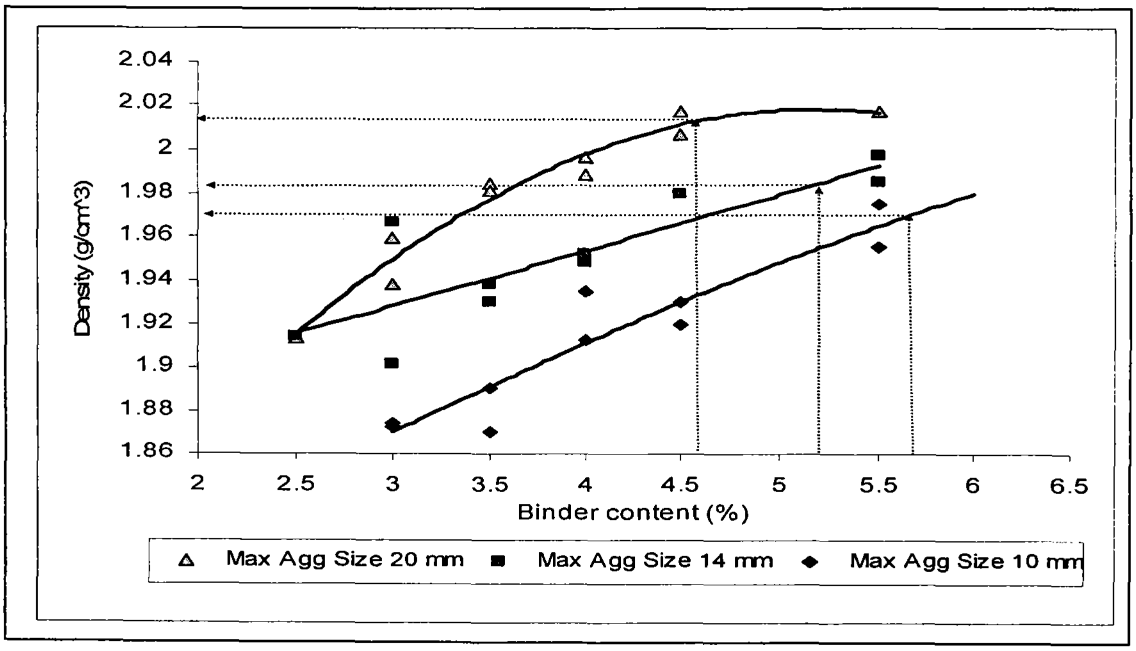

Figure 2 Density of Single Layer Porous Asphalt at Various Binder Contents (SBS)

Initial investigation has been carried out to determine the compaction method necessary to achieve the target density for single layer porous asphalt made maximum aggregate sizes 20, 14 and 10mm. All specimens were prepared using base bitumen. However, the target density was not achieved when only the Kango hammer was used whereby the average density for single layer PA reached only 1.9 g/cm^3.

Next, slab compaction was accomplished in two steps, impact mode via the Marshall hammer and vibratory compactor via the Kango hammer. The compaction procedure for single layer porous asphalt slab is similar as in the preceding trial. However, for double layer PA, the compaction procedure takes the following steps:

- The base layer mix ingredients were blended in a mixer. The required quantity of porous mix was placed inside the slab mould and the whole assembly was then transferred into the oven which was earlier set at the destined compaction temperature.

- For the next step, the porous mix for the top layer was prepared. Upon completion, the calculated amount of mix was then transferred into the slab mould and placed on top of the base layer mix.

- The mix was then manually compacted using the static Marshall hammer followed by the Kango hammer compactor. The duration for compaction process for both compaction mode ranges from 10 to 15 minutes.

5.3 Tests for Double Layer Porous Asphalt at DBC

5.3.1 Resistance to Permanent Deformation of PA

Over the past 10 years, the Austroads National Asphalt Research Committee (NARC) has developed test equipment and test methods to undertake performance testing of asphalt mixes. The tests used for deformation resistance are the dynamic creep test and the laboratory wheel tracking test.

From November 1993 to May 1995 Austroads undertook an accelerated pavement deformation trial using the ARRB Transport Research Accelerated Loading Facility (ALF) machine in Beerburrum, Queensland. The outcome of the trial was that good correlation was achieved between the rutting measured by the laboratory wheel tracking test and under the ALF. This trial resulted in the adoption by Austroads of the laboratory wheel tracking test procedure as the principal deformation test.

5.3.2 Wheel Tracking Test

The wheel-tracking test was carried out for evaluating the resistance of a bituminous mix to plastic deformation. According to the TRL, the laboratory test was carried out at 45°C, the wheel applying a load of 525 N to the surface of sample. Prior to wheel tracking test, horizontal permeability was measured using a water permeameter for slab specimen.

Wheel tracking test on asphalt mixtures implicates the following:

- Produce a test specimen of asphalt in the form of a slab with the dimensions of 300x300x50 mm.
- Condition the specimen for 1 hour at 45 °C in the wheel tracking test machine.

- Perform the wheel tracking test on the specimen surface for 1 hour. The wheel passed the specimen 21 cycles in 1 minutes or 1260 cycle in 1 hour. The machine automatically switches off after 1 hour tracking.
- Plot results to show the progression of rutting.
- The wheel tracking results are expressed as tracking rate (in units of mm/cycles or mm/hour) or final tracking depth (mm) or percentage rut.

5.3.3 Permeability Test

Permeability test for core slab specimen was measured using a water permeameter as described in Report 4. However, for slabs, the coefficient of horizontal permeability was calculated based on the TRL formula (Daines 1992) and is shown in the following equation:

$$K= [((R^2/(2dt)) \times \log(h_1/h_2) \times \log(A2/A1)]$$

Where:

R = Radius of the standpipe (cm)
D = Thickness of slab (cm)
T = Outflow time (s)
h_1 = Initial water head (cm)
h_2 = Final water head (cm)
A1 = Area of central hole in rubber padding (cm^2)
A2 = Contact area between porous surfacing and rubber padding (cm^2).

6.0 RESULTS AND DISCUSSION

6.1 Density

The density values of slab specimen for wheel tracking test is shown in Table 1. Results show that the density of all mixtures is comparable to the target density. Based on the results, all mixtures can be compacted with combination of impact and vibratory modes using the Marshall hammer and the Kango hammer. However, the density for double layer was calculated by considering that the top and base layer is homogenous.

Table 1 Density of Porous Asphalt Mixtures

Mixture Type	Density (g/cm^3)	
	60/70 Pen	SBS
20SL	2.05	2.08
14SL	2.06	2.05
10SL	2.02	2.05
14DL1	2.06	2.06
14DL2	2.05	2.05
14DL3	2.06	2.07
10DL1	2.05	2.05
10DL2	2.07	2.06
10DL3	2.05	2.07

6.2 Horizontal Permeability

Figure 1 show the horizontal permeability results of all mixes tested. The coefficient of horizontal permeability of SBS mixes is higher compared to conventional mixes. From Figure 1, the horizontal permeability of SBS mixes are on average 0.17 cm/s, 0.26 cm/s, 0.27 cm/s respectively for single layer, double layer made of maximum aggregate sizes 14 and 10 mm. However, this value increases when the top layer thickness decreases. The highest horizontal permeability 0.33 cm/s was recorded for conventional mix (14DL3) and SBS mixes (10DL3). The average permeability of double layer mix is higher compared to single layer mix. However, in terms of maximum aggregate size, the permeability of SBS and conventional mixes are not significantly different.

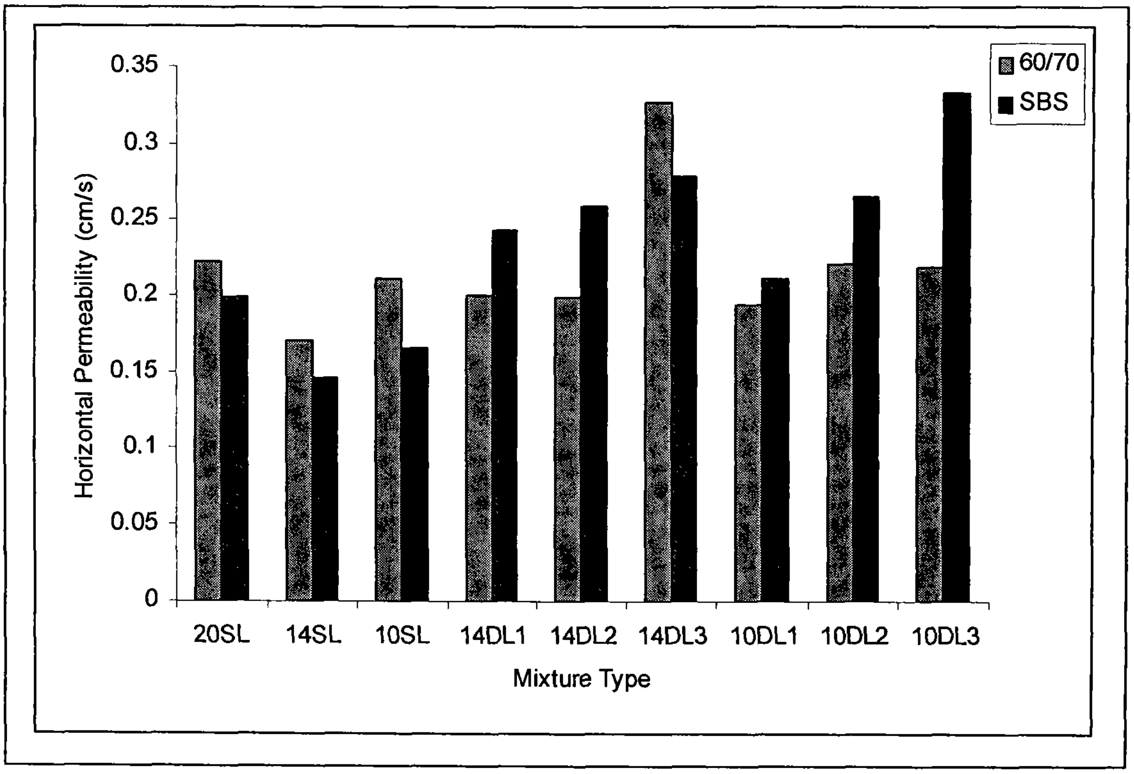

Figure 1 Coefficient of Horizontal Permeability of Porous Asphalt Slabs

7.0 CONCLUSION

The wheel tracking slab target slab density cannot be achieved using only the vibratory compactor. However, the target density of about 2.0 g/cm^3 can be achieved when the impact mode (Marshall hammer) and vibratory mode (Kango hammer) is used in combination.

The horizontal permeability for double layer is higher than the single layer. However, this value increases as the thickness of the top layer decreases. The highest horizontal permeability of double layer slab measures 0.33 cm/s.

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APPENDIX

Table 1 Mix Design for Porous Asphalt Made with Conventional Mixes

Mix Type		A	B	C	D	E	F	Lime	OPC	Total	60/70
SL-20	% wt	22.12	22.12	30.74	15.02	4.5	1.5	2	2	100	4.6
	Wt (g)	1990.8	1990.8	2766.6	1351.8	405	135	180	180	9000	
	Cumm wt (g)	**1990.8**	**3981.6**	**6748.2**	**8100**	**8505**	**8640**	**8820**	**9000**		**433.962**
SL-14	% wt	0	25.51	25.51	38.48	4.5	2	2	2	100	5
	Wt (g)	0	2272.941	2272.941	3428.568	400.95	178.2	178.2	178.2	8910	
	Cumm wt (g)	**0**	**2272.941**	**4545.882**	**7974.45**	**8375.4**	**8553.6**	**8731.8**	**8910**		**468.947**
SL-10	% wt	0	0	40.71	47.79	5	2.5	2	2	100	5.4
	Wt (g)	0	0	3608.942	4236.584	443.25	221.63	177.3	177.3	8865	
	Cumm wt (g)	**0**	**0**	**3608.942**	**7845.525**	**8288.78**	**8510.4**	**8687.7**	**8865**		**506.036**
DL1-14	% wt	0	25.51	25.51	38.48	4.5	2	2	2	100	5
	Wt (g)	0	974.1172	974.1172	1469.386	171.836	76.371	76.371	76.3714	3818.57	
	Cumm wt (g)	**0**	**974.1172**	**1948.234**	**3417.62**	**3589.46**	**3665.8**	**3742.2**	**3818.57**		**200.977**
	% wt	22.12	22.12	30.74	15.02	4.5	1.5	2	2	100	4.6
	Wt (g)	1137.601	1137.601	1580.915	772.4576	231.429	77.143	102.86	102.857	5142.86	
	Cumm wt (g)	**1137.601**	**2275.201**	**3856.116**	**4628.574**	**4860**	**4937.1**	**5040**	**5142.86**		**247.979**
DL2-14	% wt	0	25.51	25.51	38.48	4.5	2	2	2	100	5
	Wt (g)	0	649.4106	649.4106	979.5892	114.557	50.914	50.914	50.9142	2545.71	
	Cumm wt (g)	**0**	**649.4106**	**1298.821**	**2278.41**	**2392.97**	**2443.9**	**2494.8**	**2545.71**		**133.985**
	% wt	22.12	22.12	30.74	15.02	4.5	1.5	2	2	100	4.6
	Wt (g)	1422	1422	1976.142	965.5712	289.286	96.429	128.57	128.571	6428.57	
	Cumm wt (g)	**1422**	**2843.999**	**4820.142**	**5785.713**	**6075**	**6171.4**	**6300**	**6428.57**		**309.973**
DL3-14	% wt	0	25.51	25.51	38.48	4.5	2	2	2	100	5
	Wt (g)	0	487.0599	487.0599	734.6948	85.9181	38.186	38.186	38.1858	1909.29	
	Cumm wt (g)	**0**	**487.0599**	**974.1198**	**1708.815**	**1794.73**	**1832.9**	**1871.1**	**1909.29**		**100.489**
	% wt	22.12	22.12	30.74	15.02	4.5	1.5	2	2	100	4.6
	Wt (g)	1564.2	1564.2	2173.758	1062.129	318.214	106.07	141.43	141.429	7071.43	
	Cumm wt (g)	**1564.2**	**3128.401**	**5302.158**	**6364.287**	**6682.5**	**6788.6**	**6930**	**7071.43**		**340.97**
DL1-10	% wt	0	0	40.71	47.79	5	2.5	2	2	100	5.4
	Wt (g)	0	0	1546.691	1815.681	189.965	94.982	75.986	75.9858	3799.29	
	Cumm wt (g)	**0**	**0**	**1546.691**	**3362.372**	**3552.34**	**3647.3**	**3723.3**	**3799.29**		**216.873**
	% wt	22.12	22.12	30.74	15.02	4.5	1.5	2	2	100	4.6
	Wt (g)	1137.601	1137.601	1580.915	772.4576	231.429	77.143	102.86	102.857	5142.86	
	Cumm wt (g)	**1137.601**	**2275.201**	**3856.116**	**4628.574**	**4860**	**4937.1**	**5040**	**5142.86**		**247.979**
DL2-10	% wt	0	0	40.71	47.79	5	2.5	2	2	100	5.4
	Wt (g)	0	0	1031.127	1210.454	126.643	63.322	50.657	50.6572	2532.86	
	Cumm wt (g)	**0**	**0**	**1031.127**	**2241.581**	**2368.22**	**2431.5**	**2482.2**	**2532.86**		**144.582**
	% wt	22.12	22.12	30.74	15.02	4.5	1.5	2	2	100	4.6
	Wt (g)	1422	1422	1976.142	965.5712	289.286	96.429	128.57	128.571	6428.57	
	Cumm wt (g)	**1422**	**2843.999**	**4820.142**	**5785.713**	**6075**	**6171.4**	**6300**	**6428.57**		**309.973**
DL3-10	% wt	0	0	40.71	47.79	5	2.5	2	2	100	5.4
	Wt (g)	0	0	773.3434	907.838	94.982	47.491	37.993	37.9928	1899.64	
	Cumm wt (g)	**0**	**0**	**773.3434**	**1681.181**	**1776.16**	**1823.7**	**1861.6**	**1899.64**		**108.436**
	% wt	22.12	22.12	30.74	15.02	4.5	1.5	2	2	100	4.6
	Wt (g)	1564.2	1564.2	2173.758	1062.129	318.214	106.07	141.43	141.429	7071.43	
	Cumm wt (g)	**1564.2**	**3128.401**	**5302.158**	**6364.287**	**6682.5**	**6788.6**	**6930**	**7071.43**		**340.97**

Table 2 Mix Design for Porous Asphalt Made with SBS Mixes

Mix Type		A	B	C	D	E	F	Lime	OPC	Total	SBS
SL-20	% wt	22.12	22.12	30.74	15.02	4.5	1.5	2	2	100	4.6
	Wt (g)	2000.754	2000.754	2780.433	1358.559	407.025	135.68	180.9	180.9	9045	
	Cumm wt (g)	**2000.754**	**4001.508**	**6781.941**	**8140.5**	**8547.53**	**8683.2**	**8864.1**	**9045**		**436.132**
SL-14	% wt	0	25.51	25.51	38.48	4.5	2	2	2	100	5.2
	Wt (g)	0	2272.941	2272.941	3428.568	400.95	178.2	178.2	178.2	8910	
	Cumm wt (g)	**0**	**2272.941**	**4545.882**	**7974.45**	**8375.4**	**8553.6**	**8731.8**	**8910**		**488.734**
SL-10	% wt	0	0	40.71	47.79	5	2.5	2	2	100	5.7
	Wt (g)	0	0	3590.622	4215.078	441	220.5	176.4	176.4	8820	
	Cumm wt (g)	**0**	**0**	**3590.622**	**7805.7**	**8246.7**	**8467.2**	**8643.6**	**8820**		**533.128**
DL1-14	% wt	0	25.51	25.51	38.48	4.5	2	2	2	100	5.2
	Wt (g)	0	974.1172	974.1172	1469.386	171.836	76.371	76.371	76.3714	3818.57	
	Cumm wt (g)	**0**	**974.1172**	**1948.234**	**3417.62**	**3589.46**	**3665.8**	**3742.2**	**3818.57**		**209.457**
	% wt	22.12	22.12	30.74	15.02	4.5	1.5	2	2	100	4.6
	Wt (g)	1143.288	1143.288	1588.818	776.3192	232.586	77.529	103.37	103.371	5168.57	
	Cumm wt (g)	**1143.288**	**2286.575**	**3875.394**	**4651.713**	**4884.3**	**4961.8**	**5065.2**	**5168.57**		**249.218**
DL2-14	% wt	0	25.51	25.51	38.48	4.5	2	2	2	100	5.2
	Wt (g)	0	649.4106	649.4106	979.5892	114.557	50.914	50.914	50.9142	2545.71	
	Cumm wt (g)	**0**	**649.4106**	**1298.821**	**2278.41**	**2392.97**	**2443.9**	**2494.8**	**2545.71**		**139.638**
	% wt	22.12	22.12	30.74	15.02	4.5	1.5	2	2	100	4.6
	Wt (g)	1429.109	1429.109	1986.022	970.3986	290.732	96.911	129.21	129.214	6460.71	
	Cumm wt (g)	**1429.109**	**2858.218**	**4844.24**	**5814.639**	**6105.37**	**6202.3**	**6331.5**	**6460.71**		**311.523**
DL3-14	% wt	0	25.51	25.51	38.48	4.5	2	2	2	100	5.2
	Wt (g)	0	487.0599	487.0599	734.6948	85.9181	38.186	38.186	38.1858	1909.29	
	Cumm wt (g)	**0**	**487.0599**	**974.1198**	**1708.815**	**1794.73**	**1832.9**	**1871.1**	**1909.29**		**104.729**
	% wt	22.12	22.12	30.74	15.02	4.5	1.5	2	2	100	4.6
	Wt (g)	1572.022	1572.022	2184.627	1067.44	319.806	106.6	142.14	142.136	7106.79	
	Cumm wt (g)	**1572.022**	**3144.044**	**5328.671**	**6396.111**	**6715.92**	**6822.5**	**6964.7**	**7106.79**		**342.675**
DL1-10	% wt	0	0	40.71	47.79	5	2.5	2	2	100	5.7
	Wt (g)	0	0	1538.838	1806.462	189	94.5	75.6	75.6	3780	
	Cumm wt (g)	**0**	**0**	**1538.838**	**3345.3**	**3534.3**	**3628.8**	**3704.4**	**3780**		**228.484**
	% wt	22.12	22.12	30.74	15.02	4.5	1.5	2	2	100	4.6
	Wt (g)	1143.288	1143.288	1588.818	776.3192	232.586	77.529	103.37	103.371	5168.57	
	Cumm wt (g)	**1143.288**	**2286.575**	**3875.394**	**4651.713**	**4884.3**	**4961.8**	**5065.2**	**5168.57**		**249.218**
DL2-10	% wt	0	0	40.71	47.79	5	2.5	2	2	100	5.7
	Wt (g)	0	0	1025.892	1204.308	126	63	50.4	50.4	2520	
	Cumm wt (g)	**0**	**0**	**1025.892**	**2230.2**	**2356.2**	**2419.2**	**2469.6**	**2520**		**152.322**
	% wt	22.12	22.12	30.74	15.02	4.5	1.5	2	2	100	4.6
	Wt (g)	1429.109	1429.109	1986.022	970.3986	290.732	96.911	129.21	129.214	6460.71	
	Cumm wt (g)	**1429.109**	**2858.218**	**4844.24**	**5814.639**	**6105.37**	**6202.3**	**6331.5**	**6460.71**		**311.523**
DL3-10	% wt	0	0	40.71	47.79	5	2.5	2	2	100	5.7
	Wt (g)	0	0	769.419	903.231	94.5	47.25	37.8	37.8	1890	
	Cumm wt (g)	**0**	**0**	**769.419**	**1672.65**	**1767.15**	**1814.4**	**1852.2**	**1890**		**114.242**
	% wt	22.12	22.12	30.74	15.02	4.5	1.5	2	2	100	4.6
	Wt (g)	1572.022	1572.022	2184.627	1067.44	319.806	106.6	142.14	142.136	7106.79	
	Cumm wt (g)	**1572.022**	**3144.044**	**5328.671**	**6396.111**	**6715.92**	**6822.5**	**6964.7**	**7106.79**		**342.675**

BINDER CREEP, EFFECTS OF CRABIT AND WHEEL TRACKING TEST RESULTS ON POROUS ASPHALT MIXTURES

REPORT-8

MARCH 2006

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1.0 INTRODUCTION

It is widely accepted that porous asphalts are susceptible to permeability loss in service. The main source of permeability loss stems from voids closure due to traffic overcompaction and clogging of internal voids that were once interconnected. However, an interesting phenomenon was discovered in the course of this research. Old unextruded samples of porous asphalt specimens were tested for permeability and their coefficient of permeabilities were found to be significantly lower than when the permeabilities were first measured upon sample preparation. A new source of permeability loss was tentatively identified as binder creep. This phenomenon was presented by the author during his research attachment at Delft University. The encouraging support, in addition to the potential of a newly developed binder additive named CraBit as explained underneath, initiated an extension to this IRPA research project.

In the course of this research as well, a separate binder study was carried out. The new binder developed was based on crumb rubber with the addition of additives that would facilitate mixing with aggregates via the dry process. However, the binder was tested for dense ACW14 asphaltic concrete mixes. It was only after the binder was rigorously tested and found to exhibit favourable performance with dense mixes that the new binder additive, CraBit, was tested on porous mixes.

2.0 OBJECTIVES

1. To assess the phenomenon of binder creep by monitoring permeability loss over extended period of time.
2. To evaluate the engineering properties and performance of single and double layer porous asphalt prepared using CraBit blended with asphalt via the dry mixing process.
3. To report wheel tracking test results.

3.0 SCOPE OF WORK

In this last stage of the laboratory work, the scope of work focus on an assessment of permeability loss due to binder creep on single layer porous asphalt mix each made with 20 mm, 14 mm and 10 mm maximum aggregate size. The phenomenon of binder creep was tested on conventional 60/70 and SBS modified single layer porous asphalt mixes only. The effects of CraBit on single and double layer porous asphalt mixes prepared at the DBC was assessed from performance tests that include resistance to abrasion loss, permeability, stiffness modulus and overcompaction. Development in the wheel tracking test results is also presented in this report.

4.0 METHODOLOGY

4.1 Materials

All materials used in this investigation for the binder creep and CraBit modified mix performance tests were as described in earlier reports. An exception is CraBit but its initial development and composition has been described in Report 3 (Hamzah, 2004a). CraBit has been earlier tested and found to enhance engineering properties and performance of dense asphaltic concrete mixes.

4.2 Porous Asphalt Specimen Preparation and Evaluation Tests

Specimen preparation for porous asphalt was as described in earlier reports. All specimens, with the exception of those evaluated for binder creep, were prepared at their respective design binder content (DBC). The DBC for 10, 14 and 20 mm maximum size porous mixes were respectively 5.4, 5.0 and 4.5%. However, mixes incorporating Crabit were added via the dry process. In the dry mix process, aggregates were first blended dry for 30 seconds in an asphalt mixer. Then, the conventional 60/70 bitumen was mixed with the aggregate. Once the binder had amply coated the aggregates, CraBit, in 1% quantity, was added into the mixer and blended for another one minute. Specimens were then compacted via impact mode once the temperature dropped to their desired compaction temperatures. The mixing and compaction temperatures were as in previous investigation.

4.3 Binder Creep Test

In the binder creep test, freshly prepared specimens that have cooled down but still confined within the Marshall mould were tested for permeability using the water permeameter. Three readings were taken and the average results were reported. The resultant average permeability is described as virgin or initial permeability which is the permeability at day 1. Upon termination of permeability measurement at day 1, specimens were kept covered to eliminate intrusion by dust particles. Then, the specimen permeability measurements were repeated at day 2, 3, 5, 7, 10, 15, 20, 30, 44, 50, 70, 80, 90 and 120.

4.4 Tests to Evaluate the Effectiveness of CraBit

To test the effectiveness of CraBit, single layer and double layer porous asphalt specimens were subjected to the performance tests described in Report 6 (Hamzah, 2004b). However, in the overcompaction test, specimens conditioned at 60°C were subjected to 600 kPa overcompaction pressure, 300 gyrations and 2 degree angle of gyration. This represented an overcompaction pressure categorised as 'high'.

4.5 Wheel Tracking Test

The wheel tracking test methodology has been described in Report 7 (Hamzah, 2005).

5.0 RESULTS AND DISCUSSION

5.1 Binder Creep

The binder creep test results for mixes prepared using conventional binder is shown in Figures 1 through 3 while Figure 4 through 6 are the results for SBS modified mixes. The results for all tests conducted on specimens prepared at the DBC were carried out by Mohd Tahir (2006).

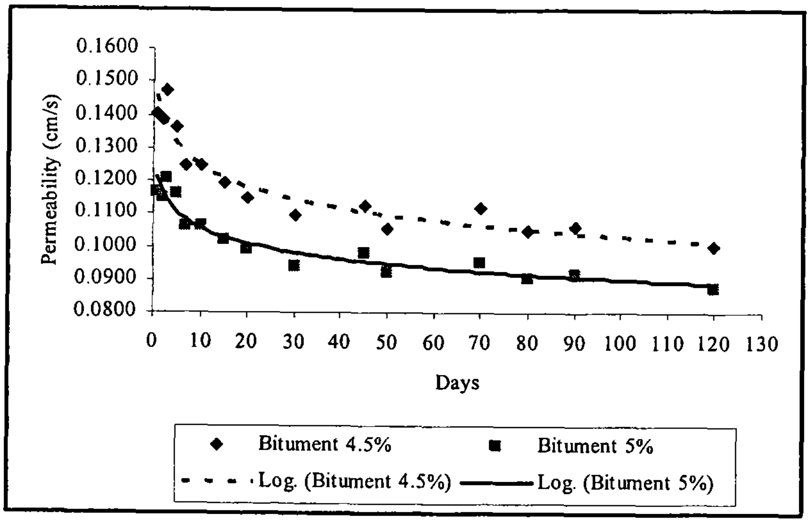

Figure 1 Permeability Loss Due to Binder Creep (Maximum Aggregate Size 20 mm 60/70 Bitumen)

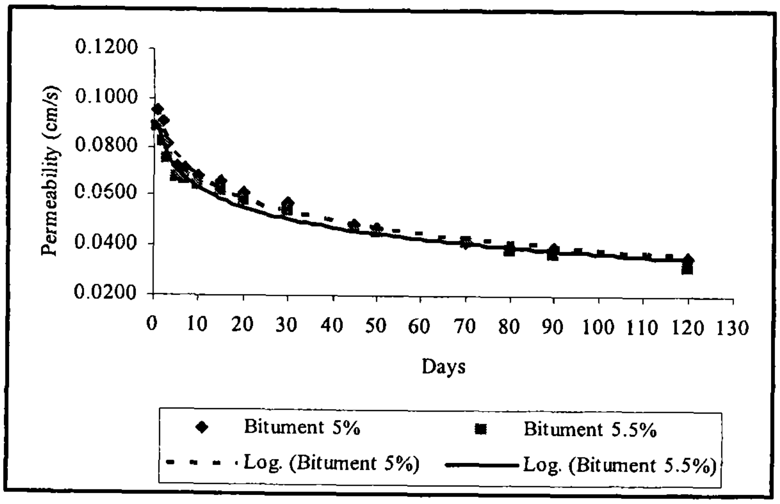

Figure 2 Permeability Loss Due to Binder Creep (Maximum Aggregate Size 14 mm 60/70 Bitumen)

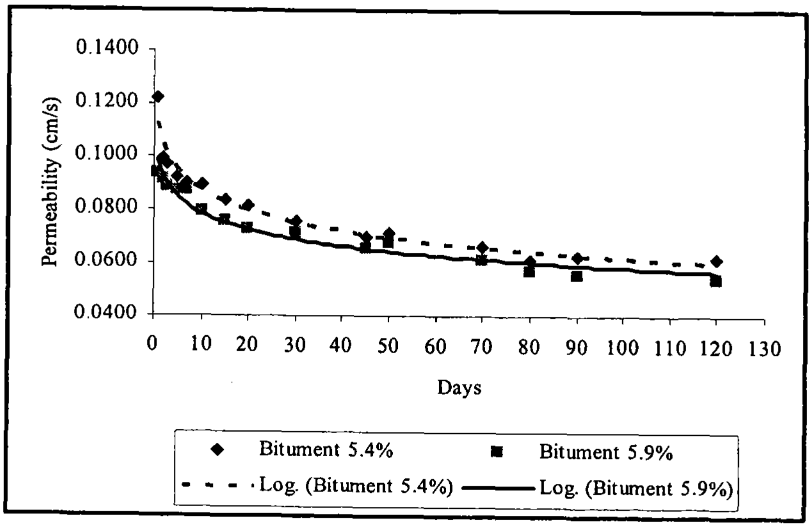

Figure 3 Permeability Loss Due to Binder Creep (Maximum Aggregate Size 10 mm 60/70 Bitumen)

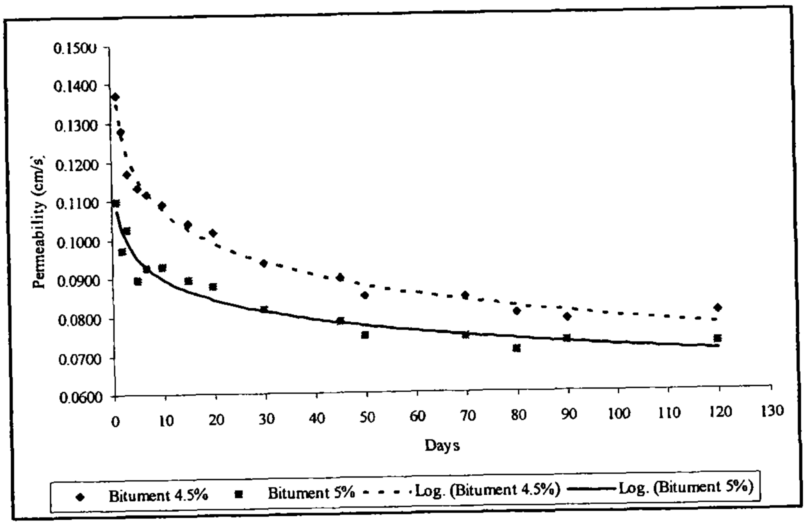

Figure 4 Permeability Loss Due to Binder Creep (Maximum Aggregate Size 20 mm SBS Bitumen)

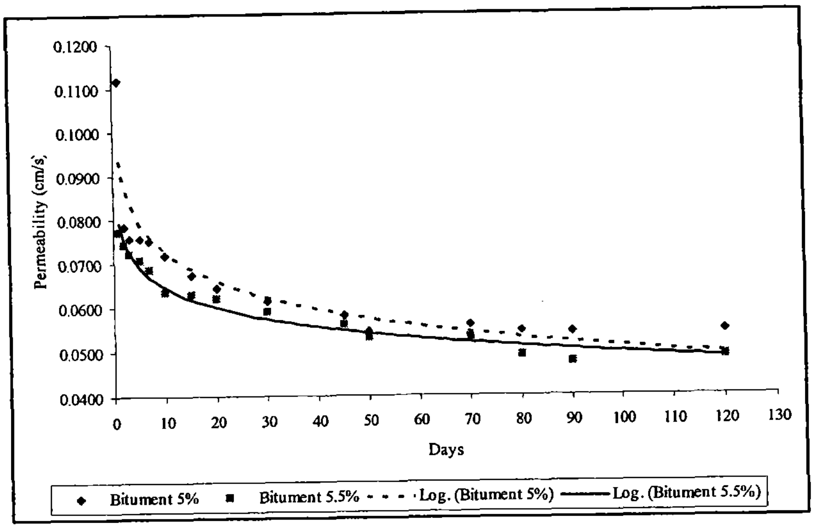

Figure 5 Permeability Loss Due to Binder Creep (Maximum Aggregate Size 14 mm SBS Bitumen)

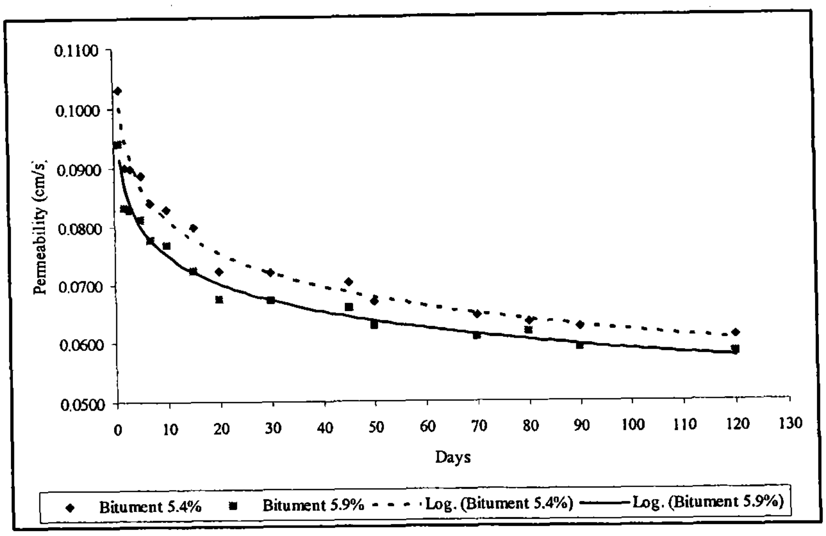

Figure 6 Permeability Loss Due to Binder Creep (Maximum Aggregate Size 10 mm SBS Bitumen)

The permeability loss results expressed in terms of discharge time is shown in Appendix A. The test results shown in Figures 1 through 6 indicate a clear advantage of using lower binder content by securing a higher initial permeability and lower net permeability loss over time. However, regardless of binder content, binder type or mix composition, the coefficient of permeability reduces with time. The reduction is more pronounced during the initial stages beyond which it tends to level off. The last permeability readings were recorded at day 120. At this instant, the permeability values are found to be steadily decreasing though not as drastic as during the initial stage.

Higher binder contents tend to fill up voids, disrupting voids continuity and a subsequent reduction in permeability. This trend is discernible in all mixes tested. Mixes with higher binder content exhibit lower initial permeability but the rate of permeability drop with time appear to be similar to mixes with lower binder contents. From the initial slope, the rate of permeability drop of the SBS modified mixes appears to be more significant compared to the conventional mixes.

It could be seen that some mixes can loose out by nearly 50% of their original or virgin permeabilities after 120 days and this represent a significant amount of permeability reduction keeping in view the relatively short time lapse. Further investigation is needed to ascertain the source of permeability loss of the specimens tested though it was initially postulated that over time, the binder in the asphalt mix tends to creep into available cavities subsequently blocking some of the air passages. One possibility is the presence of minute suspended matter in the water that acted as the permeant. Over time, the suspended matter accumulated in the pores of the porous mixes, blocking some voids, disrupting their continuities and hence a steady reduction in permeability.

5.2 Performance of Porous Asphalt Incorporating CraBit

Mix properties and performance of porous mixes incorporating CraBit were assessed from the Marshall stability, permeability, Cantabro, resilient modulus, creep and overcompaction test results and comparing with previous test results carried out on 60/70 single and double layer porous mixes without CraBit. The corresponding test results for conventional mixes without CraBit are summarized in Table 1.

Table 1 Summary Test Results of Previous Study on Single and Double Layer Porous Asphalt Without CraBit (Hamzah, 2004c, Hamzah, 2004d)

Specimen Designation	**Marshall Stability (kN)**	**Permeability (cm/s)**	**Average Discharge Time (s)**	**Abrasion Loss (%)**	**Resilient Modulus (MPa)**	**Creep Stiffness (MPa)**
20S	5.12	0.171	-	10.49	1850.05	19.58
14S	5.26	0.136	-	6.5	1708.33	24.08
10S	5.34	0.126	-	4.11	1573.08	13.32
14/20D1(30/40)	4.97	0.127*	61.2	6.10	1779.5	23.4
14/20D2(20/50)	5.41	0.128*	56.7	6.37	1848.7	21.3
14/20D3(15/55)	5.98	0.133*	54.9	7.76	1886.8	15.8
10/20D1(30/40)	4.72	0.135*	54.0	5.74	1613.4	15.1
10/20D2(20/50)	5.85	0.135*	52.8	6.34	1642.4	20.4
10/20D3(15/55)	5.50	0.138*	52.1	6.56	1924.1	14.4

* Approximate coefficient of permeability

5.2.1 Marshall Stability

The Marshall stability results for single and double layer mixes incorporating CraBit are respectively shown in Figures 7 and 8.

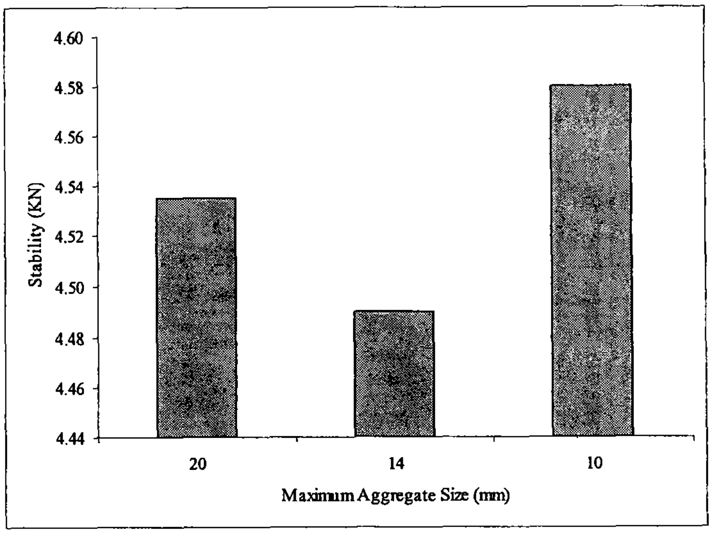

Figure 7 Marshall Stability Results for Single Layer Porous Asphalt

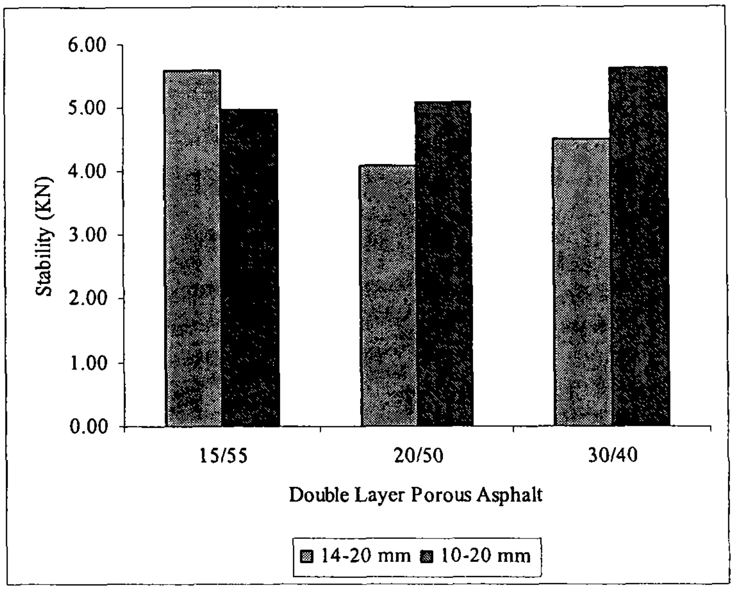

Figure 8 Marshall Stability Results for Double Layer Porous Asphalt

5.2.2 Permeability

The corresponding permeability results are shown in Figures 9 and 10.

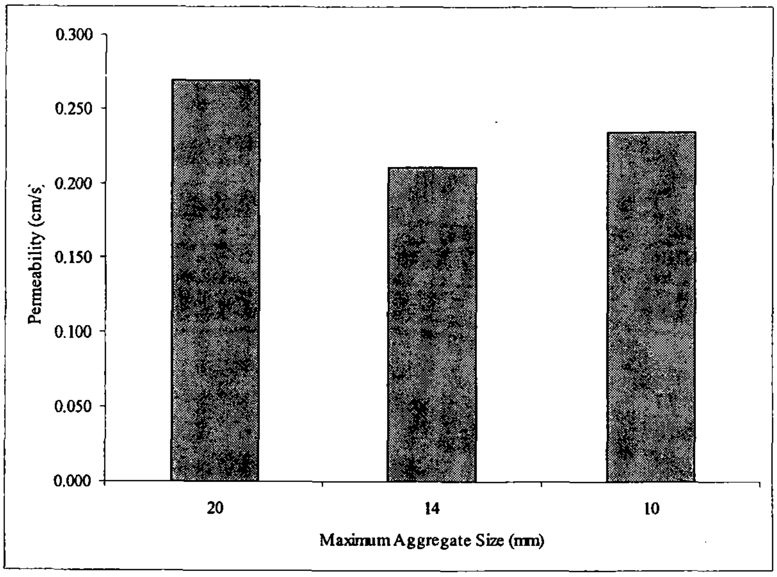

Figure 9 Permeability Results for Single Layer Porous Asphalt

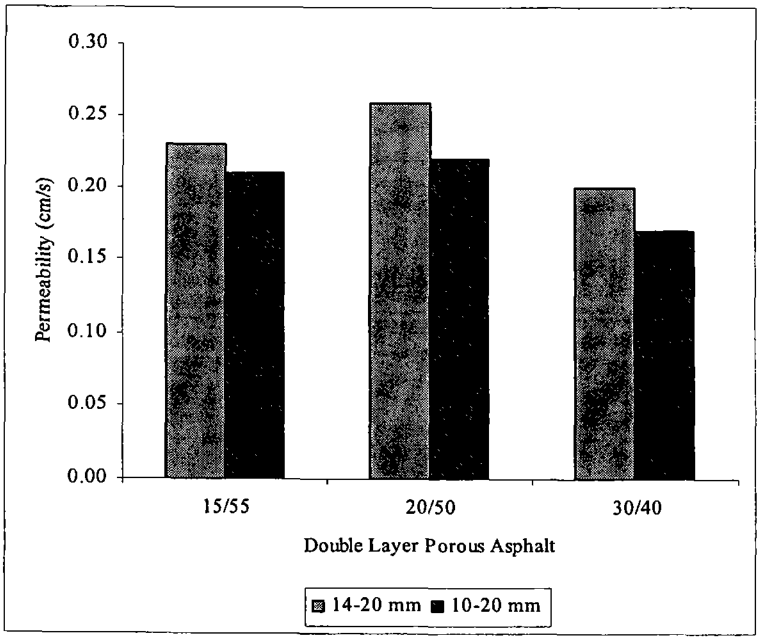

Figure 10 Permeability Results for Double Layer Porous Asphalt

5.2.3 Resistance to Disintegration

The resistance to disintegration of single porous asphalt mixes incorporating CraBit as measured via the Cantabro test is shown in Figure 11 while the results for identical test conducted on double layer porous asphalt is shown in Figure 12.

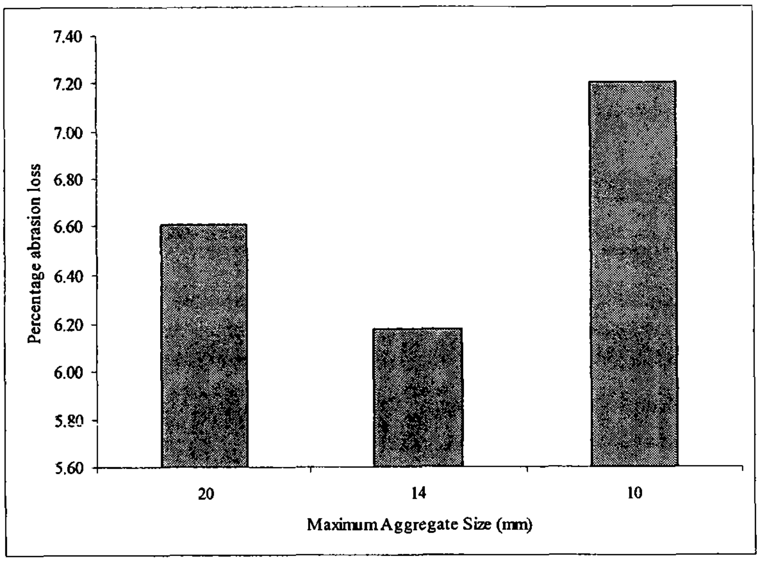

Figure 11 The Cantabro Test Results on Single Layer Porous Asphalt

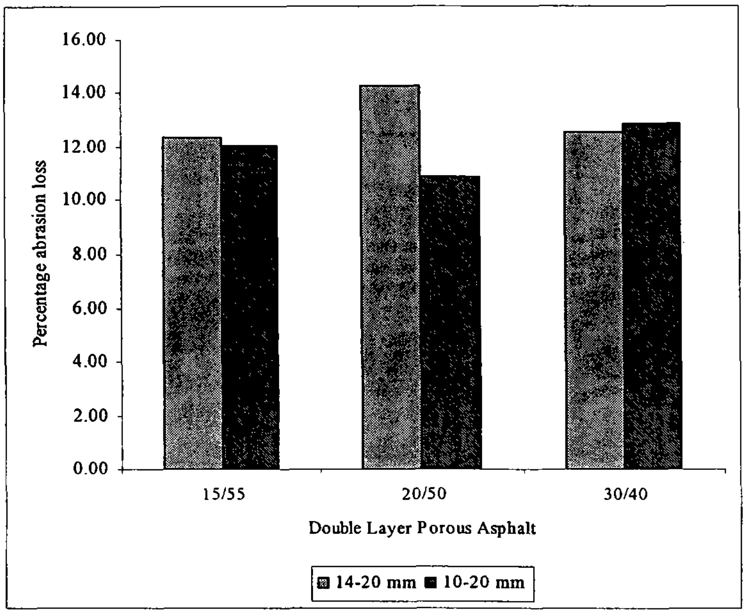

Figure 12 The Cantabro Test Results on Double Layer Porous Asphalt

5.2.4 Resilient Modulus

The corresponding resilient modulus results are shown in Figures 13 and 14.

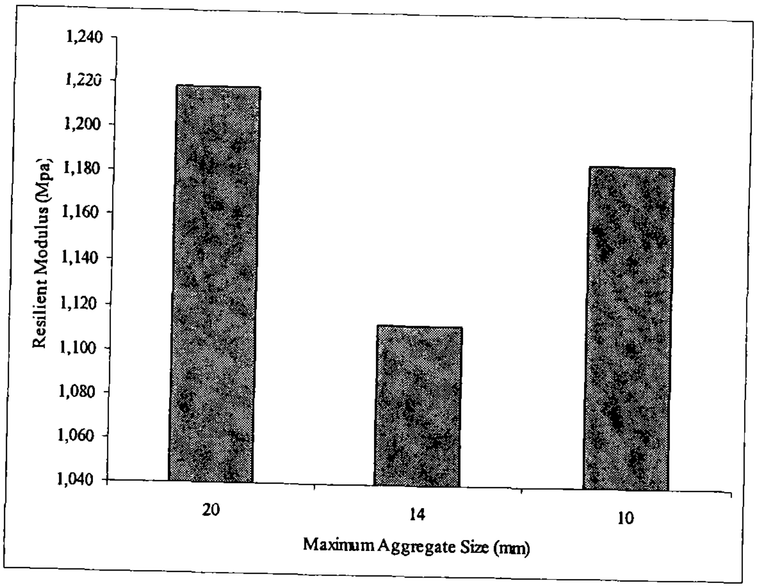

Figure 13 The Cantabro Test Results on Single Layer Porous Asphalt

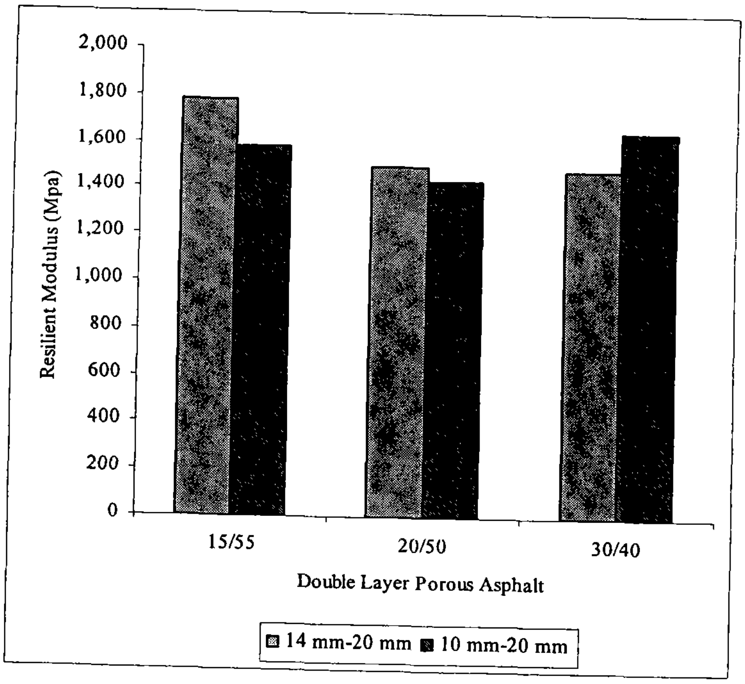

Figure 14 The Cantabro Test Results on Double Layer Porous Asphalt

5.2.5 Creep Stiffness

The dynamic creep stiffness test results on both single and double layer porous asphalt are shown in Figures 15 and 16 respectively.

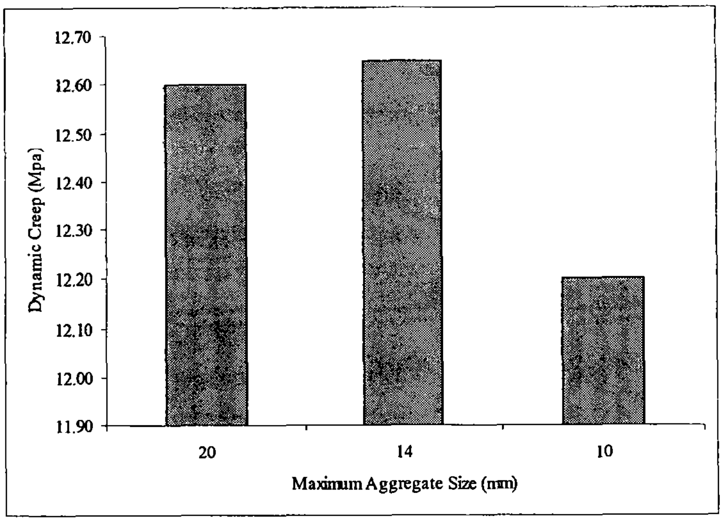

Figure 15 The Dynamic Creep Stiffness Test Results on Single Layer Porous Asphalt

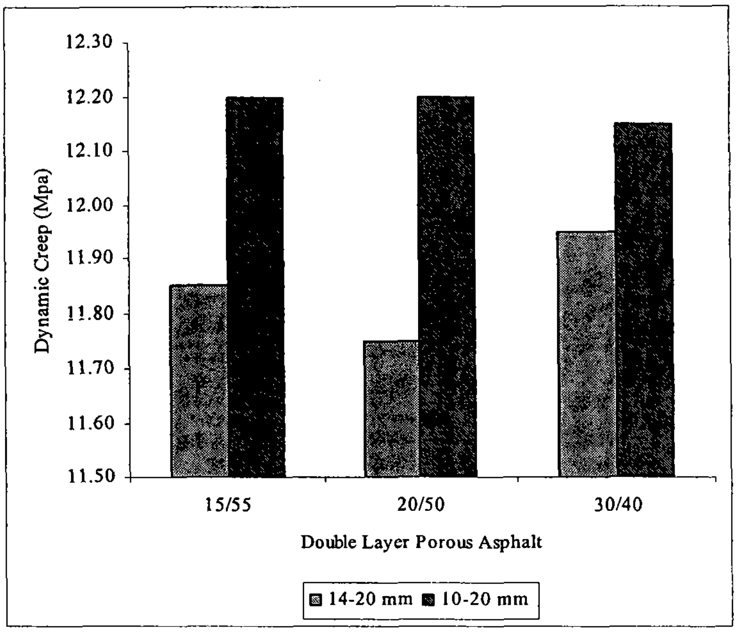

Figure 16 The Dynamic Creep Stiffness Test Results on Double Layer Porous Asphalt

5.2.6 Resistance to Overcompaction

The overcompaction test results on mixes subjected to high overcompaction pressure are shown in Figures 17 and 18.

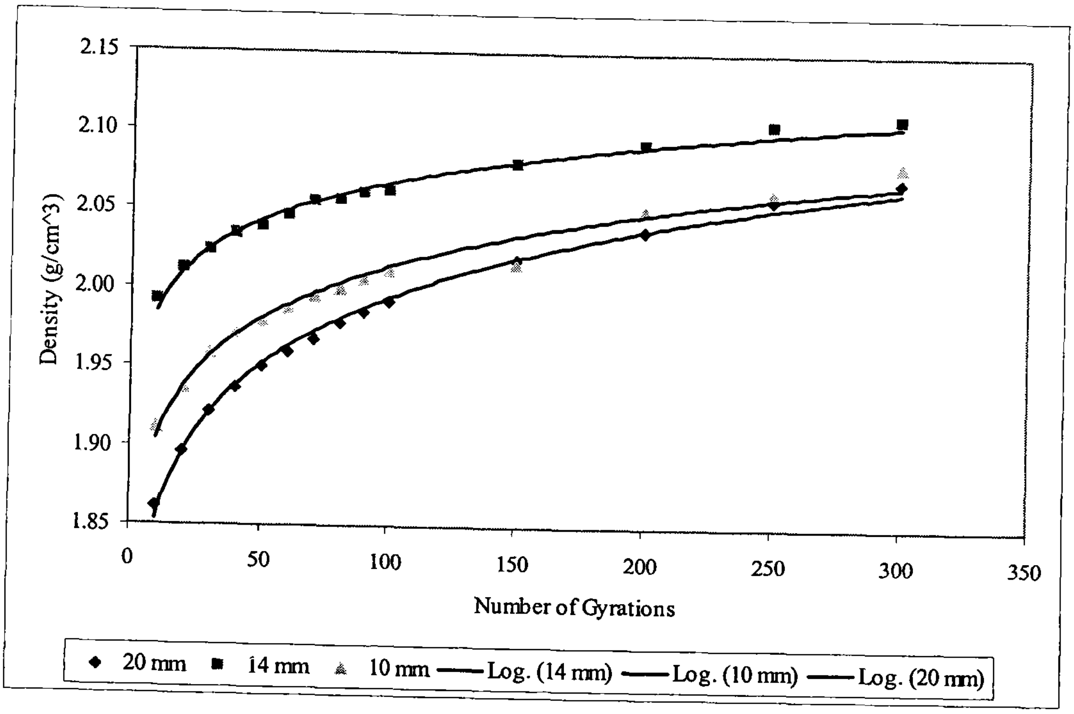

Figure 17 Mixture Densification as a Consequence of Overcompaction on Single Layer Porous Asphalt

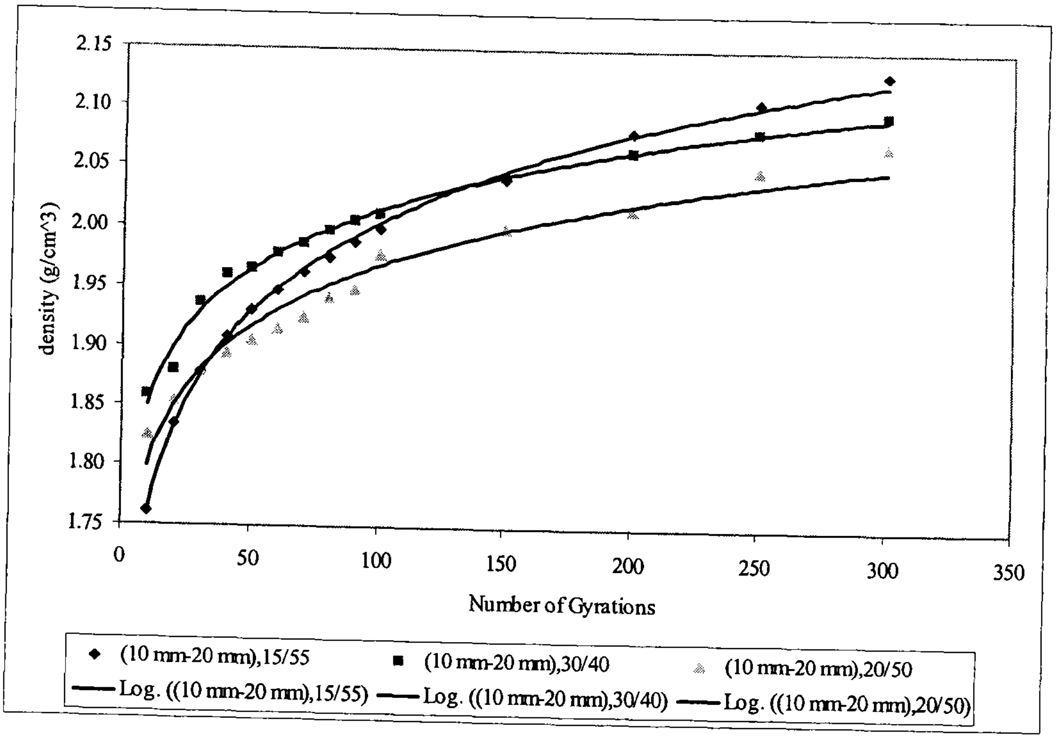

Figure 18 Mixture Densification as a Consequence of Overcompaction on Double Layer Porous Asphalt

5.2.7 Comparison of Results with Mixes Without CraBit

From the Marshall test, the stabilities of all single layer mixes incorporating CraBit register lower stability values. Nevertheless, the reduction is not significant. On the other hand, the double layer mixes incorporating CraBit exhibit similar stability values than the conventional mixes.

When tested for permeability, all CraBit mixes are more permeable compared to base bitumen mixes. While this is advantageous to resist clogging, it may lead to reduced resistance to abrasion loss and lower mix stiffness.

Comparing the abrasion loss results of Table 1 and Figures 11 and 12, porous mixes incorporating CraBit are generally less resistant to disintegration. This is more pronounced with the double layer mixes. In some instances, the abrasion loss of the double layer CraBit porous mixes is twofold that of the conventional double layer mixes. However, the proposed JKR specifications (Harun, 2005) specified an abrasion loss not more than 15% when tested at 15°C in the Los Angeles drum. None of the mixes with CraBit that were tested suffered from abrasion loss greater than 15% and hence conform to the JKR specifications.

Single layer porous mixes incorporating CraBit generally exhibit lower resilient modulus compared with equivalent mixes without CraBit. However, the resilient modulus of the double layer mixes with and without CraBit is not significantly different. The creep stiffness test results also indicate general lower stiffness values of CraBit mixes compared to mixes without CraBit.

In general, there is no obvious advantage in incorporating CraBit in both single and double layer mixes from the perspective of enhancing mix performance. However, CraBit was developed for dense mixes and has been proven to improve mix properties particularly upon subjecting to short term and long term ageing. However, tests on aged porous mixes were not carried out due to the limited time available.

5.3 Wheel Tracking Test Results

The wheel tracking test results is normally expressed in terms of the development of rutting over tracking time. Figure 19 shows a typical result for single layer slab prepared using 60/70 binder and 20 mm maximum aggregate size. The remaining test results are available in Appendix B. It could be seen that the computer has the ability to register many readings along the slab. Unfortunately, not only is the final rut depth after one hour tracking is very small, the rut depth along the tracked section is not consistent. This explains the presence of positive and negative values. Other results obtained from tracking the remaining slabs as shown in Appendix B also indicate similar pattern. It is not possible to compute the rut depth at the centre of the slab from any of the readings and graphs obtained. Efforts are being made to fabricate a simple device to manually measure the centre rut depth using a dial gauge. Alternative, modification to the wheel tracking machine rut depth recording device is essential so that the centre reading can be distinctively isolated from the rest.

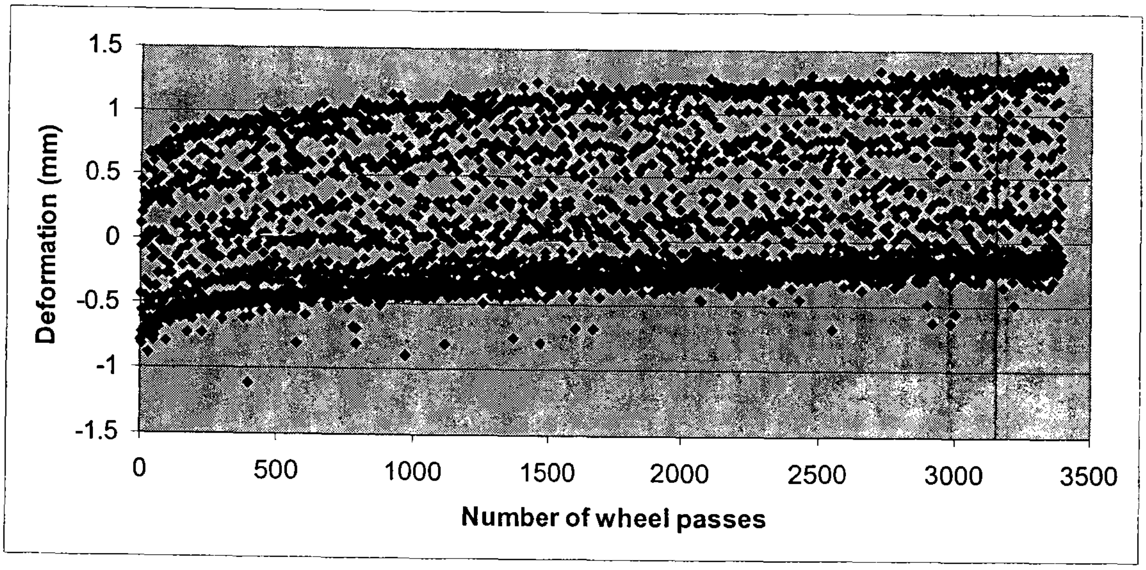

Figure 19 A Typical Wheel Tracking Test Rut Depth Distribution Along Tracked Section (20S 60/70)

6.0 SUMMARY

Another source of permeability loss; apart from traffic overcompaction and clogging, has been postulated to be due to binder creep. Mixes can loose a significant proportion of their permeabilities without being subjected to any applied load. From a comparison between the test results on single and double layer porous asphalt with and without CraBit, mixes without CraBit appear to exhibit slight advantage in terms of performance. Despite this, the abrasion loss of mixes incorporating CraBit is within the permitted values given in the JKR specifications. Further investigation is suggested to ascertain the source of permeability loss apart from binder creep, traffic overcompaction and clogging. The additive CraBit contains anti-oxidant that was developed to enhance short term and long term ageing of dense asphaltic concrete. Further investigation on performance of aged porous asphalt mixtures with CraBit is recommended.

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APPENDIX A

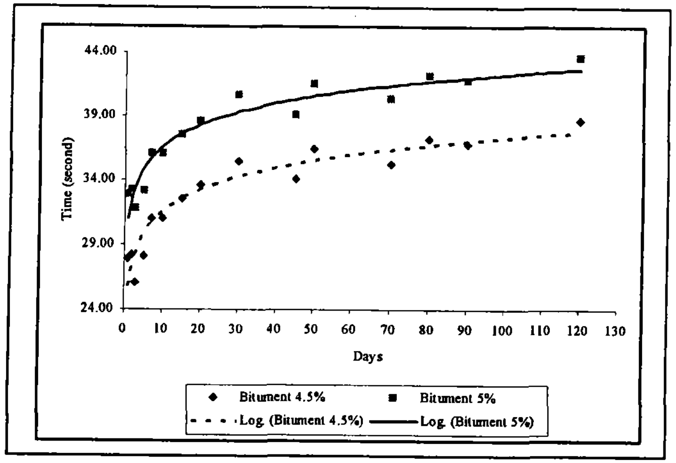


Figure A1 Permeability Loss 20 mm Conventional Binder

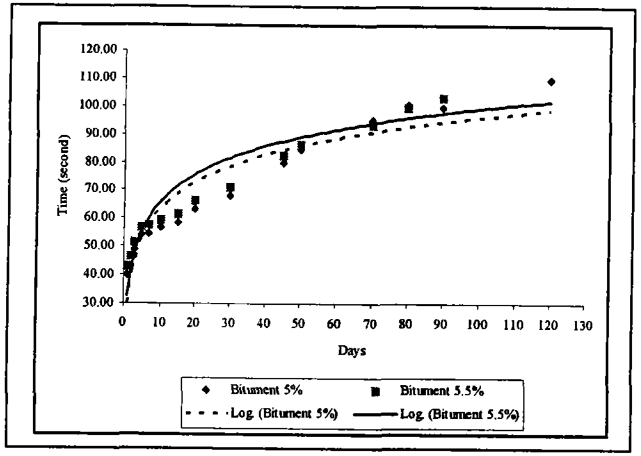


Figure A2 Permeability Loss 14 mm Conventional Binder

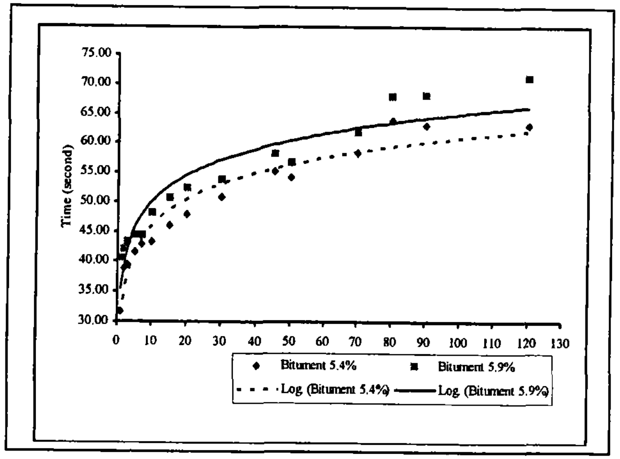


Figure A3 Permeability Loss 10 mm Conventional Binder

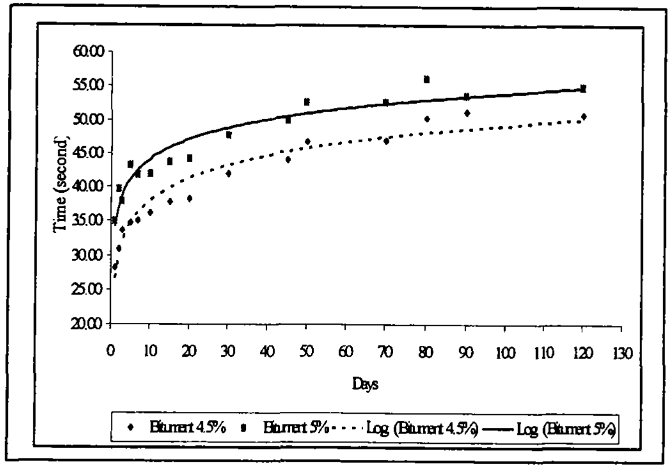


Figure A4 Permeability Loss 20 mm Conventional Binder

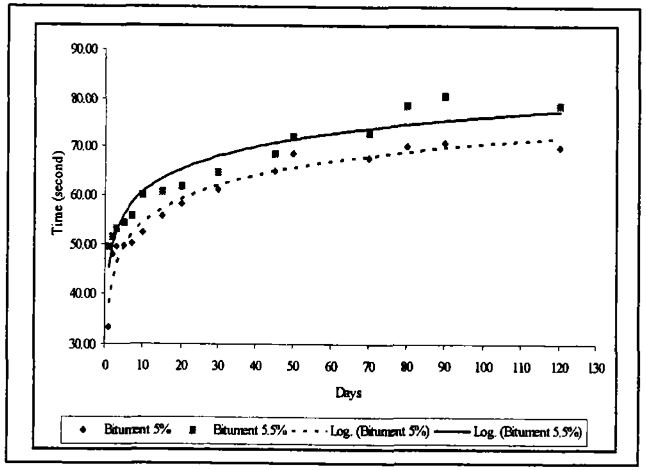


Figure A5 Permeability Loss 14 mm Conventional Binder

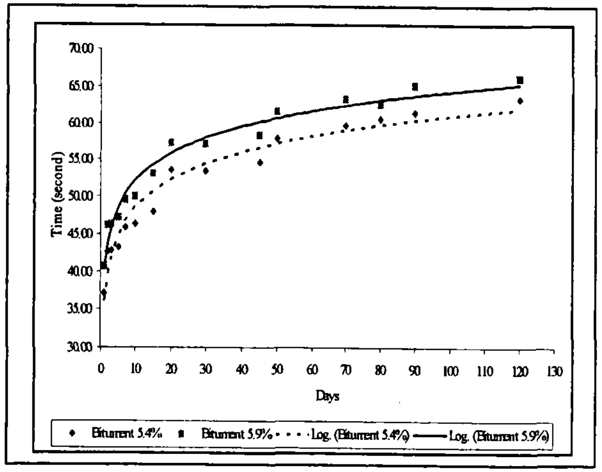


Figure A6 Permeability Loss 10 mm Conventional Binder

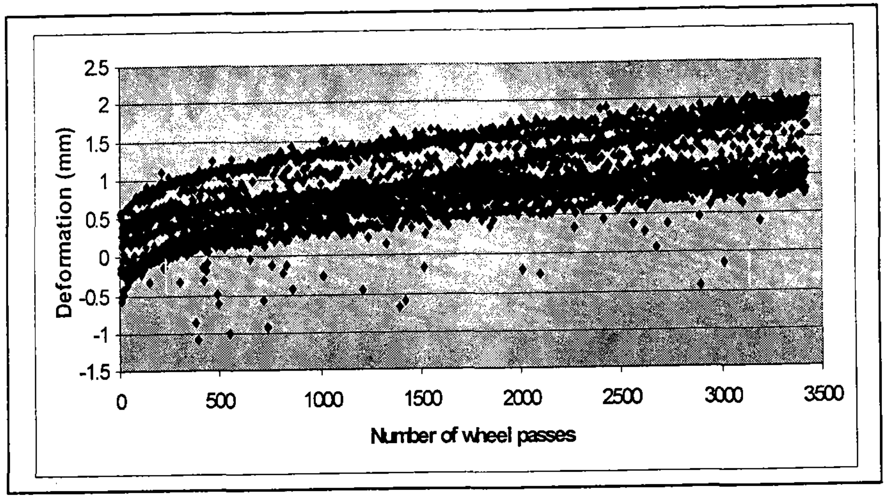


Figure B1 Rut Depth Distribution Along Tracked Slab Section (14S-60/70)

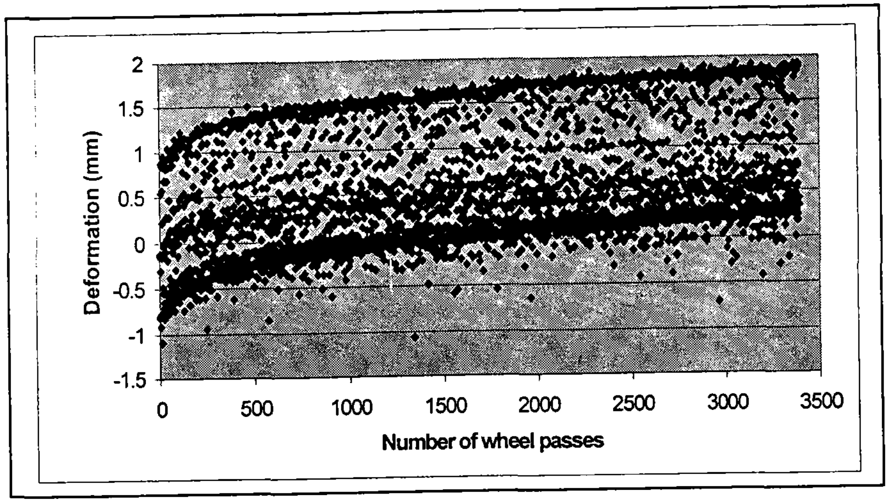


Figure B2 Rut Depth Distribution Along Tracked Slab Section (10S-60/70)

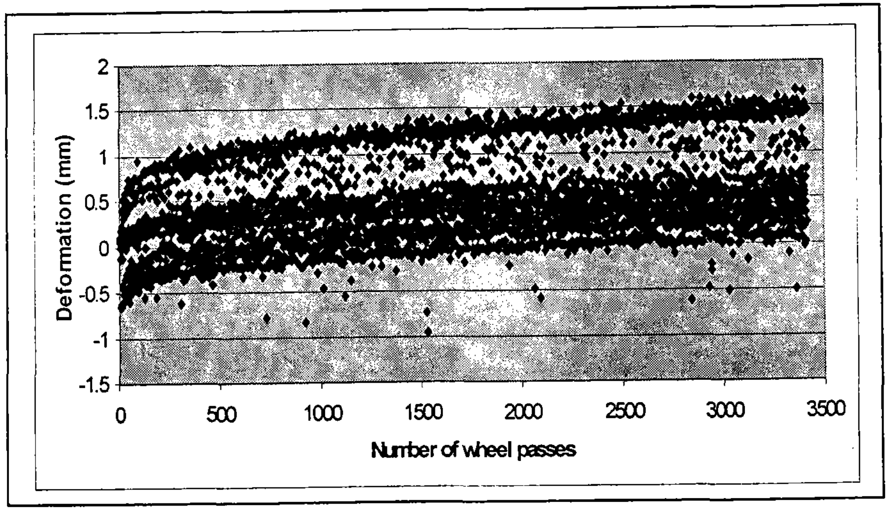


Figure B3 Rut Depth Distribution Along Tracked Slab Section (14D1-60/70)

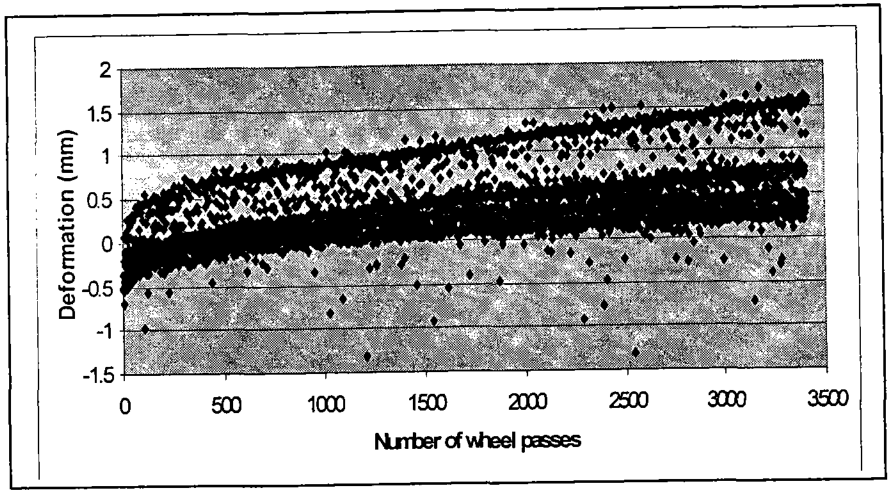


Figure B4 Rut Depth Distribution Along Tracked Slab Section (14D2-60/70)

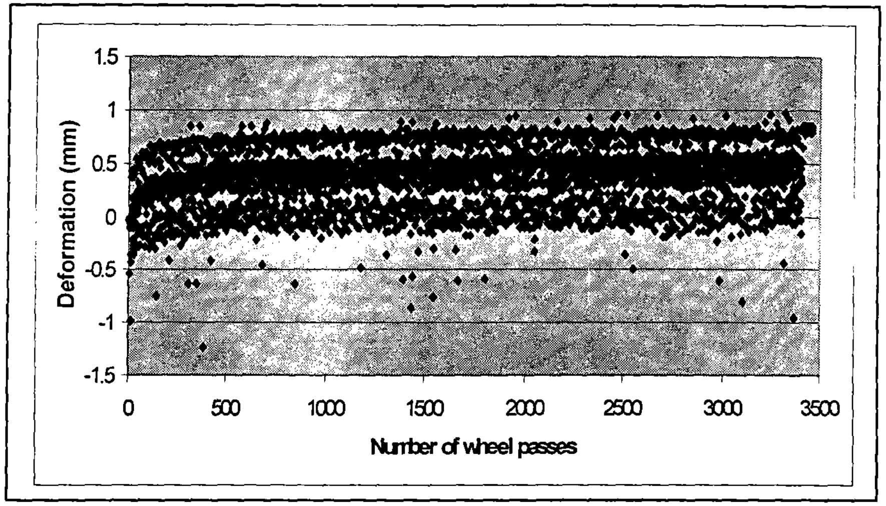


Figure B5 Rut Depth Distribution Along Tracked Slab Section (14D3-60/70)

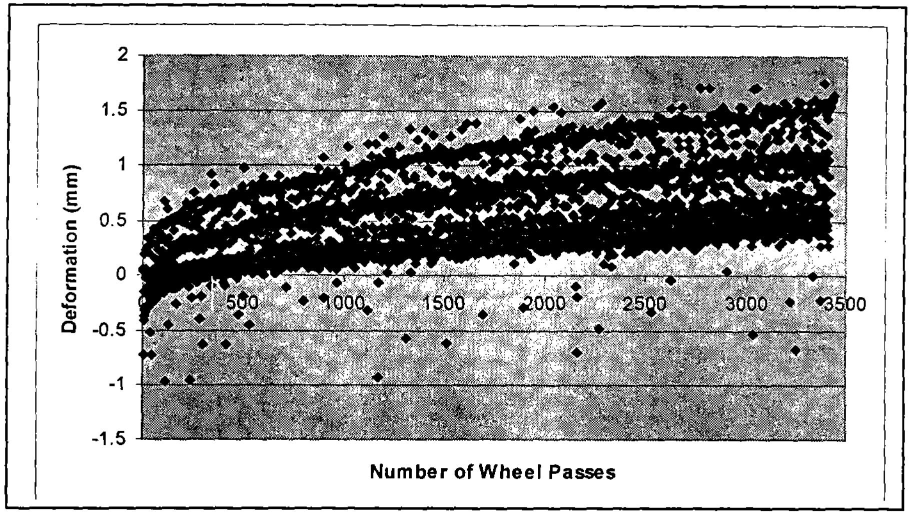


Figure B6 Rut Depth Distribution Along Tracked Slab Section (10D1-60/70)

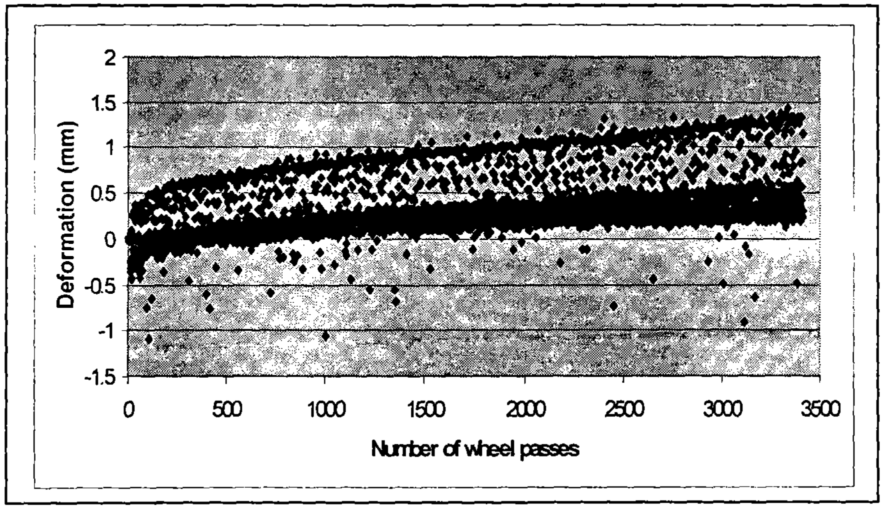


Figure B7 Rut Depth Distribution Along Tracked Slab Section (10D2-60/70)

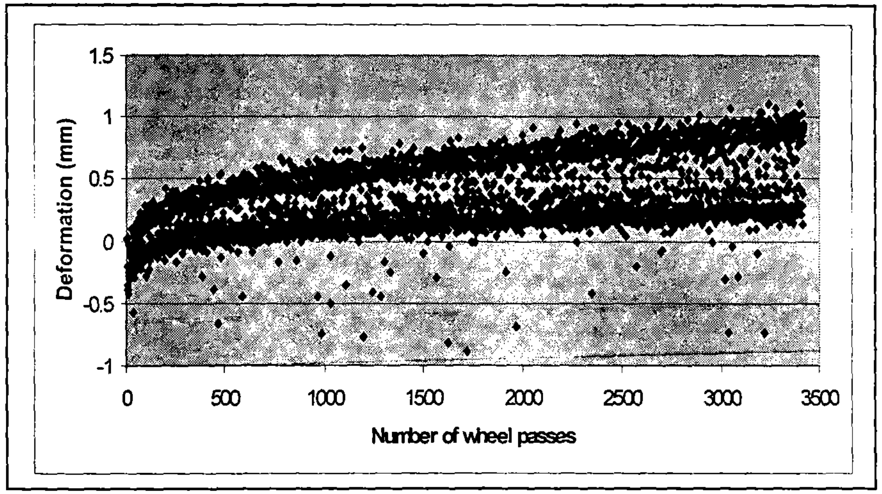
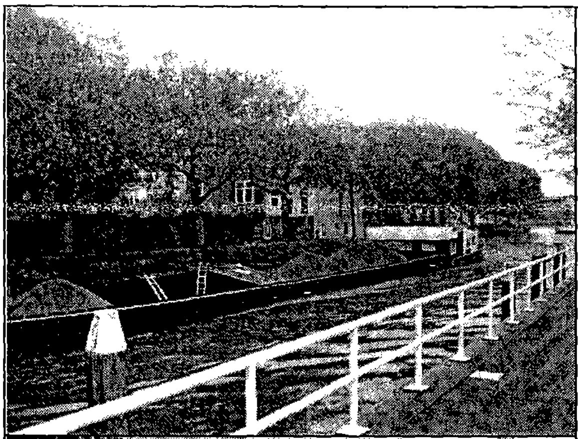
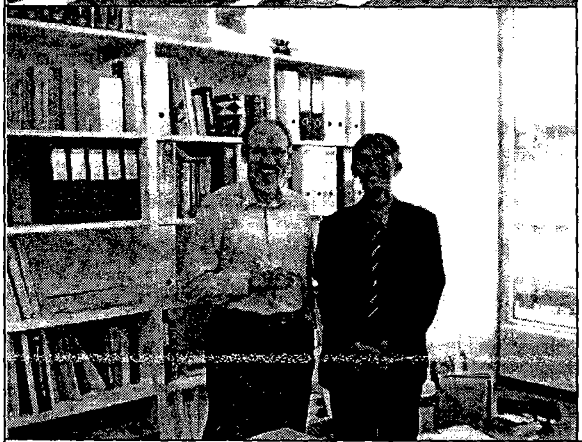
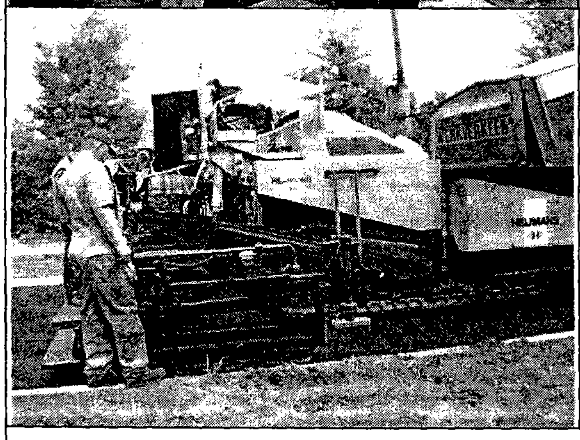


Figure B8 Rut Depth Distribution Along Tracked Slab Section (10D3-60/70)







POROUS ASPHALT PRACTICES IN THE NETHERLANDS

RESEARCH ATTACHMENT REPORT

June 2005

UNIVERSITI SAINS MALAYSIA

PREPARED BY :

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SCHOOL OF CIVIL ENGINEERING
UNIVERSITI SAINS MALAYSIA

POROUS ASPHALT PRACTICES IN THE NETHERLANDS

(RESEARCH ATTACHMENT REPORT)

June 2005

Prepared By:

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NOTICE

The contents of this report reflect the views of the author who is responsible for the data, facts and its accuracy. They do not necessarily reflect the official views of the Section of Road and Railway Engineering, TU Delft or the DWW or any individuals and organizations met while in the Netherlands.

Publication of this document is not possible without the financial support from the Malaysian Ministry of Science, Technology and Innovation.

This report is not meant to be a standard, guideline or specification but can be a useful document in the efforts by the Malaysian government to enhance porous asphalt quality and durability on Malaysian roads.

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Mr. Jaap Molenaar from the DWW
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Mr. Luc Goubert from the Belgium Road Research Centre

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Assoc. Prof. Dr. Meor Othman Hamzah

TABLE OF CONTENTS

POROUS ASPHALT PRACTICES IN THE NETHERLANDS

1.0 PREAMBLE

This report summarises a meaningful five week research attachment that the author undertook at the Section of Road and Railway Engineering, Faculty of Civil and Geosciences, Delft Technical University commencing 15th April 2005 under the supervision of Professor A.A.A. Molenaar and Associate Professor Martin van de Ven. During that period, the author reviewed and documented the historical development, research endeavour, innovative policies, programs and techniques that enhance the durability of single and double layer porous asphalt (PA) in the Netherlands. The author participated in many formal and informal discussions both at TU Delft and with the industries notably the DWW and Heijmans Infrastructuur BV. The author also presented the findings of porous asphalt research that has been carried out at the Universiti Sains Malaysia. To seek clarification on specifications for airport runways, a visit was made to the Netherlands Airport Consultants at Den Haag. The detailed itinerary through the research attachment at TU Delft is available in Appendix A.

On average, Malaysia records an annual traffic growth rate of 7.4%. This phenomenal traffic growth is accompanied with an alarming high accident rates along the major trunk roads. In the mid-1970's, the fatality index, measured in terms of the number of deaths per 10,000 registered vehicles, exceeded 20 and was very high compared to the fatality rate in developed nations. In 1991, a Cabinet Committee on Road Safety was set up and specifically targeted to reduce the fatality rate to 3.14 by the turn of the century, setting 1989 as the base year. Among the measures proposed and adopted to reduce accidents include finding alternative pavement materials to provide high skid resistant surfacings including PA. The year 1991 also witnessed the first application of PA trial in Malaysia. In 1995, it was laid on the Federal Highway. The PA section along the Federal Highway carries some of the heaviest traffic and been resurfaced with a new PA few years ago. In 1996 there were 16 locations along Federal Routes that had been resurfaced with PA and another 25 locations in 1997 that had been identified for implementation. Along the North-South Highway, PA was applied in short stretches of about 200 to 300 meters at accident prone areas such as on super-elevated horizontal curves. Recently, noise starts to become an issue, particularly expressways traversing near residential areas in urban areas notably the capital city of Kuala Lumpur. This explains the interest on quiet pavement such as the double layer porous asphalt (DLPA). The DLPA is a Dutch invention and the technology is relatively new.

This research attachment is sponsored by the Ministry of Science, Technology and Innovation (MOSTI) in an effort to enhance porous asphalt quality and durability on Malaysian roads through a better understanding of the material's behaviour. Over the years, the Universiti Sains Malaysia has undertaken a number of research projects in PA and is the centre of excellence in the area. Ever since the first PA was tried way back in 1991 along the Cheras-Beranang Road, the material has not been applied on a large scale owing to their poor performance and short service life. Judging from the relatively long service life of PA in the Netherlands, there are plenty of lessons to be learned from the durable Dutch PA that would significantly benefit the Malaysian road users. It is hoped that the observations, findings and recommendations contained herein this report will be follow-up with further research and pilot or demonstration projects to adapt the Dutch technology in the Malaysian context.

2.0 THE NETHERLANDS

The Netherlands (lowlands) or unofficially referred to as Holland is a relatively small country with a population of just over 16 million. It is land of the tulips and the wind mills. The windmills were invented in the olden days for drainage and land reclamation. The Netherlands is the largest exporter of cheese in the world and is home to the world renowned Edam cheese. The country is flat with a highest point only 321 m and covers an area of some 41.5 km^2, a quarter of this lies below sea level including the busy Schiphol Airport. The dykes play the crucial role of preventing incursion of water into the lowlands or polders. Amsterdam is the capital city while the political capital is Den Haag. In terms of language, Dutch is widely spoken in the middle and southern Netherlands while the north speaks a different language. Nevertheless, English is widely spoken and understood. Almost every man on the street can communicate in English. The 30^{th} of April is the Queen's Birthday and is celebrated with street party that extends well into the night while the 5^{th} of May is Liberation Day. In the year 2005, the 5^{th} of May coincides with the British general elections.

The Netherlands has an extensive infrastructure comprising over 116,000 km of roads and more than 6,500 km length of railway tracks with over 3000 level crossings. In Amsterdam, the canals are very popular to the thriving tourist industry. The housing shortage in Amsterdam also leads to a number of boat houses along the canals. Some canals are used for transportation of goods. Public transport is excellent. Railways and trams are widely used for intercity and urban travel respectively. Cycling is a very common mode of transport probably due to the flat topography. It is interesting to note that the city centres and its peripheries are designed to promote cycling. Cycle routes and their parking bays are widely available. Special cycle lanes, pavement markings and road signs have been developed to denote areas destined for cycling. Most cycle lanes surfacings are coloured red. Cycling is promoted under all weather conditions and is good for the health and the environment.

3.0 DELFT UNIVERSITY OF TECHNOLOGY

The Delft University of Technology is better known as TU Delft. It is a technical university situated in the south eastern part of Delft. Delft in itself is a small city but with a rich historical past. This is reflected in the ancient canals and the old churches. Delft is situated in between Den Haag and Rotterdam, the world's biggest seaport. Internationally, Delft is accessible via Amsterdam Schiphol Airport (change at Leiden Centraal) or international train network serving rail companies such as the Thalys, Eurostar or TGV. Dutch international train service operates every hour between Amsterdam Centraal and Brussels in Belgium. A ferry service operates between Hoek van Holland and Harwich in England. About 10% of Delft's 0.1 million inhabitants constitute students of TU Delft. Delft is famous for its blue pottery.

The Section of Road and Railway Engineering is under the Faculty of Civil Engineering and Geosciences. The three-year Civil Engineering programme of the faculty culminates in a bachelor of science degree. Students can then choose to continue for another two-year master program in specialised areas of Civil Engineering such as Building Engineering, Structural Engineering, Transportation and Planning, Hydraulic and Geotechnical Engineering, Water Management and Offshore Technology. The Section of Road and Railway Engineering is led by Professor A.A.A. Molenaar, a renowned international expert in asphalt technology. The Road Engineering Division has three other academic staffs including Associate Professor Dr. Martin Van de Ven. A number of PhD and MSc students, local and overseas, are currently in pursuit of their respective research degrees. The highway engineering laboratory of TU Delft has some of the most advanced testing facilities in the world. A picture gallery of laboratory facilities at the Section of Road and Railway Engineering, TU Delft is shown in Appendix B.

4.0 POROUS ASPHALT IN THE NETHERLANDS

4.1 Introduction

In the Netherlands, roads are under the jurisdiction of the Ministry of Transport, Public Works and Water Management. The Institute of Road and Hydraulic Engineering (in Dutch known as DWW) is a division of the Directorate General of Public Works and Water Management (Rijkswaterstaat) of the Ministry providing consultancy services for various technological and environmental aspects of road construction and environmental aspects of road construction and hydraulic engineering. It also conducts research and provides advisory service on specific expertise including roads and highways and environmental related issues. The DWW has 300 permanent staff and is sited in the neighbourhood of TU Delft.

Once in two years, the DWW conducted routine measurements on the entire expressway network with the Automatic Road Analyser (ARAN), skid resistance tester, noise measurement device (ROEMER) and air drainometer. The following data is secured:

- Rut depth
- Road profile or evenness
- Texture depth
- Ravelling
- Skid resistance
- Noise reduction
- Permeability
- Slope variance
- Cracks (video image)

The data enables the DWW to assess the existing pavement condition and a useful input to a pavement maintenance system. The DWW is also well-equipped with a laboratory housing the state-of-the-art equipments for binder analysis, mix design and performance evaluation. This shall be explained in later sections.

4.2 Single Layer PA

In the Netherlands, PA is described as ZOAB. A comprehensive presentation and discussions on ZOAB were conducted at the DWW in the presence of Mr. Jan Voskuilen and Ir. J.B.M. van Wieringen. In the Netherlands, PA was initiated during the seventies on grounds of traffic safety. Nevertheless, since 1990, and further to European and national legislations on noise, PA was applied to reduce traffic noise. At the time of writing, about 65% of Dutch highways are paved with PA.

4.3 Mix Design

In the early days, PA mix design in the Netherlands was empirical in nature and based on field observations. After extensive field trials, the following formulation was adopted for single layer PA:

- Crushed stones 6 to 16 mm
- Crushed sand between 2 and 0.063 mm
- Hydrated lime filler in quantities of 4.5% of overall aggregate or 25% of binder
- Bitumen type 70/100

The aggregate gradation is shown in Table 1. The aggregate gradation is expressed in cumulative percentage retained.

Table 1 Mix Design of Single Layer PA 0/16 (Source: DWW)

Sieve Size (mm)	**Desirable (%)**	**Maximum (%)**	**Minimum (%)**
16.0		0.0	7.0
11.2		15.0	30.0
8.0		50.0	65.0
5.6		70.0	85.0
2.0	85.5		
0.063	95.5		
Bitumen 70/100	4.5% (based on 100% aggregate)		

Coarse aggregates in the Netherlands are practically non-existent. Most, if not all coarse aggregates are imported from as far away as Norway and Ireland. Initially crushed river gravels were used but it is now no longer recommended. Instead, crushed coarse quarry aggregates are accepted in order to maintain good skid resistance in the long term. However, gravels are used extensively used in concreting jobs. No modified binder is used in PA. PA incorporating modified binders in 5.5% quantity is only used on curves and other similar highly-stressed areas. Despite the use of only conventional bitumen, no drainage inhibitors such as fibers are used. Use of reclaimed asphalt pavement (RAP) is prohibited for the PA surfacing. It is interesting to note that no mechanical tests are carried out. Not withstanding these practices, PA in the Netherlands exhibits an average service life of 12 and 16 years at the slow and fast lanes respectively. Hydrated lime in quantity of 4.5% is used as filler.

The mix design procedure is very simple. According to the procedure, four Marshall specimens are prepared by applying 50 blows per face. Then, the void content is determined from specimen geometry. The theoretical maximum density is determined by a procedure similar to the Rice Method. The required minimum void content is 20%. No mechanical tests are carried out on the Marshall specimens. The specified binder content is 4.5%.

Construction wise, single layer PA wearing courses are laid in 50 mm thickness. They are laid at temperatures ranging from 130 to 170°C in one layer. The binder content variation is tolerated at $\pm$0.5% from the specified value while void content can range between 15 to 25%. A minimum relative compaction of 97% is also specified. The Marshall mode of compaction at 50 blow per face is the reference density. No roller vibration is to take place in the compaction process except at the transverse jointing with an existing surface. Pneumatic tyre is also not used.

4.4 Performance

In many countries, including the Netherlands, PA is laid to reduce traffic noise at source. Porous surfacings eliminate the need for noise barriers which are not only expensive but aesthetically unpleasing, difficult to maintain and prone to graffiti. PA surfacings can attain a noise reduction by an average of 3 dB(A). In layman terms, a 3 dB(A) reduction is equivalent to doubling the distance between the noise source and the receiver or reducing the traffic volume by 50% or reducing traffic speed by 25%.

In the Netherlands, clogging appears less problematic. The porous sections were generally placed on high speed roads to take advantage of the self-cleansing action of fast moving vehicular tyres. However, clogging is quite severe at the road shoulder. To mitigate clogging problem, PAs are cleansed twice a year using a specially built hydrovac machine. The principle of cleansing is similar to that of a household wet vacuum cleaner. Jets of water are sprayed ahead to loosen the clogging agents and then sucked up behind. The DWW has developed a special equipment named air-drainometer, shown in Figure 1, to measure the degree of clogging in the field. Basically, the machine has an air-tight assembly that can maintain a constant air flow at 1.5 m^3/min through an asphalt surfacing. The compressed air is blown through a measured section on the road surfacing and the consequent air pressure build up is dependent on the degree of clogging as categorised in Table 2. Based on measurements made on Dutch PA surfacings, it was concluded that clogging is not a problem on the traffic lanes. As mentioned earlier, fast moving vehicles provide the self-cleansing effect. However, clogging is severe on the road shoulders and hence need to be cleaned once or twice a year. Despite the usefulness of the air-drainometer, measurements have to be taken in a static manner and is therefore time consuming.

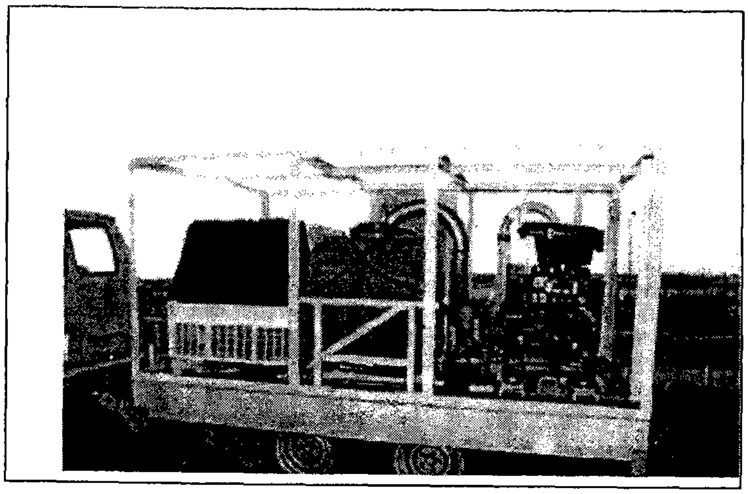


Figure 1 Air-Drainometer Developed by the DWW (Source: DWW)

Table 2 Classification for Degree of Clogging (Source: DWW)

Degree of Clogging	Air Pressure (mbar)	Void Content
Clean open pores	30 - 80	16 – 23
Slightly blocked pores	180 - 300	13 – 16
Moderate blocked pores	300 - 700	11 – 13
Almost blocked pores	> 700	< 11

In 1991, a fatal accident took place on a newly opened PA surfaced road. An in-depth study was conducted to determine the cause of the tragedy. And it was attributed to a phenomenon described as 'bitu-planing'. This phenomenon is associated with initial dry skid resistance that commonly takes place on newly surfaced PA. In contrast with dense surfacings, no crushed sand is sprinkled to improve skid resistance of PA surfacings. After the PA is laid, the stones are coated with a thin bitumen film. The texture of PA surfacings comprise of holes and hence contact pressure at the tyre-pavement interface is higher. When emergency brakes are applied, the tyre slips on the surface generating enough heat due to the high contact stresses to melt the bitumen. The melted bitumen forms a slippery surface that facilitates skidding. Ironically, this problem occurs only with vehicles without ABS system. The DWW conducted dry skid resistance tests on different road surfaces. The method measured the braking deceleration of an automobile, see Figure 2, on dry pavement surface where 100% slip was attained and the results are shown in Table 3. The results affirm the low values of dry skid resistance on PA surfacings. It was thought that sprinkling sand would mitigate the problem. However, when tested on a large scale, no positive effect was found. The resultant braking deceleration indicates the braking distance required. Hence from 1991, DWW adopted the following policy:

- Putting up special warning sign '*longer braking distance – new road surface*' plus the length of road section ahead that is affected. This sign can be removed when the braking deceleration reaches 6.5 m/s^2. An example is shown in Figure 3.
- If the braking deceleration is less than 5.2 m/s^2, a 70 km/h speed limit is imposed.

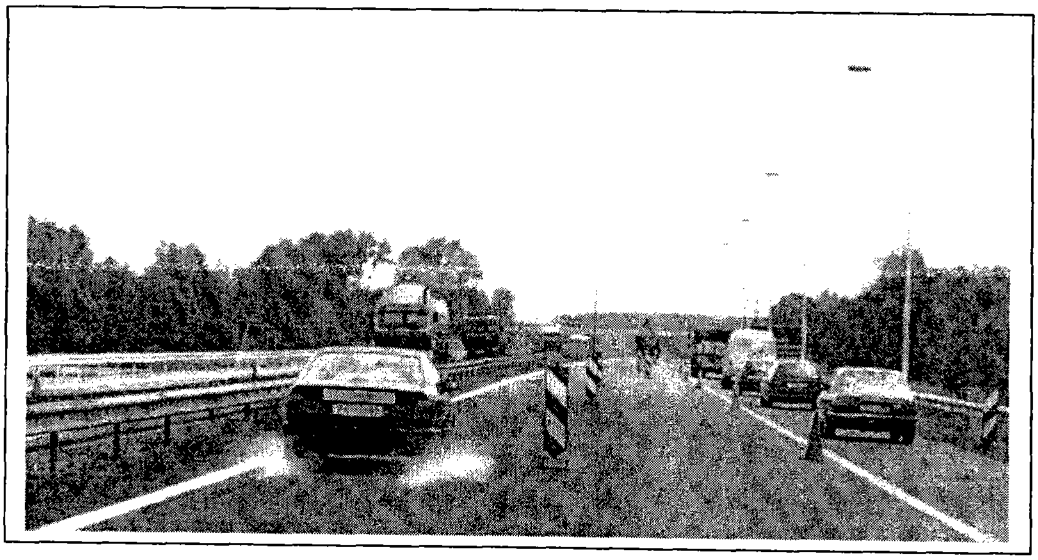


Figure 2 Braking Test On a Newly Laid PA (Source: DWW)

Table 3 Braking Deceleration Values (m/s^2) on New Dense and PA Surfacings (Source: DWW)

Brake Characteristics	Porous Asphalt		Dense Asphalt	
	New	Old	New	Old
Without ABS	5.4	7.0	7.0	8.0
With ABS	9.0 – 9.5		9.5 – 10.0	

Figure 3 An Example of a Warning Sign on Newly Paved PA Surfacing (Source: DWW)

Drivers do not expect the initial dry skid resistance problem out of a new road surfacing. The results of Table 3 affirm that the problem is confined to cars without ABS. Most new cars are equipped with ABS system and hence in future, this may be not an issue. The manner in which the braking test was conducted is not safe. Several other methods were explored. The locked wheel testing at 80 km/h is the most promising. However, correlations need to be established between the braking test and locked wheel tests for all surface types.

The DWW also discovered that the rate of accidents on wet PA sections is similar to that on dense surfacings. Observations indicate the tendency for drivers to drive faster during rain on PAC and consequently the extra safety offered by the porous surfacing disappear. However, PA helps to improve road capacity during wet weather due to higher running speeds.

In the Netherlands, PA is found to be a very rut resistant material. This finding is consistent with laboratory wheel tracking and creep tests reported in the literature worldwide. The material is favourably accepted by the road users. However, its service life is limited by the problem of ravelling. At the end of its service life, the old PA is milled and a new porous mix is laid. The recycled PA is used for the underlying layer.

4.5 Research To Extend PA Service Life

Despite the impressive service life of PA in the Netherlands, a major research was undertaken to further prolong the design life and to develop test method that can be used to study ravelling resistance. Ravelling, or loss of aggregate particles, is the most dominant mode of failure in the Netherlands. The research attempted to address two issues namely to improve resistance to ravelling and developing test methods to predict ravelling resistance.

Three different locations were selected comprising of 26 test sections with 'improved' PA mixtures as summarised in Table 4. At each test section, the standard PA was also laid as a control mix. The bitumen content of the standard mix is 4.5%.

Table 4 PA Test Sections and Mix Characteristics (Source: DWW)

Location	Mix Design
RW 12 Nootdorp	Standard PA with 4.5% bitumen 70/100 with crushed river gravel
	Standard PA with 4.5% bitumen 70/100 with porphyry
RW 10 Amsterdam	2 PA mixtures with 4.5% and 5.5% rubberised bitumen
	2 PA mixtures with 5.5% bitumen and drainage inhibitors
	2 PA mixtures with 4.5% and 5.5% SBS PMB
	2 PA mixtures with 4.5% and 5.5% EVA PMB
	1 PA mix with 5.5% bitumen 70/100 and modified grading
RW 12 Zoetermeer	PA mixture with 5.5% bitumen 70/100 and addition of naturally occurring hydrocarbon (Gilsonite)
	PA with 5.5% Multigrade bitumen
	PA mixture with 5.7% bitumen and drainage inhibitor
	Standard PA with 4.5% bitumen 70/100

It was initially thought that ravelling can be minimised if the binder film was thicker. This was evident from the literature on the Cantabro test developed by Spanish researchers. Thicker binder film demands an increase in binder content. A mix with a thicker binder film is expected to exhibit higher tensile strength, higher resistance to ravelling and age less rapidly. However, an increase in binder content in PA may be accompanied by binder drainage. In the field trial, the problem was mitigated by using modified binder including the multigrade bitumen, modifying the aggregate gradation or using drainage inhibitors. Another 'improved' PA mix utilised quarry aggregates instead of crushed river gravels.

Upon laying of the test sections, cores were extracted and tested from the test sections every 1, 2, 3 and 9 years. Every year, visual inspections were carried out on all test sections. The bitumen composition and properties were also tested. The cores were also subjected to the following tests:

- Cyclic direct tensile test at 20°C
- Indirect tensile test at 0 and 30°C
- Cantabro test at 18°C

The results of the visual inspection of the field trial were summarised in terms of their service life shown in Table 5.

Table 5 Road Category in Relation to Service Life (Source: DWW)

Category	Length of Service Life
Good	About 13 years
Moderate	From 11 to 12 years
Bad	About 10 years

In general, PA mixtures with 5.5% bitumen were 'good' while PA mixtures with 4.5% bitumen performed 'bad'. Some PA mixtures with 4.5% and 5.5% bitumen performed 'moderate'. The performance of PA with porphory was much better than standard PA with crushed river gravel. However, the skid resistance of PA with porphory was worse.

The results of the mechanical testing indicated that the ranking of the results per test method was completely different. The cyclic direct tensile test was the most discriminative for the PMB investigated. The ranking of the mixes based on the test methods did not agree with the ranking made by visual inspection.

The bitumen composition and properties results also produce interesting results. The penetration test indicated that a thicker (5.5%) bitumen film and modification did not improve the resistance to ageing compared with bitumen 70/100 in standard PA (4.5%). The PA mixtures with drainage inhibitors did not experience drainage. After 9 years, the penetration value of all recovered bitumen, including modified bitumens, approach 20.

In general, the results of the field trial indicate that there is no obvious benefit in using PMB. Standard PA with 4.5% bitumen 70/100 can perform equally well with modified PA with equivalent bitumen content. However, PA mixes with 5.5% binder generally acts better. The positive effect of using porphory to resist ravelling is offset by poor long term skid resistance. Hence it is recommended to adopt PA mixes with 5.5% bitumen 70/100. To eliminate binder rundown, drainage inhibitors are used.

Another product of the research program is the rotating surface abrasion test (RSAT) equipment which was developed a new test procedure to assess ravelling in the laboratory. The test was in principle similar to the mini fretting test developed by Shell to test adhesion of aggregates in surface dressing. In the RSAT, slabs at 20°C are rotated and abraded by a rubber tyred wheel that abrade the surface of the PA. The tyre wear and loose aggregates were both vacuumed and their masses weighed. The resistance to ravelling is expressed as the total amount of loose material after 24 hours testing.

5.0 DOUBLE LAYER POROUS ASPHALT

5.1 Introduction

Ir. J.B.M. van Wieringen presented a scenario of DLPA in the Netherlands. It was said that PA became popular in the Netherlands because of noise reduction. However, from the year 1990, PA was applied to reduce traffic noise. The Noise Innovation Programme (IPG), with a budget of 50 million Euros was formed in 2002 mainly to investigate possible traffic related noise reduction on Dutch roads.

5.2 The Innovatieprogramma Geluid (IPG)

In 1970, Europe started to legislate limitations on noise emission from road vehicles. The European Union Green Paper of 1996 had aimed that 'no person should be exposed to noise levels which endanger health and quality of life'. The European Commission vision on noise policy up to year 2020 aims 'to avoid harmful effects of noise exposure from all sources and to preserve quiet areas'.

The IPG program was formed to seek innovative means to reduce traffic related noise by improvements to road surfaces, tyres and vehicles and enhanced noise barriers. The IPG was initiated in response to strict Dutch noise legislation rather than a result of EU policy. The expected goals of the IPG programme for demonstrated products but further work needed prior to implementation up to year 2007 is shown in Table 6. The reference surfaces is shown in Table 7.

Table 6 Expected Outcomes of IPG (2003 – 2007) (Source: IPG)

Source	Noise Reduction
Road Surfaces	6 dB(A)
Tyres and Vehicles	3 dB(A)
Noise Barriers	3 dB(A)
Total Noise Reduction	**12 dB(A)**

Table 7 Reference Table for Road Surfaces Showing IPG Goals in Noise Reduction (Source: IPG)

Type of Surfacing	Noise Reduction
Two Layer PA (2005)	4 dB(A)
Two Layer PA (2007)	4 to 6 dB(A)
Thin semi-dense surfaces	3 dB(A)
3rd Generation Surfaces	6 to 8 dB(A)

To achieve these goals, five research clusters as shown in Table 8 were formed. Various project groups were created under each research cluster. A Scientific Board was also formed to serve as an auditing and advisory panel. The Board comprises of experts from national road research institutes throughout Europe. Documents were prepared that defined strategic goals for as far ahead as 2030. Research at IPG was directed towards reducing noise at source by developing silent roads or quiet pavements. One potential material is double layer porous asphalt (DLPA).

Table 8 Research Clusters and Project Groups Within IPG (Source: IPG)

CLUSTER	**Knowledge Management**	**Silent Roads**	**Enhanced Noise Barrier Efficiency**	**Assessment Methods**	**Silent Vehicles and Tyres**
PROJECT GROUPS	Knowledge infrastructure	Wide application of two-layer PA	Active noise barriers	Standardised measurement methods	International legislation
	Optimising the acoustic performance of road surfaces	Improvement of porous surfaces	Barrier top efficiency	Legislative and procedural framework	National stimulation
		Improvement of non-porous surfaces	Barrier position optimisation	Cost-effectiveness	
		Third generation silent roads			
		Thin layer surfaces			

The DLPA was the brainchild of Mr. Gerbert van Bochove, the Innovation Manager of the RND Department of a contracting company Heijmans Infrastructuur BV. The DLPA consists of a thin layer of fine PA overlying a coarser and thicker PA as graphically illustrated in Figure 4. In this type of construction, the bottom layer is more permeable compared to conventional PA through which the sideways discharge of water improves considerably. This is essential to permit faster drainage where runoff accumulates. The fine top layer traps coarse dirt from entering into the underlying layers. The trapped dirt can be easily removed by cleansing techniques using spraying and vacuum suction. Double layer PA will not clog up rapidly as single layer PA but even if the material is clogged, the double layer PA is relatively easier to clean. The surface texture of the top layer is said to be finer and offers acoustic advantages.

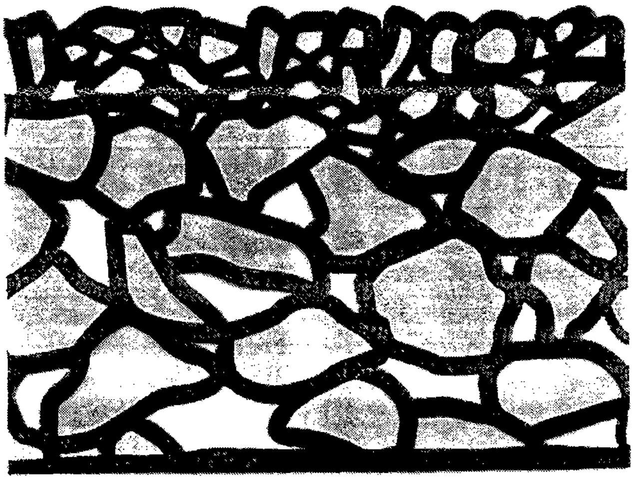


Figure 4 A Schematic Diagram of the Double Layer Porous Asphalt (Source: Van Bochove, 1996)

The noise levels emanating from various road surfacing types in the Netherlands is shown in Figure 5.

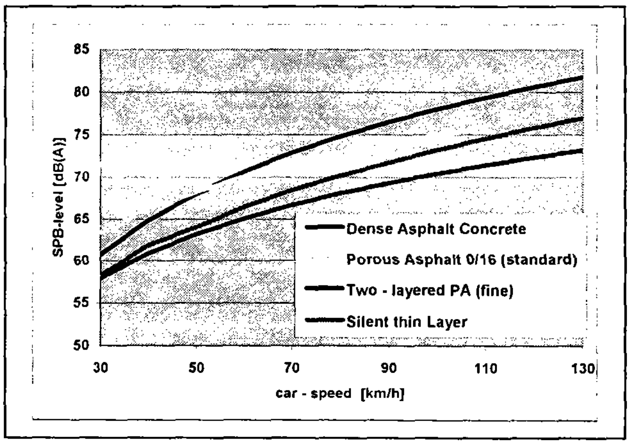


Figure 5 Noise Levels Generated from Dutch Roads (Spurce: Van Bochove)

The graphs indicate that at speeds less than about 50 km/h, the single layer PA can be noisier than normal asphalt. Hence, it is not appropriate to lay single layer PA on urban roads. However, the DLPA is consistently less noisy than dense surfacings regardless of the speed. Hence, DLPA is one of the paving materials selected for research by the IPG to achieve significant noise reduction at source. For the DLPA, the IPG set the following goals:

Although DLPA has been used in the Netherlands for over six years, the material is not widely applied due to poor performance and acoustic life time. Three areas of improvement that IPG focus on are outlined underneath:

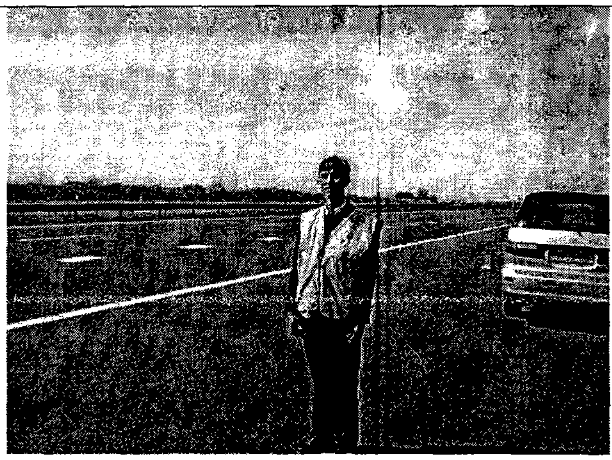
1. To extend the structural life time while retaining current noise performance. An acceptable technical service life is between 8 to 10 years.

2. To improve the acoustical performance while retaining the current structural performance. Use of existing best DLPA mixes and construction method should result in higher initial noise reduction between 5 to 6 dB(A) compared with dense surfacings but provide a life time average noise reduction of 4 dB(A).
3. To reduce the range of climatic restrictions under which the surfaces cannot be laid.

The Dutch DLPA consists of a 25 mm fine-graded 4/8 top layer and a 45 mm coarse-graded 11/16 top layer. An alternative construction is a 20 mm 2/6 top layer and 50 mm 11/16 bottom layer.

5.3 Site Visits and Discussion with the Inventor of DLPA

Site visit to a highway was conducted together with Mr Gerbert Van Bochove of Heijmans Infrastructuur BV and who is the inventor of the double layer PA. The site, shown in Figure 6, was less than half an hour's drive away from TU Delft. All along the highway, PA was extensively used as the surfacing material. However, the section length made of double layer PA was about 4 kilometers and at the time of visit was already 8 years old. The binder used was crumb rubber modified bitumen and was the last of its kind ever constructed in the Netherlands. Now, use of crumb rubber modified bitumen is not permitted on health grounds. Mr. Van Bochove pointed out the effect of the weather on the pavement surfacing quality. The section that was constructed during bad weather with drizzle exhibit many surfacing problems particularly ravelling. The fundamental principle of 'NO PAVING IN THE RAIN' must be strictly adhered to in the construction of PA.

	
Figure 6(a) Edge Details of the DLPA	Figure 6(b) Highway Section Paved with Rubberised DLPA

DLPA can be compacted via either one of the following methods:

- Hot-on-hot
- Hot-on-cold

Quite surprisingly both methods have been simulated by the author in the USM laboratory corresponding to Methods A and B respectively. A research on the differences of filed compaction methods has been undertaken by a postgraduate student at TU Delft in an MSc thesis entitled '*Effecten Gradering op de Functionele Eigenschappen Tweelaags Zoab*'. The thesis was publicly lectured by Ton van der Steen at TU Delft. The lecture was well attended by audience drawn from the industries including road contractors. The researcher discovered that Method B produced specimens that are less permeable compared to Method A. A constant head permeameter was fabricated to measure the coefficient of permeability. It was suggested that the transition zone was crucial in determining the coefficient of permeability values of the DLPA.

Construction of DLPA was found to be weather sensitive. This was particularly true for the hot-on-cold construction method when the second ultra-thin layer cools down very rapidly. In this respect, the hot-on-hot was favoured but modifications need to be made on current road paving machines.

The aggregate gradation chart of a porous asphalt is characterised by the presence of a breakpoint or knee. There is a tendency in the Netherlands to move the breakpoint further to the right or increase the sieve size at the occurrence of the breakpoint. The aggregate gradation for DLPA used by Heijmans is shown in Table 9.

Table 9 Aggregate Gradation for Double Layer PA (Source: Van Bochove)

Sieve Size (mm)	**Twinlay toplayer (4/8)**	**Twinlay M toplayer (2/6)**	**Twinlay underlayer (11/16)**
16.0			7.1
11.2	0		80.5
8.0	6.2	0	87.2
5.6	57.4	2.6	87.6
4.0	85	30.8	
2.0	87.2	80.5	88.7
0.5	88.0	86.5	93.2
0.18	90.3	88.8	94.7
0.063	93.4	92	95.7
< 0.063	6.6	8.0	4.3
SBS Modified Binder	6.0	6.5	4.2

Ravelling is one of the problems faced by PA. When ravelling becomes severe, potholes may develop. Patching of potholes can be accomplished using dense mix. The relative small area of the patch in relation to the pavement area does not disrupt continuity of water flow.

Ravelling or particle loss is similar but not identical to abrasion resistance or resistance to disintegration. Test procedure to assess the former was developed by the Spanish by subjecting a Marshall specimen to 300 rotations in the Los Angeles drum without balls. To assess resistance to ravelling of porous asphalt slabs, Mr. van Bochove invented the Rolling Surface Abrasion Test (RSAT) as shown in Figure 7. The principle of the test is similar to the mini fretting test developed by Shell to test adhesion of chippings in a surface dressing. The test realistically simulates the abrasive action of vehicle tyres on the DLPA surfacings. Resistance to ravelling is quantified in terms of the percentage loss in mass.

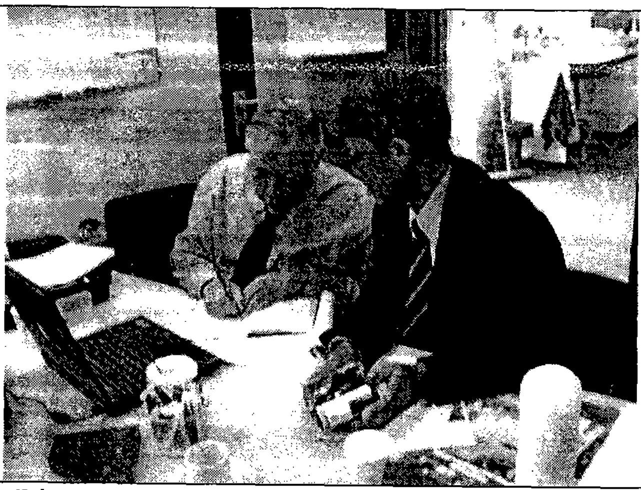
	
Discussion with Mr. Van Bochove at Heijmans Headquarters	At the Rotating Surface Abrasion Tester

Figure 7 Discussion with the Inventor of the RSAT

6.0 THIN SURFACINGS

An alternative to double layer PA to mitigate noise is thin surfacing with negative texture and marketed by Heijmans under the tradename Microflex. Mr. van Bochove categorised asphalt materials based on their voids as shown in Table 10. In this regards, Microflex is regarded as a semi-dense material. The cross section of the material is shown in Figure 8.

Table 10 Asphalt Category in Relation to Porosity (Source: Van Bochove)

Mix Category	Porosity (%)
Dense	4 – 9
Semi-dense	9 – 14
Half porous	14 – 19
Porous	> 19

Figure 8 The Structure of Microflex

Field experience indicates that this material is more durable compared to PA. As shown in Figure 5, Microflex is less noisy compared to conventional asphalt at all speeds. Between 2 to 0.5 mm, a gap is created in the gradation. Hydrated lime is used as the filler while bitumen content is about 6.5% but incorporating modifier. The target porosity ranges between 10 to 13% measured by specimen geometry. The rotating surface abrasion test is used to assess resistance to ravelling. The voids can be easily clogged and if this happens, the noise reduction properties reduce. However, the material still exhibits normal performance. Noise restoration can be made by cleansing in the normal way.

Construction of Microflex was witnessed at a site near Eindhoven. Many aspects of construction were taught by Mr. van Bochove at the site which includes choice of roller, rolling pattern, material transfer from lorry to hopper, temperatures, paving operations and joint construction. Figure 9 show pictures of thin surfacing construction.

Figure 9 Thin Surfacing Construction Near Eindhoven

7.0 SEMI-FLEXIBLE PAVEMENT

Another new invention by van Bochove of Heijmans is a road material that is meant as a hybrid between flexible and rigid pavement, tradenamed stabiflex. The material, modelled in Figure 10, comprises of an initial porous asphalt, similar to the bottom layer of a DLPA. In the next step, the pores are slowly filled up with slurry. The slurry must be sufficiently fluidic to fill up all pores without the need for vibration. To achieve high skid resistance, sand is applied on the surfacing. Otherwise, the slurry is poured to a level that allows the aggregate to protrude or stand proud. Colourings can be added as well for instance to demarcate bicycle lanes. In the Netherlands, bicycle lanes are coloured red.

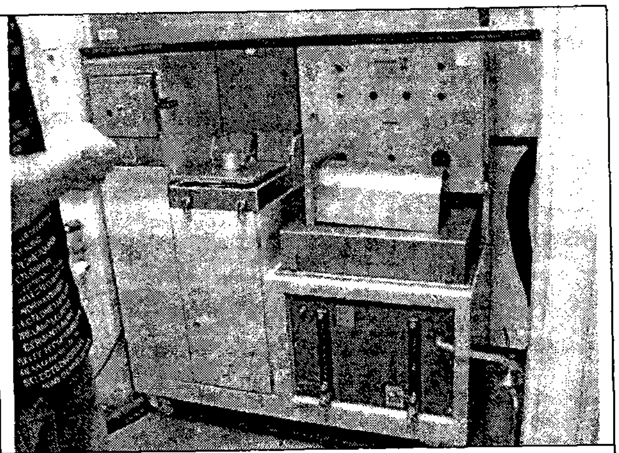
Figure 10 A Model of Stabiflex

The material is suitable for highly stressed areas such as approaches to junctions and roundabouts. Since the material exhibits very high stiffness, they are suitable for pavements subjected to heavy axle loadings such as ports, airport aprons or bus laybys. Another potential use is for pavements that are subjected to oil droppings or spillage.

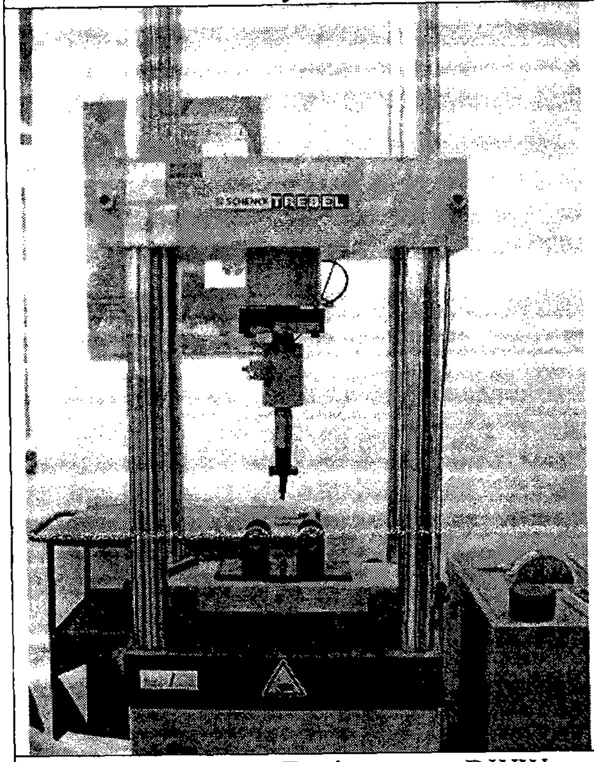
8.0 THE DWW LABORATORY

Visit to DWW laboratory under the supervision of Dave van Vliet and Eyassu was very memorable. DWW is the research arm of the ministry responsible for roads and traffic in the Netherlands. The laboratory is well-equipped with the state-of-the-art equipments particularly for binder analysis. Apart from basic equipments such as the automatic penetrometer and ring and ball, the laboratory also houses advanced equipment that could characterise the fundamental behaviour of both virgin and aged binder. This includes a fluorescent infra-red capable of characterisation of the ageing behaviour of binders. A more interesting equipment is the dynamic accelerated ageing of thin binder film at elevated temperature and with pulses of oxygen in low quantity. This is certainly an improvement over the pressure ageing vessel proposed by SHRP where the binder is ageing in a static mode. The laboratory has excellent glasswares and equipments for binder extraction. Binder extraction is done for the dual purpose of determining the mix binder content while the recovered binder can be subjected to further analysis. For mix performance testing, a triaxial test equipment and apparatus for the semi-circular bending (SCB) test are heavily used. Many equipments at DWWare also available at TU Delft. Pictures of some of the equipments are shown in Figure 11.

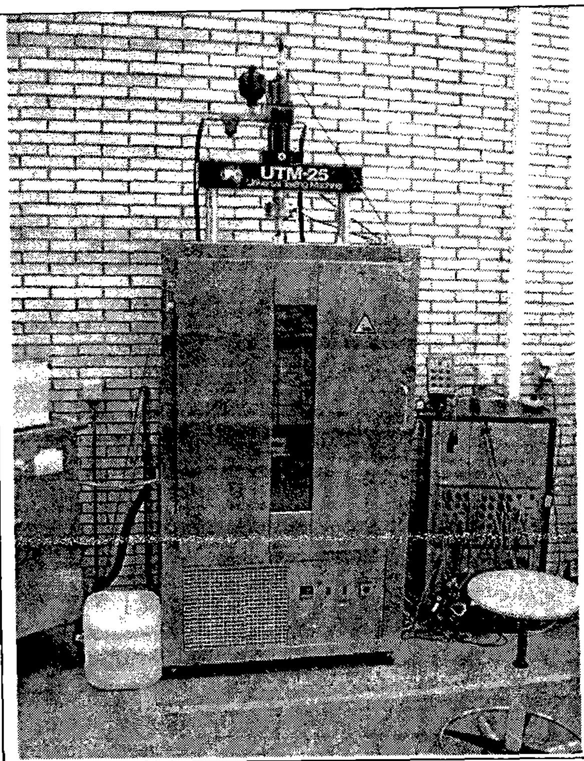
Visiting DWW Laboratory with Dave and Eyassu



Binder Recovery Apparatus at DWW



The SCB Test Equipment at DWW



The IPC Universal Testing Machine at DWW

Figure 11 Visit and Equipments at the DWW

The SCB test is a simple and rapid test to determine the modulus and tensile characteristics of a semi-circular asphalt specimen. The big advantage of the SCB test over the indirect tensile test is the fact that a nice crack develops in the SCB without wedging near the loading strip. The latter phenomenon is quite often observed in the ITT especially when this test is performed at temperatures of 15°C and higher.

9.0 BINDER AGEING STUDIES

A discussion was made with Mr. Erik Jan Scholten From Kraton Polymer. Kraton Polymer used to be part of Shell but is now fully independent. This company is the main supplier of the SBS that is widely used as a bitumen modifier. Discussion centred on future binder modification which should focus on retarding the process of binder ageing. This is particularly suited to countries in the tropics including Malaysia where binder hardening results in the prevalence of alligator top-down cracking. A multi-disciplinary approach towards binder studies should be undertaken to achieve this goal.

10.0 CHARACTERISATION OF POROUS ASPHALT USING A CT SCANNER

In the year 2000, the Department of Earth Science of TU Delft bought a third generation Computerised Tomography (CT) Scanner made by Siemens from a French hospital. This is the CT scanner that has been used world wide to scan and produce cross section images of the human body. Since its acquisition by TU Delft, use of the CT scanner has been extended to analysis of asphalt mixes. A discussion was made with Wim Verwaal at the Department of Geological Sciences in the presence of Dr. Martin.

To produce images or slices of the asphalt specimens, X-rays pass through the specimen along several paths in several different directions. The X-rays map out density differences within the object which corresponds to density differences. Hence it is possible to obtain an image that displays difference in density at each of several thousand points in a two-dimensional slice through an object. In the TI Delft CT scanner, the minimum thickness of each slice is 1 mm though it can be set to obtain a 0.5 mm thick slice. By stacking up a series of continuous two-dimensional images, a three-dimensional mapping of density variations can be obtained. The data set is processed by a software called OSIRIS 4.18. To quantify the data images, the software AMIRA 3.0 is used. A 3-dimensional view of the slice can be made.

The CT scanner assigns a value to each 3-D pixel element, known as the Hounsfield Unit (HU). The value which is assigned to a pixel depends on its chemical elements. In medicine, different HU values would be assigned to for example fat, muscle and bone. However, in an asphalt, the following groups have been established:

- Porosity or voids
- Mortar comprising of bitumen, filler and additives
- Aggregates

The lowest HU values are assigned to the voids, then to mortar and finally to aggregate. Boundaries are set and each pixel is assigned to one of the three groups. Typical examples of scanned images or slice of a double layer PA is shown in Figures 12 and 13. From the assigned HU values, it is therefore possible to determine the volume of voids, mortar and aggregate per slice. A typical example is shown in Figure 14.

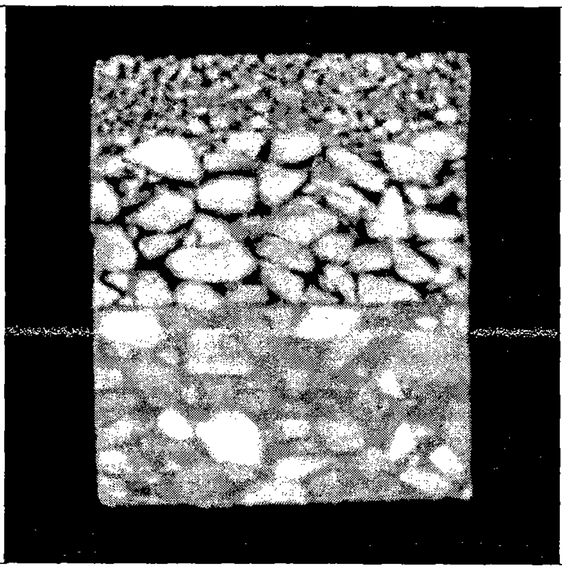


Figure 12 Scanned Vertical Profile of a DLPA Sample (Source: DWW)

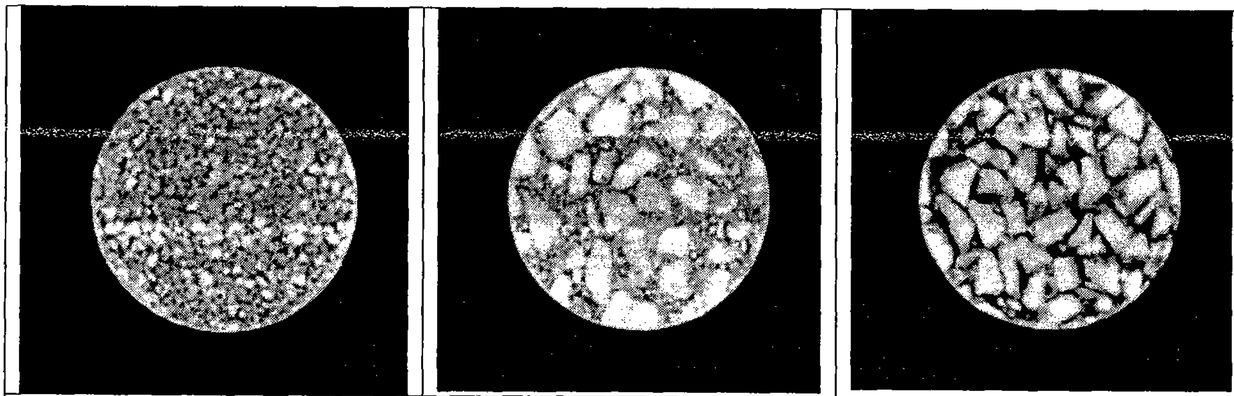


Figure 13 Scanned Top, Transition and Bottom Layers of DLPA (Source: DWW)

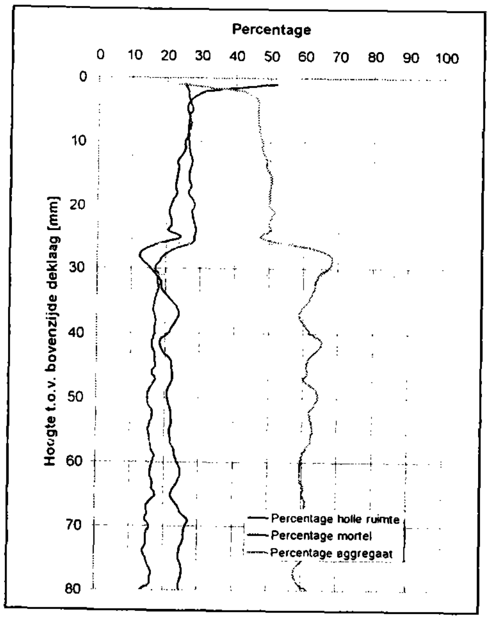


Figure 14 Volume Percentage Per Slice of Voids, Mortar and Aggregate (Source: Ton van der Steen)

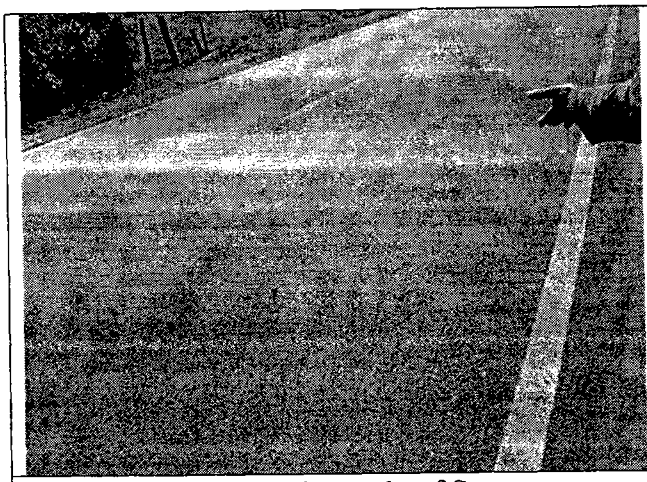
By viewing the image slice by slice, the CT scanner technique provides researchers with a powerful tool to make a more microscopic examination of internal voids distribution and more importantly their inter-connectivity. In the case of the double layer PA, it is possible to evaluate the voids distribution at the transition zone which is at the interface between top and bottom layers. The extent of the transition zone can be determined as well. In addition, if a core from a PA mix that has undergone substantial ravelling is scanned, it is possible to judge the extent or variation of ravelling over the entire depth of the pavement thickness.

Clearly, the CT scanner provides various possibilities of asphalt core analysis. The only limitation is the cost factor. In the future, it is expected that a CT scanner will be developed for the sole purpose of engineering materials analysis such as asphalt and hence the equipment cost is expected to be more affordable.

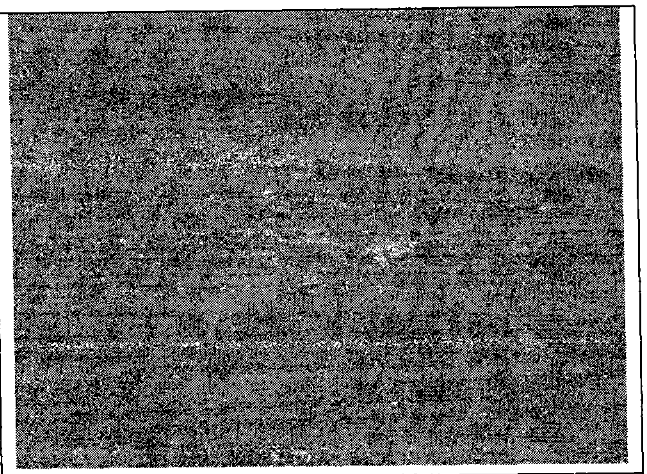
11.0 THE BELGIAN ROAD RESEARCH CENTRE

The history of the formation of the Belgian Road Research Centre (BRRC) dates back after the end of World War II when the Belgian government decided to create a tool to accelerate innovation in industry. In January 1947, the decree-law was issued for the creation of centres aimed at promoting and coordinating technical progress through research. As a consequence, several brabches of industry established their very own research centres. Further to this, the BRRC was founded by royal decree on May 1952 at the request of the national federation of road contractors in agreement with the road authorities and under the protection of the Ministry of Economic Affairs. This explains the peculiar status of the BRRC as a private administered public utility research institute. Under the decree-law, each Belgian or foreign accountable contractor is mandatory to contribute to BRRC a 0.8% assessment on the total project costs carried out in Belgium, irrespective of whether those works have been awarded by open or limited tendering or are performed under private contracts. This to date remains the BRRC's main source of income. Other sources include funding from regional, federal or European authorities for research and through the provision of services such as courses, workshops, testing, analysis and advisory, royalties or sale of publications. After being in existence for more than fifty years, it has build a solid reputation at both national and international levels

The visit to the BRRC head office in Brussels coincided with routine measurements on a double layer PA, the only one constructed in Belgium. The DLPA field trial covering a length of approximately 500 m was located at Bambois, a truly rural two-lane bothways road with a relatively low traffic volume. The DLPA pavement structure comprises of a 25 and 45 mm top and bottom layer respectively and was laid on an existing road. The gradation was selected based on local experience with single layer PA. The first trial lay took place in September 2000. After a year in service, excessive ravelling took place and the entire top layer was milled and replaced in June 2002. In November 2003, one lane was cleaned using two passes of the hydrovac machine travelling at about 5 km/h. Field permeability was measured using a specially developed permeameter while density was measured using a nuclear gauge. The equipment only recorded discharge time as the permeability indicator. Measurements made in May 2004 indicated that the lane that was cleaned recorded lower average discharge time (18 sec compared to 25 sec), indicating high permeability. However, the cleaned lane exhibited more spots, shown in Figure 15, which are the starting points or building blocks for ravelling. Before cleaning, 25 spots were detected but the number increases to 44 immediately after cleaning. Noise measurements were also made. It was found that in general the DLPA is much quieter compared to dense asphalt. However, noise levels at the DLPA increases with time, most likely due to clogging. Measurements made before and after cleaning shows the favourable effects of cleaning in terms of noise reduction. Figure 15 shows portion of the PA section that has been clogged by animal droppings or their carcass. Pictures of site and equipments used during the measurements are shown in Figure 16.

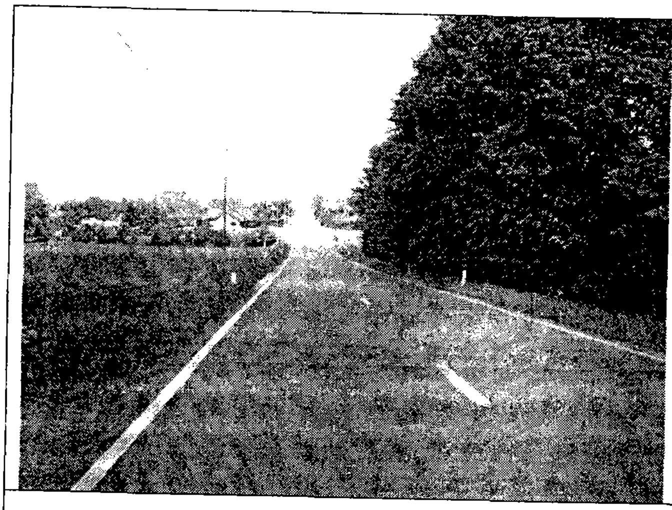


A Typical Example of Spots



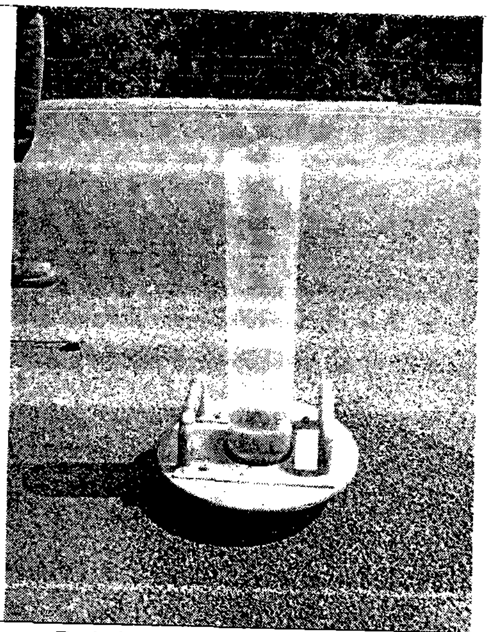
PA Surfacing Clogged by Horse Droppings

Figure 15 Example of Spot and Clogged PA Surfacing



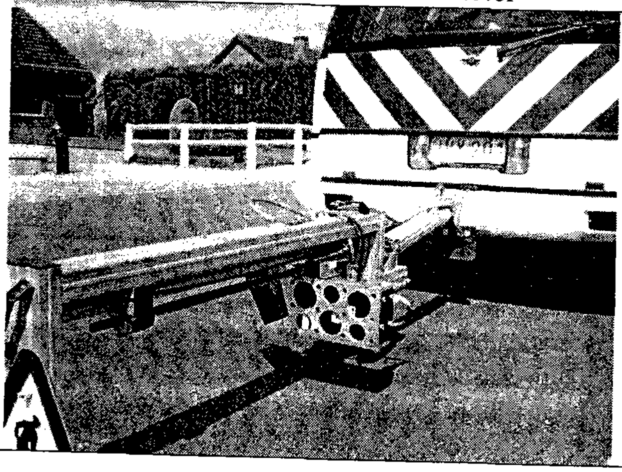
The Site at Bambois

The Author with Mr Claud at the Test Site

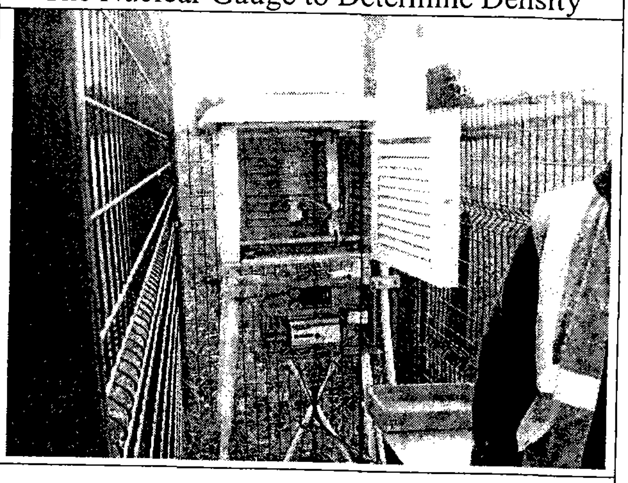


Belgium Field Permeameter

The Nuclear Gauge to Determine Density



Laser Profilometer for Measuring Texture Depth



The Weather Station at Bambois Site

Figure 16 Pictures of Site and Equipments Used for Monitoring

12.0 ASPHALT FOR AIRPORT RUNWAY

The Netherlands Airport Consultants (NACO) is probably the most credible airport consultant in the world. NACO has been consultant to more than 580 airports in all parts of the world. Current major project is the Beijing International Airport. The specifications for the KLIA runway were based on the NACO. A visit was made to the NACO office in Anna Van Saksenlaan, Den Haag. The office was conveniently located close to Den Haag Laan Van Nooi railway station. The main officer in the discussion was Mr. Alur Nataraj, Figure 17, who is the senior (retired) airport planner and pavement specialist. Others met included Mr. Arjan Kuin and Mr. Fer Mooren, the former was responsible for the construction of KLIA. The prime aim of the meeting was to get acquainted with NACO's innovations in airport runways, to know the current runway specifications and to ascertain any differences with the then KLIA runway specifications.

At NACO Headquarters in Den Haag

Discussion with Mr. Nataraj

Figure 17 Visit to NACO

Eventhough the scope of the author's research attachment focus on PA and does not cover airport runways, nevertheless, there are similarities between asphalt for roads and those for airport runways. NACO current pavement design incorporates a PA layer that functions primarily as a crack relief layer. Few weeks prior to the departure to TU Delft, the author was involved in the rehabilitation project for the Penang International Airport Runway with Mr. Walter Tappeiner as the advisor. In that project, the specifications for the resurfacing materials were identical to that of KLIA.

During the construction of the KLIA runway, polymer modified asphalts (Novophalt and sealoflex for Runway 1 and EVA for Runway 2) were specified. Since experience with polymer modified asphalt (PMA) in Malaysia at that time was very limited, the (old) IKRAM, insisted on good quality control. Mr. Walter Tappeiner of Advanced Asphalt Technologies was taken in for this purpose. Until now, the KLIA runways have shown good performance.

Based on NACO's field trial and experience with polymer binder (PMB) since 1994, the SBS modified asphalt performed better compared to other asphalt mixes made with EMA or EVA and is the most preferred PMB. The break strain of SBS modified bitumen is superior compared to all other PMBs. For instance, mixes modified with EMA could not meet NACO's current stringent specifications.

NACO introduced many innovations in the field of airport runways. This includes:

- Specifications for fuel resistance. Fuel spillage from aircraft, especially on take-off, has the potential to dissolve bitumen causing the asphalt mix to disintegrate. This problem may not be particularly prevalent for the runway proper but very relevant for mixes used on the taxiway. Currently, the specifications for fuel resistance is accepted worldwide. It was found that the SBS modified mixes are not fuel resistant and further modification has been made to the SBS PMA. Shell for instance has developed a binder named fuelsafe that is fuel resistant when used in asphalt mixes.
- Specifications for toughness value. The test is carried out by subjecting the specimen to an indirect tensile force and the specimen progressively loaded to failure. Toughness is defined as the area under the curve of the plot between force and displacement. Without this graphical relationship, only the maximum load that the specimen can withstand can be determined. However, the maximum load does not differentiate the load sustaining capacity of the specimen up to and beyond failure. The toughness value indicates the flexibility of a specimen beyond the failure load or the capacity of the material to absorb energy. This is important in the assessment of the crack relief capacity of a particular layer.
- Use of asphalt crack relief layer to mitigate reflective cracking. This has been used by NACO with success in many airport runways including the KLIA runways. This will be explained later.
- Use of cement treated basecourse layer. NACO recommends the use of low quality or even recycled materials including concrete from demolished buildings, low quality aggregates or even approved steel slag for this layer. However, this layer must be pre-cracked otherwise the cracks will be reflected into the upper layers. In the absence of pre-cracking, a crack relief layer as explained earlier is essential to mitigate the problem.

Mr. Nataraj rightly pointed the failure of highway pavement specialists to appreciate the loading scenario of airport runways. For instance, in the context of airport runways, fatigue cracking is not an issue due to the infrequent load repetitions. For airport runways, voids content is specified at 2%. This alarmed many highway pavement engineers fearing that it may cause bleeding or fatting-up. However, in highway pavements, bleeding is caused by voids closure due to overcompaction by millions of traffic axle loadings. However, the number of load repetitions on airport runways is very low and bleeding is not a frequent occurrence. Rutting or permanent deformation is also not a serious issue due to the short time of loading. At the apron, concrete pavement is used. On the taxiways, some aircrafts may have to queue, not withstanding this fact, the loading time is not lengthy. Hence, proper binder has to be selected to satisfy the true needs of an airport runway. For instance, EVA mixes are known to exhibit superior resistance to creep but rutting is not an issue. The issue associated with runway pavements remain ageing, reflective cracking and skid resistance. Another important issue is the use of indigenous materials especially in the construction of airport in developing countries.

A lengthy explanation was given on the use of open-graded or PA as a crack relief layer (CRL) to prevent propagation of reflection cracks in airport runways. Historically, the material has been used successfully in road pavements in the 1980's in the USA culminating in the issuance of a Technical Bulletin 4 by the Asphalt Institute. The high voids in the CRL helps to dissipate stress and arrest crack propagation. The first experience with CRL was in the rehabilitation of an airport in Saudi Arabia many years ago that had badly cracked to the full depth. Initially the client doubted the ability of the material to sustain heavy loads imposed by heavier aircrafts, its structural contribution and susceptibility to ageing. For the CRL material, NACO adopted PA that was then widely used in the Netherlands. Triaxial tests on cylindrical specimens were carried out on the mix at TU Delft. At the moment, this test continues to be carried out regularly at TU Delft on unbound materials. The main aim of the test is to see the effect of confinement on the strength and stability of the CRL material. The results of the tests recorded large initial strains beyond which further development of permanent deformation is very limited. After 7200 cycles, the axial deformation of all specimens tested was less than 1%. Clearly, the CRL is expected to resist rutting under heavy aircraft loading due to the presence of confinement in the field. Ageing of the mix was expected to take place due to high voids. However, ageing can be expedited in the presence of ultra-violet light and oxygen. Since the CRL mix is placed underneath the asphalt surface layer, it will be protected from the weathering agents. The KLIA runway also incorporates a CRL. The pavement structural layer is shown in Figure 18.

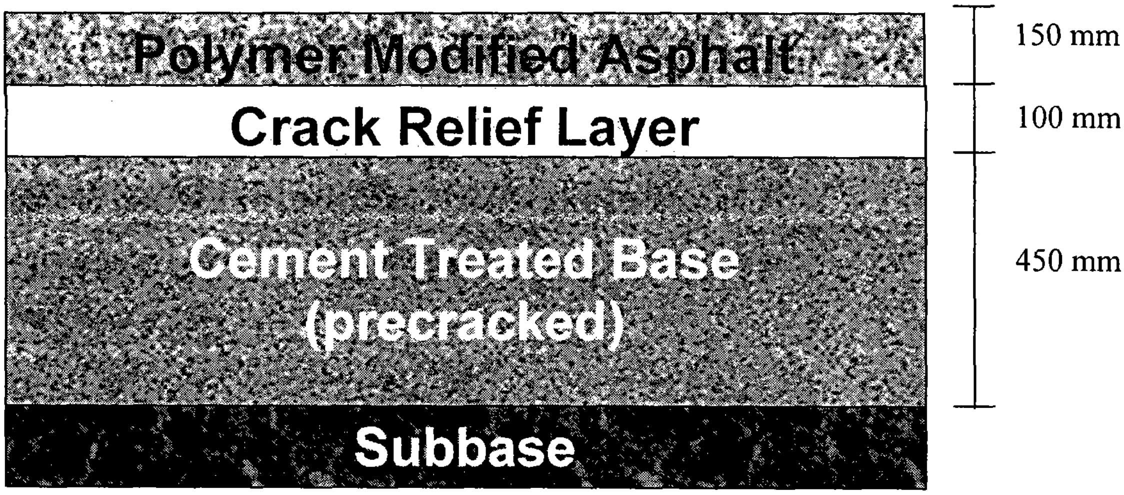


Figure 18 Schematic Drawing of the Pavement Structure for the KLIA (Source: Nataraj)

In the KLIA specifications, and the specifications for the recent Penang International Airport Runway rehabilitation, one of the specification items is a limiting static creep at 75 MPa. The author queried the relevance of static creep in the context of airport runways. Mr. Nataraj and Mr. Kuin affirmed that this item has been withdrawn from NACO specifications. Static creep was specified for the KLIA runways due to the limited availability of dynamic testing facilities at that moment in time. NACO suggested for the removal of this item in future Malaysian airport runway specifications.

The current NACO specifications for airport runway is shown in Table 11.

TABLE 11(a)
Aggregate Requirements

	Test	Size	Requirement	Reference
	Physical Properties			
-	Grading	all	Table 11(c)	
-	Shape			
	- Two or more fractured faces	coarse	> 95% (w/w)	Clause 2.1.2
	- One or more fractured faces	coarse	> 100% (w/w)	Clause 2.1.2
	- Flat and elongated pieces	coarse	< 8% (w/w)	Clause 2.1.2
-	Water Absorption	coarse	< 2%	ASTM C 127
		fine	< 2%	ASTM C 128
-	Sand Equivalent	fine	> 65%	ASTM D 2419
	Mechanical Properties			
-	Aggregate Impact Value	10-14 mm	< 30%	BS 812
-	Ten Percent Fines Value	10-14 mm	> 150 kN	BS 812
	Durability			
-	Wear (Los Angeles Abrasion Value)	coarse	< 35%	ASTM C 131
-	Sodium Sulphate Soundness Loss (1)	coarse	< 9%	ASTM C 88
-	Magnesium Sulphate Soundness Loss (1)	coarse	< 12%	ASTM C 88
	Chemical Properties			
-	Clay/friable particles	Fine	< 3%	ASTM C 142
-	Plasticity Index	Filler	Non plastic	ASTM D 4318
-	Swelling	filler	< 0.7 gm/100g filler	Pr EN 933-9
(1) only one of these tests needs to be conducted. Tests to be conducted at five cycles. Tests are not applicable to high carbonate aggregates ($CaCo_3$ > 65%).				

TABLE 11(b)
Properties of Modified Bitumen

SUPERPAVE PERFORMANCE GRADE		PG 82-10	
Property	**Test Condition**	**Requirement**	**Test Designation**
ORIGINAL BINDER			
Flash Point		> 230 °C	ATSM D 92
Viscosity	135 °C	< 3Pa.s (3000 cP)	ASTM D 4402
Dynamic Shear, G*/sin delta	82 °C @ 10 rad/s	> 1.00 kPa	AASTHO TP 5
Softening Point		> 65 °C	ASTM D 36
ROLLING THIN FILM OVEN (AASHTO T240)			
Mass Loss		< 1.00 %	
Dynamic Shear, G*/sin delta	82°C @ 10 rad/s	> 2.20 kPa	AASTHO TP 5
PRESSURE AGEING VESSEL RESIDUE			
PAV Ageing Temperature	110 °C		
Dynamic Shear, G*.sin delta	40 °C @ 10 rad/s	< 5,000 kPa	AASTHO TP 5
Physical Hardening		Report	
Creep Stiffness *	0°C @ 60 s	S < 300 MPa m-value >0.300	AASTHO TP 1
Direct Tension	0°C @ 1.0 mm/min	failure strain > 1.0%	AASHTO TP 3
* If creep stiffness is less than 300 MPa, then Direct Tension Test (TP-3) is not required.			

TABLE 11(c)
Marshal Design Criteria

Criterion	Requirement
Number of Blows	2 x 75
Stability, in Newton	> 15,000
Flow, in mm	3 – 6
Air Voids, percent	2 – 4
Marshall Quotient, N/mm (= Stability/FLow)	4,000 – 6,000
Percent Voids in Mineral Aggregate (VMA)	see Table 5

TABLE 11(d)
Stage 2: Modified Mix Performance Requirements

Property	Test Conditions	Requirement **	Test Designation
Air Voids		2 – 2.5%	
Indirect Tensile Strength	0 °C 25 °C	Average > 3.5 MPa Minimum > 3.15 MPa Average > 1.0 MPa Minimum > 0.75 MPa	ASTM D 4123
Toughness	0 °C 25 °C	Average > 7.5 N/mm Average > 7.5 N/mm	
Total Resilient Modulus	0 °C @ 1 Hz at stress 30% of IDT 25 °C @ 1.0 Hz at stress 15% of IDT	< 20,000 Mpa)average > 10,000 MPa) > 2,500 MPa	ASTM D 4123
Dynamic Creep Test Hydraulic or pneumatic controlled equipment	- Temperature 40 °C - Static Load 400 kPa - 10,000 load repetitions - Square loading pulse 0.2s load, 0.8s rest	Perm. Deform. < 1.5% Creep Slope < 0.25 (log-log)	
** A minimum of four (4) tests shall be performed to determine the average			

13.0 OBSERVATIONS AND RECOMMENDATIONS

The Netherlands is truly the land of porous asphalt and with many success stories. Some PA sections survived up to 16 years. There is great governmental encouragement to the use of PA. By 2010, all highways in the Netherlands will be paved with PA. However, the material was adopted from the standpoint of noise reduction in harmony with European and Dutch legislations on noise.

A number of well-funded research organisations undertook the issue of developing quite pavements or silent roads such as the DWW in the Netherlands and the BRRC in Belgium. Impartial organisations such as these conduct and monitor research on various aspects of asphalt technology. The bulk of the funding can come from the government either directly such as the DWW or indirectly such as that practiced by the BRRC.

Dutch contractors seriously undertook innovation by establishing RND departments. In Heijmans Infrastructuur, the RND department is a stand alone department and is not part of the QA and QC department. This enables the RND department to focus solely on providing and developing innovative solutions to highway problems and to translate them to practice. Inventions and innovative ideas from Heijmans' RND department extend well beyond DLPA, Microflex surfacings and Stabiloflex, but into the realms of decorative asphalt, tapping energy from pavements and embedded rail in asphalt (ERIA) to reduce noise from trams. The noise reduction from the ERIA project, carried out in association with TU Delft, was proven from a trial section in Den Haag. The main factor that motivates Dutch contractors to establish RND department is the recent implementation of performance contract. Under this contract, a contractor is required to provide a guarantee period of seven years and the contractor will receive a bonus if the road exhibits good performance at the end of the guarantee period. Under such a competitive climate, creating innovations is the way forward to ensure survival. The MSc thesis of Ton van der Steen on DLPA was sponsored by KWS, one of the main Dutch road contractors. Malaysian contractors should be encouraged to establish RND department. The government can play the role of accelerating the need of innovative solutions from local road contractors.

Dutch road authorities and industries are blessed with the expertise available at the Section of Road and Railway Engineering at TU Delft under the able leadership and guidance of a world authority in asphalt, Professor Molenaar who works closely with Associate Professor Martin van de Ven and his other academic staffs. The expertise of Dr. Rien Huurman in computer modelling is another asset to the Section. The DWW is conveniently located within the vicinity of the Section of Road and Railway Engineering. There are many postgraduate research projects supervised by both parties and sharing of equipments and resources is at its best practiced here. Both TU Delft and the DWW housed state-of-the-art laboratory equipments with excellent and very competent technical back-up.

Sometime in the future, traffic noise will become an issue in Malaysia. This is more evident as more expressways are planned to traverse residential or developed urban areas. A proactive stance should be taken by gearing research to develop quite pavements or silent roads so that noise can be reduced at source.

There is a wealth of knowledge and experience with porous asphalt residing in the Netherlands. This expertise has to be made known to the Malaysian road community. The bi-annual Malaysian Road Conference has long established as the premier conference in Malaysia attracting huge crowds. It is recommended that a special session on PA during the 7th MRC due in year 2006 with Dutch panel of speakers from TU Delft, DWW and Heijmans Infrastructuur BV. The author can relate research on single and double layer PA that has been carried out at the Universiti Sains Malaysia over the last 10 years.

In asphalt technology, there are some Dutch practices that are unique. Dutch aggregate grading specifications expressed the limits in terms of cumulative percentage aggregate retained. In most countries, the value is expressed in terms of cumulative percentage passing. For instance, the smallest sieve in a gradation chart is 63 micron instead of the more typical 75 micron which relates to the definition of a filler material. Dutch practice specifies bitumen content as a percentage of dry aggregates. When expressed as a percentage of total mix, as is common practice, the actual binder content is lower. Dutch specifications for PA also permit hydrated lime content up to 4.5%. This value is rather high. The beneficial effects of hydrated lime to mitigate stripping are well documented in the literature. However, when hydrated lime is blended in an asphalt mix, it becomes part of the filler-binder system rather than the filler-aggregate system causing the binder to stiffen considerably when used in high quantities. This may hasten ravelling.

The phenomenon of bitu-planing should be more serious on Malaysian road surfacings that are exposed to higher temperatures. Hence, an in-depth study is required based on Dutch experience.

The application of CT scan technique to analyse asphalt is truly impressive. It also indicates the importance of highway material researchers to communicate with specialists from other disciplines. More meaningful results can be attained if the CT scanner can detect particles less than 2 mm. Nevertheless, the author believes that the CT scan technique offers the following possibilities that could not be achieved using conventional methods of asphalt core analysis:

- In the context of PA, it is possible to map out the degree, extent and severity of clogging over the entire area and depth of the asphalt core and slab.
- Study on the packing behaviour of aggregate mass so that the aggregate proportion that would lead to the most stable matrix can be obtained. Better aggregate gradations can be designed in this manner.
- Porous mix is known to suffer segregation both in terms of binder content and aggregate. The CT scan technique can be to profile the extent of segregation and probably assign an index.
- From the slice to slice analysis, the aggregate surface area can be calculated. Once aggregate surface area can be computed, it is possible to determine the bitumen content for a desired bitumen film thickness that coat the aggregate. However, due to the current resolution o f the CT scanner used, it is quite difficult to distinguish particle size less than 2 mm. Over the years, the Asphalt Institute Surface Area Factors have been used to calculate aggregate surface area in dense asphalt gradations. Not only are the factors inappropriate for PA, the literature behind the development of those factors is not available.
- It is possible to study the aggregate orientation and hence evaluate appropriate laboratory compaction procedure that mimics field roller compactor.

Specifications for future airport runways or runway resurfacings in Malaysia need to be updated. To keep up with availability of modern dynamic testing facilities in Malaysian universities, the clause on static creep requirements should be removed. The specified creep stiffness 75 MPa was based on practice in the USA and hence the creep test parameters must also be adopted accordingly. The 1977 static creep test configurations adopted during a colloquium on plastic deformation of bituminous mixes in Zurich, Switzerland would lead to stiffness values less than 30 MPa. The USA practice stresses the importance of proper seating of the loading platen prior to the actual creep test so that the actual strain deformation measured is that which is due to the plastic deformation of the mix. Professor Carl Monismith at the University College Berkeley (UCB) carried out extensive creep tests on asphalt mixtures. The graph shown in Figure 19 is based on work carried out at UCB.

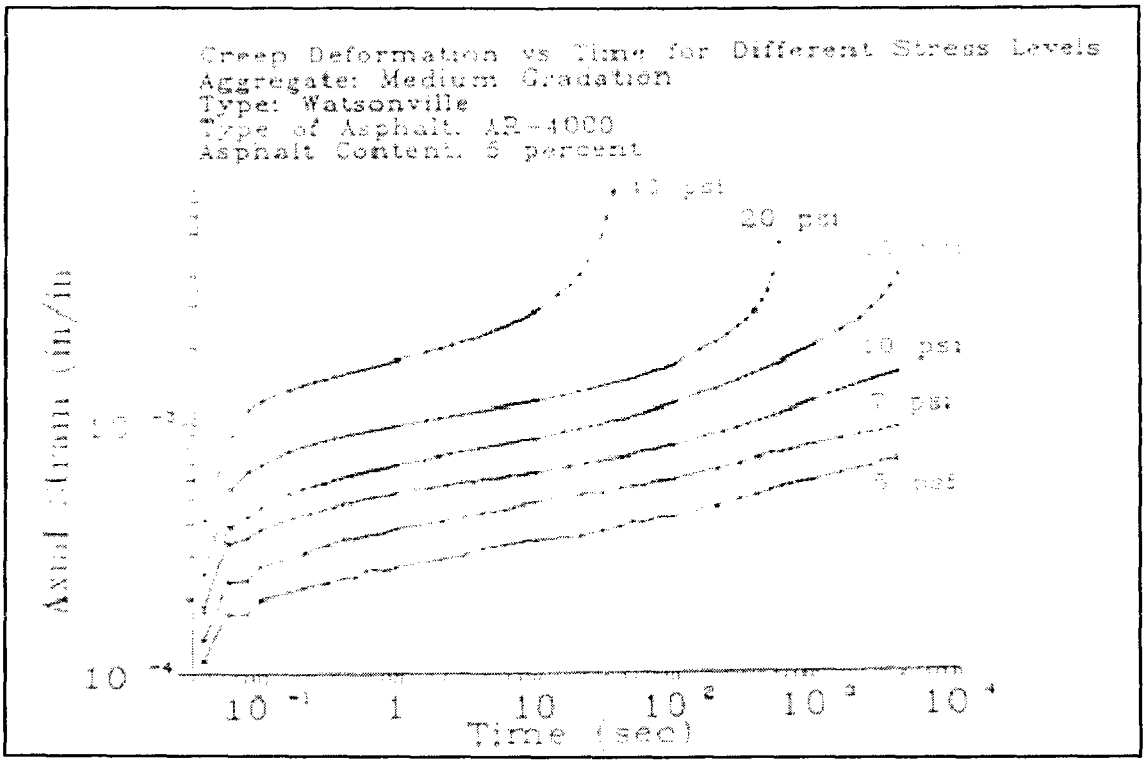


Figure 19 Relationship Between Axial Strain and Time of Loading at Various Stress Levels

The graphs clearly indicate the magnitude of the applied stress significantly influence the deformation behavior of asphalt mix. Up to about 100 kPa (14.5 psi), the strain response of the (unmodified) asphalt mix is about linear but at higher stress levels, plastic failure occurs before the end of the one-hour loading time. This rationalizes the necessity to condition specimens at higher load prior to the actual static creep test. If the specifications for static creep remains, then the test procedure in NCHRP Report 338: Asphalt-Aggregate Mixture Analysis System (AAMAS) should be used as the reference and the recommended modified parameters are as follows:

- Pre-Conditioning: Repeated Load Testing Program using 45 Load Pulses @ 150 kPa (with a rise time of 60 ms and a frequency of 1 Hz); after the conditioning, reset LVDTs to zero.
- Testing: Static Creep Test program; apply 300 kPa for one hour; with recovery measurements if needed.

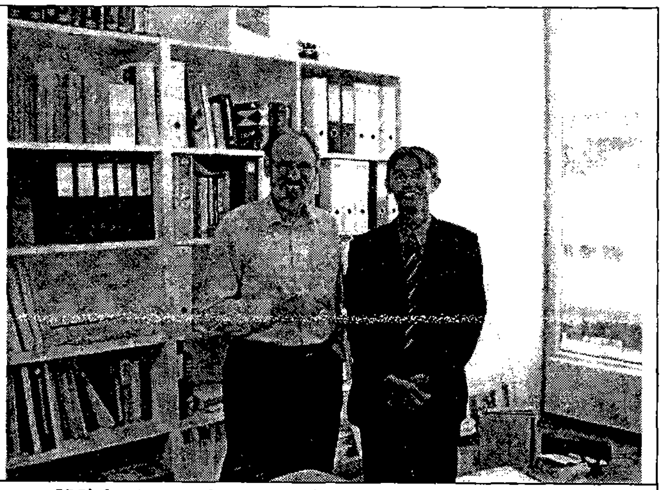
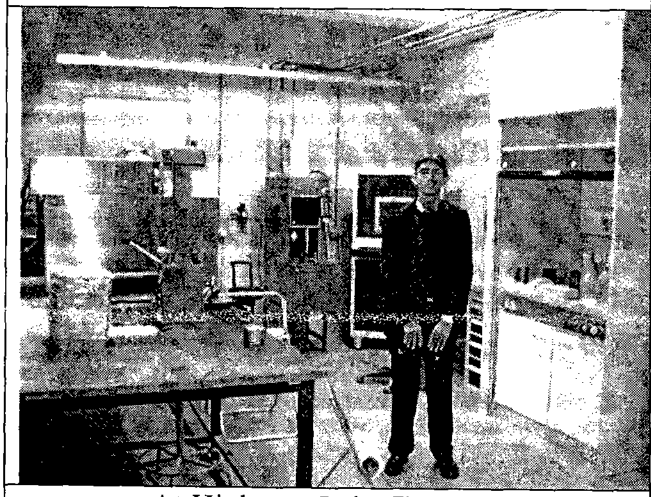
Appendix A

ITINERARY

Date	Activities
14 April 2005	Depart from house in Parit Buntar to Bayan Lepas, Penang. Flight from Penang International Airport to Amsterdam Schiphol Airport
15 April 2005	Arrival at Schiphol Airport. Travel to Delft on the same day, arrive TU Delft and meeting with Professor A.A.A. Molenaar and Dr. Martin on tentative program. Allocated a room at the Section of Road and Railway Engineering, TU Delft. Then visit DUWO for accommodation.
16 – 17 April 2005	Weekend holiday
18 April 2005	Settling down in TU Delft Office
19 April 2005	Going through literature and European Standards given by Dr. Martin
20 April 2005	Meeting PhD work of Eyassu on ageing of binder in porous asphalt with Prof. Molenaar, Martin and staffs from DWW and IPG
21 April 2005	Meeting with Jan Voskuilen (single layer PA) and Koos van Wieringen (double layer PA) at DWW
22 April 2005	Going through AAPT and TRB papers. Visit lab at TU Delft – Specimen preparation procedures and practices
23 – 24 April 2005	Weekend holiday
25 April 2005	Moving to new offices. Offices of the Section of Road and Railway Engineering move to a new building next door close to the laboratories
26 April 2005	Meeting with Dr. Rien Huurman on modelling of PA
27 April 2005	Meeting with Mr Gerbert van Bochove of Heijmans Infrastructuur B.V. at TU Delft on PA, silent thin surfacings and semi-flexible pavement. Also site visit to a highway paved with rubberized two-layer PA. Mr Gerbert, the inventor of double layer PA has arranged for a return whole day visit to his office in Rosmalen on the 20th of May 2005. Focus of discussion would be on construction of PA and other new pavement materials
28 April 2005	Laboratory tour of DWW laboratory focusing on bitumen lab together with Dave van Vliet and Eyassu. DWW laboratory is well-equipped for cutting edge research. Then discussion with Mr Erik Jan Scholten from Kraton Polymer on binder modification with SBS. According to Mr Erik, linear and radial SBS polymers are respectively suited for pavement and runway applications. Then visiting Ing. Wim Verwaal , Head Laboratory Engineering Geology, Department of Geotechnology on CT scanning of PA. Arrangements were made to visit a CT scan presentation the following day setting off from TU Delft at 8.00 am.
29 April 2005	Outstation presentation trip with Dr. Martin and Wim Verwaal on Jansen de Jong: viaral as an alternative two-layer system cancelled due to my poor health
30 April 2005 - 1 May 2005	Weekend holiday
2 May 2005	Triaxial test on porous aggregate skeleton at TU Delft – Specimen preparation and testing
3 May 2005	Visit to NACO at Den Haag for a discussion on airport runway specifications
4 May 2005	Laboratory test procedure at TU Delft – Indirect Tensile Test (dry and moisture induced)
5 and 6 May 2005	TU Delft closed in conjunction with Liberation Day.
7 – 8 May 2005	Weekend holiday

9 May 2005	Laboratory tests at TU Delft – Direct Shear Test and Fatigue Test
10 May 2005	Scheduled SMA production meeting at Ureterp with Dr. Martin and DWW postphoned to 26th May 2005. Attend lecture on 'Application of Artificial Neural Network in Asphalt' by Professor Halil Ceylan from Iowa State University at TU Delft
11 May 2005	Discussion with Dr Martin on various aspects of laboratory tests and European standards.
12 May 2005	Discussion with PhD student Patrick Muraya on Dutch practice in porous asphalt
13 May 2005	Sorting out short stay accommodation with DUWO
14 – 15 May 2005	Weekend holiday
16 May 2005	Public holiday
17 May 2005	Meeting with Mr Robert Naus on construction of PA. In the evening, attend MSc thesis public lecture on double layer porous asphalt
18 May 2005	Journey to Brussels, Belgium with Dr. Martin. Discussion with Mr Calude from Belgium Road Research Centre (BRRC). Observe taking measurements on the only Belgium DLPA field trial in Bambois. Then back to BRRC for a discussion on noise attenuation of porous asphalt with Mr. Luc.
19 May 2005	Presenting research findings entitled "Porous Asphalt Research at the Universiti Sains Malaysia" to TU Delft. Discussion with Jan Voskuilen and Koos van Wieringen on porous asphalt construction. Meeting with Professor Molenaar and Dr. Martin. Last working day at TU Delft
20 May 2005	Whole day visit Heijmans Infrastructuur BV. Discussion and presentation with Gerbert van Bochove on PA, stabiloflex, thin surfacings and construction method. Also view construction of thin layer surfacings which is similar to the top layer of a double layer PA
21 – 22 May 2005	Weekend holiday
23 May 2005	Took taxi from Delft to Schiphol Airport. Flight from Schiphol Airport to Penang
24 May 2005	Arrive at Penang International Airport. Journey home to Parit Buntar

Appendix B

	
With Section Head, Professor Molenaar	With Associate Professor Dr. Martin who arranged my schedule throughout my stay at TU Delft
	
With MSc Students Tufail and Daniel	Working session with Dr Rien and PhD Students Eyassu and Patrick
	
At Highway Lab, TU Delft	At Highway Lab, TU Delft

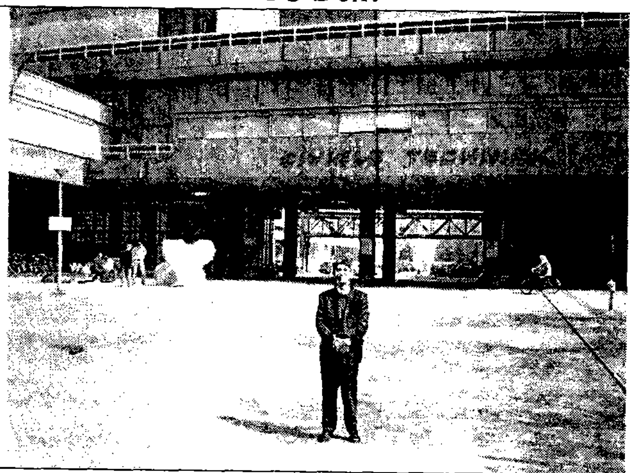
With Jan at Highway Lab, TU Delft

With Seeth Molenaar at Highway Lab, TU Delft

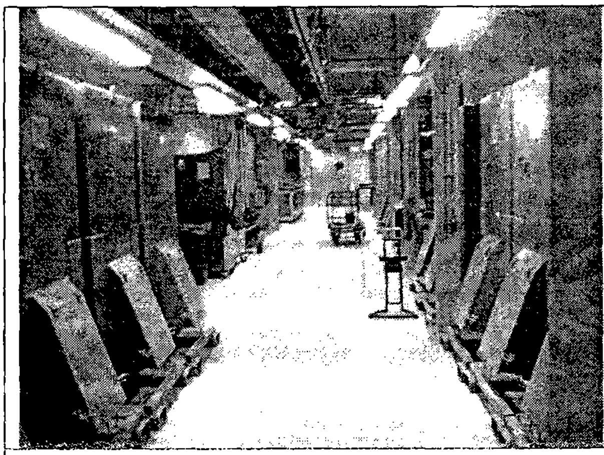
After discussion with Robert Naus at TU Delft

After discussion with Gerbert Van Bochove at TU Delft

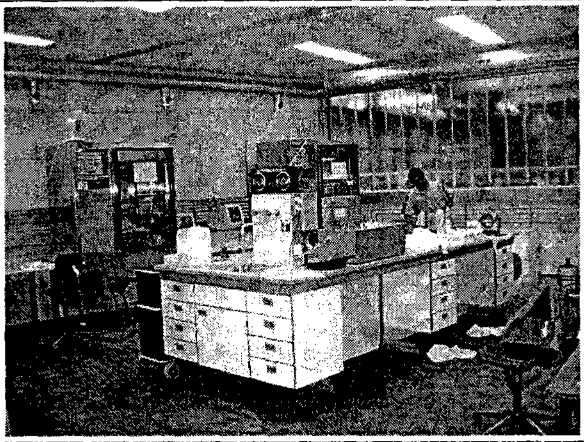
With Eyassu of TU Delft and Dave of DWW



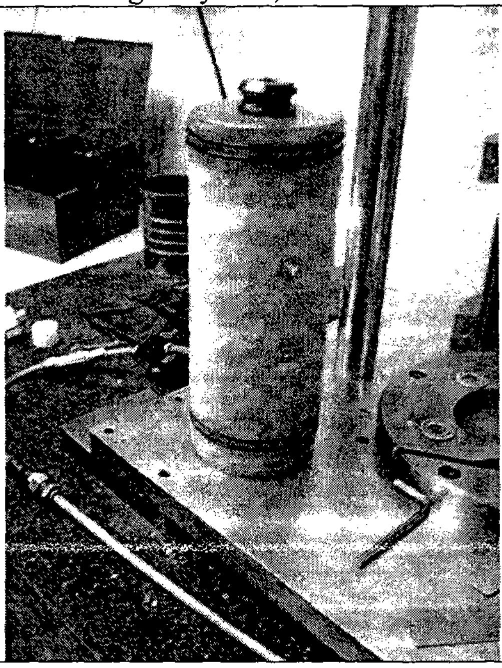
In front of Faculty of Civil Engineering and Geosciences, TU Delft



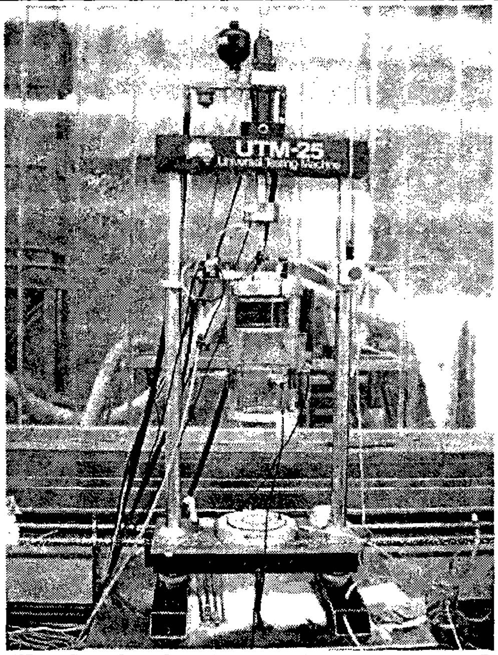
Highway Lab, TU Delft



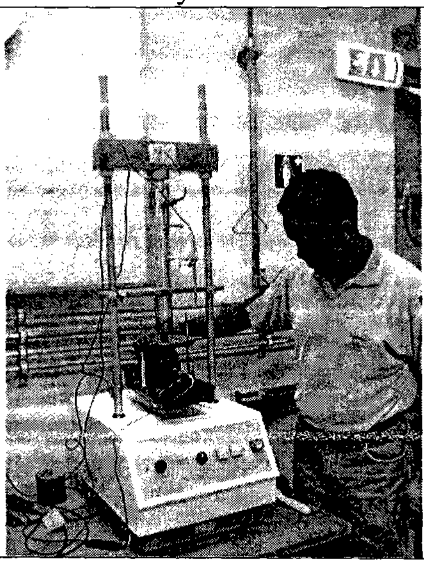
Highway Lab, TU Delft



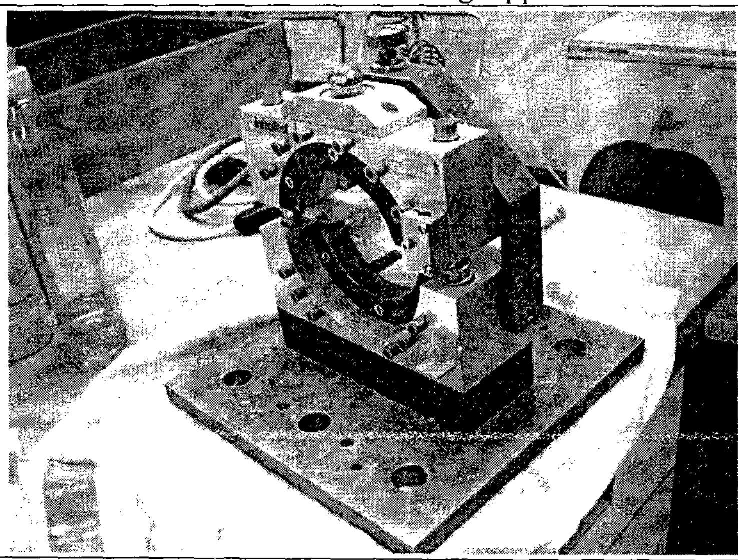
Specimen Ready for Triaxial Testing



IPC UTM Triaxial Testing Apparatus



Indirect Tensile Testing Equipment Capable of Measuring Horizontal and Vertical Deformation



Asphalt Direct Shear Test Apparatus