

WHITETOPPINGS - SWEDISH FIELD TESTS AND RECOMMENDATIONS

Johan Silfwerbrand, Ph.D., Assoc. Professor
Dept. of Structural Engineering
Royal Institute of Technology (KTH)

ABSTRACT

In order to find methods to restore rutted asphalt pavements and increase their rutting resistance, field tests have been carried out with thin, high strength concrete overlays (whitetoppings) placed on top of the milled asphalt pavement. The concrete overlay was limited to 70 mm. Both plain and steel fibre reinforced concrete have been studied. Two different joint spacings have been investigated: 1.25 and 3.5 m. Also a test section with an overlay consisting of open asphalt filled with high strength cementitious mortar has been included in the investigation. After three years in service, the test section performance is promising. Besides test results, the paper provides recommendation for the construction of whitetoppings and suggests needs for further research.

1. INTRODUCTION

1.1 Background

Track bound rutting is common along the Swedish roads. The rutting is especially obvious on old, heavily trafficked, flexible pavements. The rutting is caused by the frequent use of studded tyres during winters and by heavy vehicles during hot summer days when the asphalt pavement is soft. Heavily trafficked roads have to be repaired and the rutting adjusted very often.

A new asphalt surfacing will solve the problem temporarily, but is not a long-lasting solution, since new rutting will occur after a rather short time. By using high quality asphalt materials and wear resistant aggregate, the winter wear and the summer deformations can be reduced. Measurements on, e.g., the Swedish Arlanda road, where a concrete pavement has been placed on the west bound direction, show that rutting can be further reduced by using concrete instead of asphalt (Hultquist and Carlsson (2)). Consequently, placing a thin high strength concrete overlay (a whitetopping) on top of the old asphalt pavement is an interesting renovation technique. High strength concrete with high quality aggregate has a high wear resistance. Concrete is also a stiff material. Contrary to asphalt, the stiffness is independent of temperature. By using concrete, we will obtain an overlay with less winter wear and no summer deformations. New concrete pavements are usually about 200 mm thick. On old asphalt pavements, the thickness has to be much less due to both technical and economical reasons. Otherwise, the connections against the shoulders will be difficult, the clearances under bridges too small and railings too low. On the other hand, the overlay must not be so thin that it cracks under traffic load.

1.2 Experiences from Previous Tests

Whitetoppings have been built in both Europe and USA. A literature survey has been written by Petersson et al (4) and summarised in Petersson and Silfwerbrand (5). In USA, whitetoppings have been used since 1950. Usually, they are either 125 mm thick jointed concrete pavements or 150 mm thick continuously reinforced concrete pavements.

Experiences of very thin whitetoppings needed to fulfil Swedish demands, i.e., overlay thickness less than 100 mm, are, however, limited. Field tests have been carried out in Louisville, Kentucky, USA (Risser et al (7)). Two test sections were 50 and 90 mm, respectively. In both cases, the pavement was sawed into square shaped joint patterns. On the thick section, the joint spacing was 1.8 m. On the thin section, the joint spacing was sometimes 1.8 m, sometimes 0.6 m. The test sections were exposed to heavy traffic. After two years, only minor cracking had been observed. The short joint spacing is likely to contribute to the promising result.

In 1989, a 375 m long test section was built on the north bound lane of the old European road No. E4, today Swedish national road No. 255, 10 km north of Märsta and 50 km north of Stockholm (Petersson and Silfwerbrand (5)). In 1987, the average traffic consisted of 2700 vehicles every day (total traffic on two lanes). In 1993, this figure was slightly less, 2400 vehicles every day. The amount of heavy vehicles is rather high (>10-15 %). The Swedish design axis load is 100 kN.

On the milled asphalt, 70-90 mm steel fibre reinforced concrete was placed. The concrete strength was about 80 MPa (Swedish concrete grade K80). The pavement was cast without joints, with exceptions for six or seven construction joints. Today, a longitudinal crack along half the test section and several wide transverse cracks can be observed at the site. Coring has, however, shown that the asphalt beneath the concrete overlay is very thin, sometimes even absent. Despite the cracking, the test results can be regarded as promising, at least, if you consider the presence of heavy vehicles, a very thin asphalt layer, and a long joint spacing.

1.3 Aim of the Study

The primary aim of the study was to find new methods to repair old asphalt roads with extensive rutting. Plain and steel fibre reinforced whitetoppings have been examined as well as an overlay consisting of open asphalt subsequently filled with a high strength cementitious mortar (Densiphalt). In the paper, field tests on whitetoppings are described and discussed. Bond strength measurements and crack and damage surveys are emphasised.

2. DESCRIPTION OF FIELD TESTS

The construction of the field tests is described in detail in the field test report (9). The most important items are summarised and discussed below.

In Subsection 1.2, the 1989 field tests close to Märsta are briefly discussed. The test section was situated on a small hill. Consequently, the subgrade strength is likely to be high. The project group decided to use the south bound lane of the same road at the same location for the new field tests. Consequently, the new test sections were exposed to the same traffic loads and volume as the previous ones, see Subsection 1.2 above.

The asphalt layer needs to have a significant thickness to ensure sufficient load carrying capacity. Calculations have shown that the asphalt layer needs to be 150 mm beneath a 70 mm thick concrete overlay, see (9). The calculations were carried out for an average Swedish pavement exposed to 10 millions 100 kN standard axes and resting on clay subgrade, i.e., with a 600 mm thick untreated gravel base underneath the asphalt layer. On the previous field test, coring showed that the thickness of the asphalt layer at some locations was limited to 20 or 30 mm. That resulted in longitudinal cracking. Before starting the new field test, coring was carried out in order to discover and avoid areas with a very thin asphalt layer. On some small spots, new asphalt was added on top of the old one to ensure a sufficient asphalt thickness of 150 mm for concrete overlays and 120 mm for Densiphalt (open asphalt subsequently filled with a high strength cementitious mortar, see below). The asphalt thickness was found to be suitable for this site. Other subgrade conditions and traffic loads may lead to other demands.

In order to promote a good bond between asphalt and concrete or Densiphalt, the asphalt was milled before overlay placement. The milling depth varied between 5 and 60 mm.

There were four new test sections with a total length of 450 m. The width was 3.75 m. The tests comprised the following overlays:

- 70 mm steel fibre reinforced concrete (SFRC) overlay with conventional joint spacing
- 70 mm SFRC overlay with short joint spacing (Figure 1)
- 70 mm plain concrete overlay with short joint spacing
- 40 mm continuous Densiphalt overlay

The main data of the four test sections are summarised in Table 1. Coring after completed construction showed the following averaged thickness values:

Densiphalt (2 cores)	40 mm
Plain concrete and SFRC (12)	74 mm (60 - 90 mm)
Asphalt (14)	161 mm (130 - 220 mm)

The range of the thickness values are given in brackets. Only a very few cores did not fulfil the thickness demands according to Table 1.

The concrete mix was of Swedish concrete grade K80 (28 day compressive cube strength 80 MPa) summarised as follows:

aggregate 0-8 mm	840 kg/m ³
aggregate 8-16 mm	910 kg/m ³
cement	480 kg/m ³
silica	15 kg/m ³
water	150 kg/m ³
water/(cement+silica)	0.30

A Swedish Portland cement with a moderate strength development rate developed for civil engineering structures (construction cement, *anläggningscement*) was used. When computing water-binder ratio, cement and silica have been given equal binder efficiency. The use of silica fume did not cause any problems such as shrinkage cracking.

Concrete compressive strength was measured on cubes (side length = 150 mm) stored in the laboratory and on cores (diameter = 69 mm) taken from site. Flexural tensile strength was measured on beams (length×width×height = 800×150×100 mm) stored in the laboratory and loaded in third-point bending. The following average values were obtained:

Plain concrete, cube strength at 28 days	94 MPa
SFRC, cube strength at 28 days	86 MPa
Plain concrete, core strength at 280 days	105 MPa
SFRC, core strength at 280-281 days	111 MPa
Plain concrete, flexural strength at 29 days	8.1 MPa
SFRC, flexural strength at 28-29 days	7.2 MPa

The obtained flexural strength (cracking strength) of SFRC was lower than the flexural strength of plain concrete. The probable cause is slightly worse compaction due to the disturbance from the fibres. The fibres gave, however, considerable increases in ductility, see (9).

Flexural strength of Densiphalt was measured on beams (length×width×height = 160×40×40 mm) loaded in third-point bending. The average strength was 3.4 MPa at 14 days and 3.7 MPa at 43 days. After flexural failure, one of the ends of each beam was loaded in compression to estimate compressive strength. The average was 13.1 MPa at 14 days and 13.0 MPa at 43 days.

On test sections Nos. 2 and 3, 40 to 50 kg/m³ steel fibres (Dramix 30/0.5, i.e., a hooked fibre with fibre length = 30 mm, fibre diameter = 0.5 mm) were added. The concrete was placed with a slip form paver. The concrete surface was cured by spraying with a curing compound.

Test section No. 1 consisted of Densiphalt. Densiphalt is the trade mark of a material that is a hybrid between concrete and asphalt. It is based on a promising development in the Scandinavian countries. The aim is to find a material that combines the durability and wear resistance of concrete with the strain capacity of asphalt. The material consists of an open asphalt with aggregates between 8 and 12 mm and 25 to 30 percent void content. The voids are subsequently filled with a high strength cementitious mortar (Figure 2). The mortar is produced by the Danish company Densit A/S. Finally, the surface is cured by spraying with a curing compound. Since Densiphalt is assumed to have a high strain capacity, the Densiphalt layer may be placed continuously, i.e., without transverse joints.

Both asphalt placement and placement of Densit mortar were carried out during 8 hours. After overlay placement, all four test sections were cured by spraying with a curing compound. About 24 hours after concrete overlay placement, the

joint sawing was started. Intended sawing depth was 25 mm and intended sawing width 3 mm. It is important that sawing operations are carried out at an optimum time. Too early sawing leads to a risk of damaged joint edges, whereas too late sawing means a risk of uncontrolled cracking, i.e., cracking between the joints. Optimum time depends on, e.g., concrete mix, concrete overlay thickness, and weather conditions after overlay placement. For the new test sections, the joint sawing caused the only significant problem. Small edge damages have been found on several joints (Section 4). Causes are difficulties in determining optimum sawing time, the large amount of joints, and insufficient sawing capacity. The short joint spacing resulted in more than 900 m joints. The temperature was only a few degrees above freezing postponing most joint sawing to a weekend. Due to limited experience from similar projects and a limited project budget, neither the number of technicians nor machinery capacity was sufficient.

The test sections were opened to traffic in October 1993, less than one week after concrete overlay placement. Tests and measurements have been carried out on the surface roughness of the asphalt surface prior to overlay placement, on bond between asphalt and overlay, on temperature development within the overlay and on friction, evenness, and wear. No FWD tests were performed due to a limited budget. In the following Sections, bond tests and crack and damage surveys will be discussed. Remaining tests are described in the test report (9).

3. BOND BETWEEN OVERLAY AND EXISTING ASPHALT

A good bond between asphalt and overlay is essential for the behaviour of the composite pavement. The bond strength has been measured by drilling and testing cores with a nominal diameter $\phi=70$ mm. In all, 13 cores were drilled out of the four test sections on April 25, 1994, i.e., eight months after completed construction. The cores were drilled from the top of the overlay, down through overlay and asphalt layer. Thus, layer thicknesses and bond strength could be estimated from the same core. The cores were cut in order to provide the cores with end faces perpendicular to the cylinder axes. On the end faces, steel plates were glued. Subsequently, the steel plates were attached to the tension testing machine. Finally, the core was subjected to pure tension. The tests were carried out on May 11, 1994, by K. Lindgren at the Department of Structural Engineering at the Royal Institute of Technology in Stockholm. Between drilling and testing, the cores were stored in closed plastic bags.

Contrary to concrete strength, asphalt strength is strongly dependent on testing temperature and loading rate. Consequently, the choice of loading rate and testing temperature is important for obtaining useful and meaningful test results. High temperature and slow loading give very low strength values. Decreasing the testing temperature is complicated and expensive, but loading rates can easily be altered. Traffic loads have a very short duration and, consequently, a high loading rate may be suitable. The testing machine has, however, some limitations, and the results have to be repeatable. The Department of Structural Engineering has carried out a large number of bond tests (pure tension) to determine the bond strength between old concrete and concrete overlays. A loading rate of 500 N/s has been used for cores with a diameter $\phi=100$ mm. That means 0.064 MPa/s and corresponds to 245 N/s for cores with $\phi=70$ mm. Verhoeven (10) has tested bond between asphalt and concrete using shear tests according to Iowa Department of Transportation (3). Expressed in SI units, Iowa DOT uses a loading rate of 0.046 to 0.057 MPa/s. In present study, the loading rate was chosen to 250 N/s or 0.065 MPa/s for cores with $\phi=70$ mm. Note that these cores were tested in pure tension and not in shear as in the Iowa test.

The test results are summarised in Table 2. Note that the failure stress is a measure of bond strength only if there is an interface failure. For other failure modes, the failure stress is only a lower limit of the bond strength. The real bond strength might as well be higher.

The average failure stress for cores of concrete and asphalt was 0.25 to 0.30 MPa. At the sole interface failure, the failure stress was much higher (0.60 MPa). Thus, the bond strength might be considered to be fairly high. Through shear tests, Verhoeven obtained failure stresses around 1 MPa for some road sections of concrete overlay on asphalt. Pure tension is, however, more critical for the bond than pure shear. It has been shown in a comparing study for the case of new concrete on old concrete (Silfwerbrand (8)). Consequently, the result in this study ought to be equivalent to Verhoeven's results.

For cores of Densiphalt and asphalt, the average failure stress was 0.1 MPa. Interface failures were obtained for both cores. The reason for the fairly low value has not been determined.

It is also interesting to compare the bond strength with the magnitude of the shear stresses in the interface between concrete overlay and asphalt layer. A shear stress of 0.06 MPa has been computed for summer conditions, see (9). The computation was made with F. van Cauwelaert's computer program GIPI dealing with multilayer pavement systems (1).

Under winter conditions, when the asphalt is stiffer, the shear stress increases to 0.08 MPa. As shown above, measured bond strength exceeded 0.25 MPa. Consequently, the shear capacity measured as bond strength is considerably higher than the maximum shear stress. In reality, the shear capacity might be even higher. The result above is obtained without considering different stress rates. The stress rate used in the bond tests was slower than the stress rate under normal traffic load. If the bond tests have been run more rapidly, it is likely that higher bond strength values had been measured.

4. CRACK AND DAMAGE SURVEYS

4.1 Observed Cracks and Damages

Crack and damage surveys have been carried out at least once a year since the construction in October 1993. The observations are summarised in Table 3. With exception for the long crack of type III on test section No. 1 (Densiphalt), the observed cracking is not alarming. On the contrary, it may be concluded that the test sections have performed well during the three years they have been in use.

4.2 Discussion of Observed Cracks and Damages

Cracks of type I

Transverse cracks usually appear in concrete pavements with a long joint spacing. The cracks are caused by stresses due to restrained shrinkage and sometimes also due to restrained thermal movements. Here, transverse cracks arose on the 150 m long continuous Densiphalt section. Shrinkage cracks are not considered to appear in this kind of pavement, since Densiphalt and similar materials are said to have a high strain capacity and low shrinkage. The shrinkage of the material in question might have been unusually high and the strain capacity might have been unusually low, but no tests or other observations support these hypotheses. The development of the crack width will influence the future behaviour of the test section. If the crack width remains small, neither durability nor riding comfort will be reduced. Repeated crack surveys show that the number of transverse cracks is increasing, but the widths seem to be of the same, harmless magnitude.

Damage type II

The presence of palm sized spots without Densit mortar has not been explained. A possible explanation is that the mortar leaked through the open asphalt into a permeable base. Another explanation is that leaves prevented the mortar from penetrating the open asphalt. (The defoliation was intensive during the placement operations). In both cases, only a very thin mortar layer would be established on top of the open asphalt. Such a layer would be worn out rapidly after a number of heavy truck passages. Since only a small number of such spots have been observed, they will hardly influence the pavement behaviour. It would also be possible to repair these spots by pouring new Densit mortar into the exposed open asphalt. After three years of trafficking, nothing indicates that these spots are causing any problems.

Crack type III

The 10 m long crack close to the shoulder on the Densiphalt test section is the most serious damage at the site. It was observed after the most severe winter in five years. It can be compared with a 40 m long crack (crack width about 10 mm) observed simultaneously and located outside the test sections, 30 m from the crack in question. The 40 m long crack arose in a new asphalt pavement. These two longitudinal cracks have such a magnitude that the corresponding pavement sections ought to be strengthened. If the cracks only are sealed, the cracks are likely to reopen after the next strong winter. It must, however, be emphasised that the Densiphalt overlay was placed in order to enhance wear resistance, and not load-carrying capacity.

Damage type IV

The damages at the transverse joints are the most frequent distress along the test sections (Figure 3). These local damages arose possibly already during the sawing operations. It is also possible that the joint edges were visibly intact after sawing, but that micro cracking was introduced by the sawing. Soon after traffic opening, parts of the joint edges were successively broken. The primary causes are likely to be too early sawing and insufficiently careful sawing. The damages are more frequent on test section No. 4 that was last to be placed. Consequently, this section was youngest at both joint sawing and traffic opening. Furthermore, test section No. 4 was placed by plain concrete. The absence of steel fibres might be a cause of the worse performance. It may be added that the joints are straight on test section No. 4, but more irregular on test section No. 3. That might also be a result of the fibres.

The damaged joint edges make the pavements look ugly, but will hardly influence the pavement behaviour. The extent of this distress has not increased considerably since October 1993.

Crack type V

The corner cracks have arisen on test section No. 2 with joint spacing 3.5×3.75 m. The crack width is, however, very small. The small crack width was not measured due to intensive traffic, but can be supported by the following observations. At a very good visibility (drying-up after slight rain) in October 1994, seven cracks were observed. At worse visibility (wet surface) in February 1995, only five cracks could be seen.

The cracks are probably caused by traffic stresses exceeding available strength. On test sections Nos. 3 and 4, with shorter joint spacing there are no corner cracks yet. Due to the short joint spacing, the traffic stresses are lower on these test sections. It is too early to estimate the degree of detrimental effects of the corner cracks on the pavement behaviour. The rather long distance between the corner and the corner cracks and the presence of steel fibres are, however, two beneficial factors. The corner crack separates a triangle from the rectangular pavement part between two transverse joints. The size of the triangle is about half the size of the pavement panels on test sections Nos. 3 and 4. Consequently, the triangle might behave satisfactory as a separate unit. Besides, steel fibres will limit the crack width and keep the pavement parts together.

Damage of type VI

After three winters, the surface had been superficially worn. The wear was concentrated to the inner part of the traffic lane and mainly to the test sections with steel fibre reinforced overlays. It is not clear, why the wear was concentrated to the steel fibre reinforced overlays. A less degree of homogeneity as a result of worse consolidation is one possible explanation. There was no obvious concentration of wear to the wheel tracks, on the contrary, the wear was rather evenly distributed. The wear of aggregates and surrounding cement paste had similar magnitude.

5. RECOMMENDATIONS

The following recommendations are based on the field tests discussed in this report, on stress calculations, and on American experiences on thin whitetoppings. They are grouped under the following headlines:

1. Assessment of the old asphalt road prior to intervention
2. Design of whitetoppings
3. Construction
4. Overlays of open asphalt filled with high strength mortar

5.1 Assessment of the Old Asphalt Road prior to Intervention

1. *The load carrying capacity of the old asphalt road and the thickness of the asphalt pavement have to be determined before intervention. If the load carrying capacity is too low, there is risk that the overlaid pavement will crack, probably during the frost heave period. A thin concrete overlay is used to increase the rutting resistance, and not to increase the load carrying capacity.*
2. *For 70 mm thick concrete overlays, the thickness of the asphalt layer (or asphalt layers) after milling should not be less than 150 mm, if it cannot be shown that a minor thickness is sufficient due to stiffer subgrades, smaller loads, or other joint spacings. Note that the milling depth usually exceeds 20 mm.*

5.2 Design of Whitetoppings

1. *The designer of whitetoppings ought to consider traffic, thermal, and shrinkage stresses as well as thermal buckling, load transfer efficiency at joints and wear resistance when choosing overlay thickness, joint spacing, strength, steel fibre content, and aggregate quality.*
2. *For medium traffic volumes, the thickness of a high strength concrete overlay should not be less than 70 mm. The ratio between joint spacing and overlay thickness should not exceed 20. High strength concrete with a cube strength of 80 MPa will normally give the desired wear resistance. An addition of steel fibres does not seem to be necessary, but will increase the safety against uncontrolled cracking that may occur if there are spots of poor bond between concrete and asphalt along the pavement. For high traffic volumes, the overlay thickness ought to be increased to 80 or 90 mm.*

5.3 Construction

The following items have to be carried out:

1. *The road or one or two traffic lanes of the road is closed to traffic.*
2. *If the asphalt layer locally is too thin, the thickness should be adjusted through placement of new asphalt on these spots.*
3. *Entire asphalt surface should be milled prior to overlay placement in order to promote a good bond between asphalt and concrete.*
4. *The milled asphalt surface ought to be cleaned before overlay placement. If oil or any other pollution has arisen through careless execution or unfortunate circumstances, the surface has to be cleaned carefully by application of absorbing materials and water cleaning.*
5. *The concrete overlay ought to be placed by a slip form paver.*
6. *The overlay should be cured properly starting immediately after paving. Special care has to be taken under hot, arid, and windy weather conditions.*
7. *The joint sawing procedure has to be started at right time.*
8. *Thin saw blades ought to be used. It is important to have a sufficient amount of sawing equipment not disregarding the need of substitute saws.*
9. *The sawing depth ought to be 1/3 of the overlay thickness. The sawing width ought not to be more than 3 mm.*
10. *Asphalt adjustments should be made against shoulders to make the connections even.*
11. *The repaired road may be opened to traffic when the overlay has gained sufficient strength.*

The choice of curing procedure is strongly dependent on the weather conditions. In cold weather (0 to 10°C) like in current field tests, spraying with a curing compound might be appropriate. In warmer weather, the application of a curing compound should be followed by protection with plastic sheeting or water curing.

Two problems are to determine right time for joint sawing and suitable time for traffic opening. The determination may be facilitated through measurements of overlay temperature and subsequent temperature based computations of strength development. According to current field tests, the sawing procedure should hardly start before the compressive strength has reached 10 to 15 MPa. According to Petersson & Sundbom, the pavement could be opened to traffic when the compressive strength has reached 25 to 26 MPa (6). Such a low strength value has, however, to be considered in the overlay design procedure. This early loading may decide the overlay thickness.

If a thin saw blade and a short joint spacing are used, the joints do not need to be sealed.

5.4 Overlays of Open Asphalt Filled with High Strength Mortar

The experience with overlays of open asphalt filled with high strength cementitious mortar is too limited to give any specific recommendations. The following items have, however, to be taken into consideration when designing and planning new overlays composed of Densiphalt or similar products:

1. *The thickness of the asphalt layer (or asphalt layers) after milling ought to be more than 120 mm.*
2. *For medium traffic volumes, the thickness of the overlay should not be less than 40 mm.*
3. *The open asphalt should not be exposed to intensive rain, since water-filled voids may interfere with the mortar, that is subsequently poured into the voids.*

4. *The high strength mortar is poured out on the open asphalt. It is important to ensure that the mortar fills the voids completely.*

6. NEEDS OF FURTHER RESEARCH

The following items concerning whitetoppings need further research:

- Studies on the behaviour of whitetoppings and development of design methods for whitetoppings with short joint spacings. The research should be focused on pavements with joint spacings approaching the diameter of the loaded area (joint spacings less than 1.5 m).
- Development of methods to choose an optimum time for joint sawing.
- Development of concrete saws with thin saw blades.
- Theoretical and field studies on fast-track concrete overlays on asphalt pavements. Development of methods for easy handling and reuse of materials to insulate the young concrete overlay with.
- Development of design methods for Densiphalt and other similar overlays.

7. CONCLUDING REMARKS

Based on the performance and evaluation of four Swedish test sections with whitetoppings, the following conclusions can be drawn:

- A good bond between concrete overlay and underlying asphalt is mandatory for the composite action and, hence, the optimum pavement performance.
- The concrete overlay has to be placed on an asphalt layer of decent thickness in order to ensure sufficient load carrying capacity.
- Milling of the asphalt surface prior to the concrete placement will promote a good bond between concrete and asphalt. The measured bond was sufficiently high to ensure the beneficial composite action.
- In very thin whitetoppings (70 mm or less), the joint spacing should not exceed 1.5 m in order to prevent the pavement from corner cracks. Tiny corner cracks have been observed in the test section with a joint spacing of 3.5 m, but not in test sections with shorter joint spacings.
- The absence of steel fibres did not change the good performance of the thin overlay with a short joint spacing. Contrary, the evenness was better on the test section without steel fibres, see (9). Consequently, steel fibres do not seem to be necessary in overlays with short joint spacings.
- Choice of sawing time and sawing blades are crucial for the joint edge performance. Early sawing and thick saw blades may lead to damages on the joint edges. Too late sawing may lead to an uncontrolled cracking.
- Hybrids between concrete and asphalt seem to be interesting overlay alternatives. An open asphalt filled with a high strength cementitious mortar (Densiphalt) has been used with promising result at the site, even though the bond between Densiphalt and asphalt was less than expected (around 0.1 MPa). A 40 mm thick Densiphalt overlay cannot, however, strengthen an old flexible pavement with insufficient load carrying capacity considerably.

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TABLE 1 - Important data of the test sections

TABLE 2 - Bond strength tests

TABLE 3 - Observed cracks and damages

TABLE 1 Important Data of the Test Sections

Characteristics	Data for Test Sections Nos.			
	1	2	3	4
Length (m)	150	150	65	83
Overlay type	Densiphalt	Concrete K80 ^a	Concrete K80	Concrete K80
Fibre content (kg/m ³)	-	40 - 50	40 - 50	-
Intended overlay thickness (mm)	40	> 70	> 70	> 70
Intended asphalt thickness (mm)	> 120	> 150	> 150	> 150
Joint spacing (m)	-	3.5×3.75	1.25×1.25	1.25×1.25
Age at traffic opening (days)	13	6	5	5
Average air temperature between placement and traffic opening (°C)	>5	5	5	5

^a K80 = Swedish concrete grade having a 28 day compressive cube strength of 80 MPa.

TABLE 2 Bond Strength Tests

Core No.	Test section No.	Description	Core diameter (mm)	Core length (mm)	Overlay length (mm)	Failure stress (MPa)	Failure mode
B1	1	Densiphalt	71.7	93	41	0.17	interface
B2	1		72.3	90	40	0.04	interface
A1	2	SFRC	68.8	96	58	0.24	asphalt
A2	2		68.8	108	84	0.36	asphalt
A3	2		68.8	102	79	0.60	interface
C1	3	SFRC	68.8	117	65	0.23	asphalt
C2	3		68.8	138	81	0.13	asphalt
C3	3		68.8	117	60	0.30	asphalt
D1	4	Plain concrete	68.6	128	74	0 ^a	asphalt
D2	4		68.7	112	82	0.20	asphalt
D3	4		68.8	84	73	0.41	asphalt
D4	4		68.8	92	79	0.41	asphalt
D5	4		68.8	120	72	0.29	asphalt

^a Failure occurred already during attachment of the core in the testing machine.

TABLE 3 Observed Cracks and Damages

Type of crack or damage	Date of first observation	Test section No.	Location within pavement	Extent	Description
I	Oct. 27, 1993	1	Arbitrary	4 cracks over entire lane + 5 to 10 cracks limited to edge part of the lane	Transverse crack
II	Nov. 15, 1993	1	Arbitrary	5 to 10 palm sized spots	Exposed asphalt, the Densit mortar is missing
III	March 15, 1994	1	1 m from shoulder	2 cracks, L=10 and 2 m, resp.	Longitudinal crack, max w = 20 mm
IV	Oct. 27, 1993	2 - 4	At transverse joints	About every second transverse joint	Half-moon shaped damages, L×w×d=50×15×10 mm
V	Oct. 9, 1994	2	Corner	7 cracks, L=1.5 m	Diagonal cracks cutting off a triangular corner 1×1×1.5 m, w = 0.1 mm ?
VI	Aug. 8, 1996	2 - 3	Centre part of lane	Larger part of test sections	Wear

Note. L = crack length, w = crack width, d = crack depth.

LIST OF FIGURE CAPTIONS

FIGURE 1 - Outline of test sections Nos. 3 and 4.

FIGURE 2 - A high strength mortar is poured out on the newly placed open asphalt.

FIGURE 3 - Example of half-moon shaped damages at transverse joints.

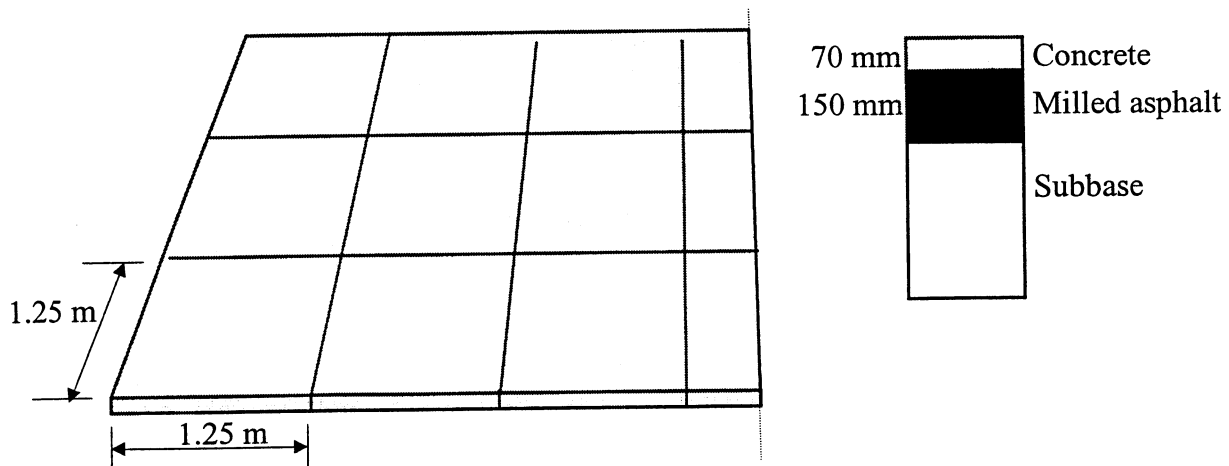


FIGURE 1 - Outline of test sections Nos. 3 and 4.

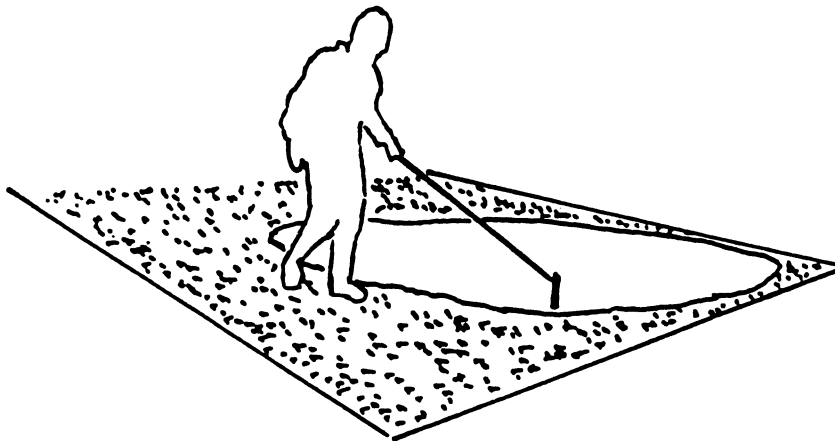


FIGURE 2 - A high strength mortar is poured out on the newly placed open asphalt.

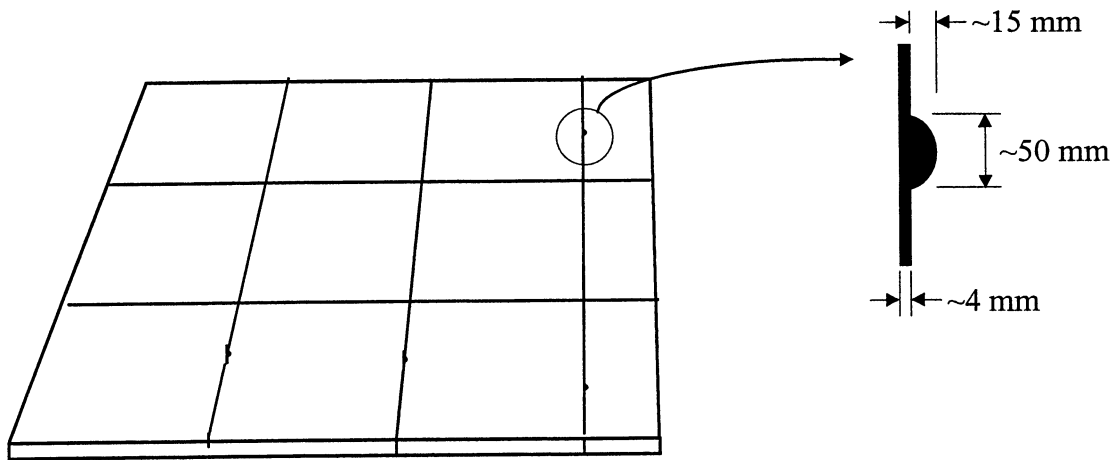


FIGURE 3 - Example of half-moon shaped damages at transverse joints.

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