FINNISH ROAD ADMINISTRATION

Matti Huhtala, Jari Pihlajamäki, Janne Sikiö

HVS-NORDIC
Reseach Report No 3

Tests 01-02 base course tests, 03-05 loading mode tests in Otaniemi

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Technical Research Centre of Finland

Swedish National Road Administration

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ABSTRACT

Finland and Sweden have jointly invested in HVS-NORDIC, which is a mobile Accelerated Loading Test facility with full temperature control. A six-year period of research collaboration between 1997 and 2003 has been agreed. The activity of the first period in Finland is described in the Periodic Report. This research report covers tests 01-02 (base course tests) and 03-05 (loading mode tests), which also served as HVS-NORDIC performance and warranty tests.

The research idea of tests 01-02 was to test two kinds of base course material. The research idea was to test the effect of loading mode (bi-directional, single-wheel) and compare the results to those obtained from loading with different modes (uni-directional, single-wheel -test 04) and (bi-directional, dual-wheel -test 05).

The tested pavement structures were instrumented. The standard instrumentation consists of strain sensors in bituminous layers, stress sensors in unbound granular layers and in subgrade, total deflection sensors and temperature sensors. Initial response measurements with different parameters were made, and measurements during testing as well. These are important data for analysis and modelling.

The first result was that the pavements lasted for much longer than expected, when the water table level was more than 2 m below the road surface. Deterioration was mainly rutting in dry circumstances, only a few cracks were found. Raising the water-table level up to the bottom of the base layer had a dramatic effect on rutting and cracking. In dry circumstances, low quality base course material behaved well because it is tough. In wet circumstances the situation is probably unlike. Under high wheel load (80 kN) high quality base course material crushed much. Under normal wheel load (60 kN) the situation could be otherwise.

Initial strain measurements revealed that the strains in thin bituminous pavement not only increase with load, but may also decrease. The reason is that the length of the contact interface between the tyre and pavement increases as the load increases at the same tyre inflation pressure. No difference was found in rutting when the loading mode was uni- or bi-directional. Consequently, the bi-directional loading mode can be used to give HVS double efficiency compared to the uni-directional loading mode.

The personnel of VTT learned a lot of APT during the first tests in Finland. Despite the teething problems in the beginning, the HVS facility performed reasonably well during the first tests; the unscheduled down time was less than 20% of the total time.

FOREWORD

Finland and Sweden have jointly invested in HVS-NORDIC, which is a mobile Accelerated Loading Test facility with full temperature control. The HVS-NORDIC research is a part of the Finnish National Road Structures Research Programme, TPPT, which is funded by the Finnish Road Administration (Finnra). The HVS-NORDIC research is being carried out in co-operation with the Swedish National Road and Transport Research Institute (VTI), and the Swedish National Road Administration (SNRA). HVS-Nordic has been in Finland in 1997-1998, 1998-2000 in Sweden and from November 2000 in Finland.

The research is reported first in Weekly Reports (main results by e-mail), then in Periodic Reports, which include preliminary results after each period, and later in Research Reports and Conference Papers. This Research Report covers tests 01-02 (base course tests) and 03-05 (loading mode tests), which also served as HVS-NORDIC performance and warranty tests.

The members of the HVS team at the Technical Research Centre of Finland (VTT) are Matti Huhtala, Jari Pihlajamaki, Pekka Halonen, Veikko Miettinen, Janne Sikiö, Risto Alkio, Juhani Idman and Heikki Kangas.

The report has been written by Matti Huhtala, Jari Pihlajamaki and Janne Sikio, and was compiled according to the guidelines given by Aarno Valkeisenmaki (Finnish Road Enterprise) and Kari Lehtonen (Finnra).

ABBREVIATIONS

APT	Accelerated Pavement Testing
HVS	Heavy Vehicle Simulator
Finnra	Finnish Road Administration
VTT	Technical Research Centre of Finland
SNRA	Swedish National Road Administration
VTI	Swedish Road and Transport Research Institute
NVF	Nordic Road Association
OECD	Organisation for Economic Co-operation and Development
STRO	Scandinavian Tyre and Rim Organisation
TPPT	Finnish National Road Structures Research Programme
LCPC	Laboratoire des Ponts et Chaussées, France
CAPTIF	APT facility in New Zealand
FWD	Falling Weight Deflectometer
AC	Asphalt Concrete
ACB	Asphalt Concrete in base course
ACBi	Asphalt Concrete in binder course
SMA	Split mastic asphalt
AC20	Asphalt Concrete, maximum grain size 20 mm
B80	Bitumen, penetration 80
Q1	Quality class of base layer material
Q2	“
Q3	“

CONTENTS

1 INTRODUCTION

1.1 Background

An accelerated pavement-testing facility, Heavy Vehicle Simulator (HVS), was bought from South Africa in 1996. It is owned on a 50/50 basis by Sweden (VTI) and Finland (VTT and Finnra). A six-year period of research collaboration between 1997 and 2003 has been agreed /1/. The co-operation is organized on three levels with Steering, Programme and Operative groups.

The HVS-NORDIC was used in Finland during 1997—1998, and in Sweden 1998—2000. The activity of the first period in Finland is described in the Periodic Report /2/. This report is a research report, which is a conventional (scientific) research report. Preferably, this includes several similar tests, base course tests and loading mode tests. It is the compilation of similar kinds of tests. In principle, the report may include tests made both in Finland and in Sweden.

1.2 Technical specifications of the machine

The machine is called HVS-NORDIC. The HVS Mark IV is a mobile full-scale accelerated pavement-testing facility (Figure 1). It can be run over a short distance by itself at walking speed, in practice only at the same test site. It can be moved as a semi-trailer over longer distances to other test areas or to VTI in Sweden. Since it has steering wheels, it can turn even relatively sharp corners, in spite of its long length. Its highway speed is 50 km/h, and special permits are needed.

Figure 1. HVS-NORDIC

The HVS-NORDIC has a heating/cooling system and thus temperature can be held constant. The air temperature inside the heating/cooling box is controlled in order to keep the pavement temperature constant. The standard temperature of bituminous layers is selected to be +10°C. The HVS can be run by diesel engine or by electric motor. The diesel engine also provides power for the heating/cooling system, which means it is not dependent on external power.

The main technical characteristics are: length 23 m, width 3.5 m, height 4.2 m and weight 46 t. The loading wheels are dual or single; the standard dual wheel type is 295/80R22.5 and the wide-base wheel type is 425/65R22.5. Loading can be uni- or bi-directional, and lateral movement is 0.75 m (excluding the width of the loading wheel). The wheel load is from 20 kN to 110 kN (corresponding axle loads 40...220 kN) at speeds up to 12 km/h. The number of loadings is 25 000 in 24 hours (including daily maintenance).

The HVS-NORDIC is the only mobile APT facility in Europe and the only mobile APT facility in the world with full temperature control. The loading of the HVS-NORDIC can be varied dynamically +/- 20 % within the loaded area during a single pass. As far as we know, there are no possibilities for dynamic loading in any other APT facility.

1.3 Test site

The test site is located a few hundred metres from the office of VTT's personnel. The site includes two test pits. Test structures 01 and 02 were constructed in test pit no. 1, which is made of concrete and where the water table can be controlled. Tests 03 - 05 were built in a test pit that is excavated mainly in rock (rock pool) and it has no water table regulation. Their dimensions are roughly: length is 36 m, depth is 2.5 m and its width 4 m at the top and 3 m at the bottom.

1.4 Instrumentation

The standard instrumentation consists of strain sensors in bituminous layers, stress sensors in unbound granular layers and in subgrade, total deflection sensors and temperature sensors. The position of the sensors in the standard instrumentation is shown in Figure 2. Possible variation in instrumentation of test is described under each test.

From the year 2000 onwards, also strains in unbound granular layers and subgrade are measured (Emu-coils) and surface deflection is also measured with accelerometers.

The instrumentation for response measurements is mainly based on the experience that has been gained at the Virttaa test site during the last 15 years.

The basic instrumentation is based on strain gauges, which are installed at the bottom and on the surface of the bituminous layers. VTT uses retrofit strain gauges.

Five longitudinal and five transverse gauges are installed at the bottom of the bituminous layers and three longitudinal and three transverse gauges on the surface of the asphalt layer.

Stress in unbound layers was measured with stress sensors that have been bought from the University of Nottingham. These were installed three at three levels in base-course, sub-base and in subgrade.

Deflection under the moving wheel load was measured by a deflection rod that was anchored in the rock.

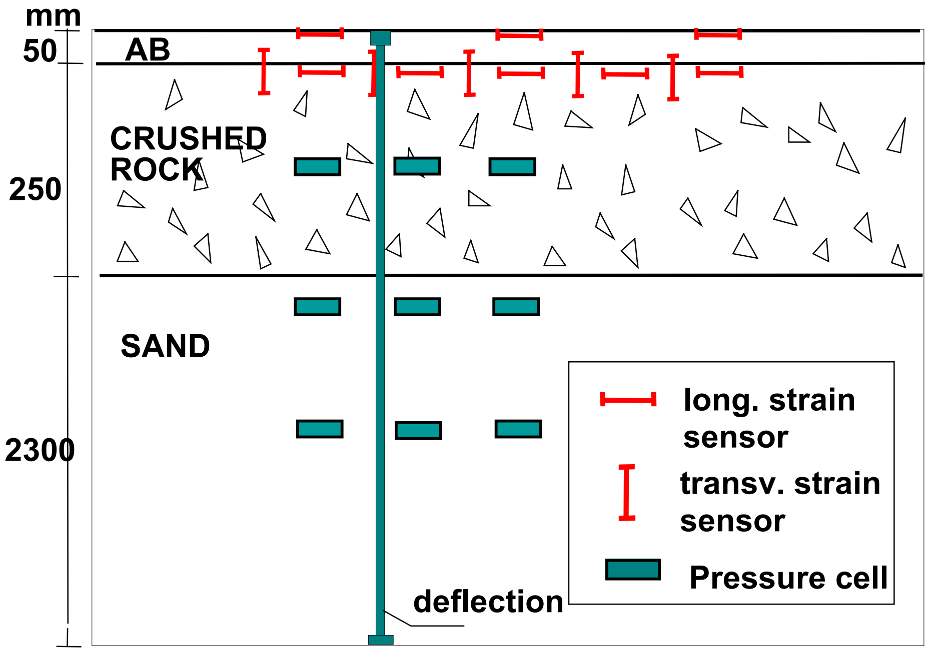

Figure 2. The standard instrumentation.

The temperature of bituminous layers is measured at three depths, on the surface, in the middle and at the bottom of the bound layer.

1.5 Standard test procedure

1.5.1 Preloading

The test was started first with a pre-run in order to relax possible residual stresses and to cause some post-compaction. This was done with a small wheel load, 30 kN single-wheel load, 20 000 passes.

1.5.2 Test parameters

The standard temperature of bituminous materials for the tests has been selected at +10°C, which is close to the weighted mean temperature in Southern Finland and Sweden. The standard dual-wheel load is 60 kN but sometimes different loads are used depending on the nature of the test. The speed is 12 km/h but in the first test it was 10 km/h in order to see if the facility works properly.

1.5.3 Response measurements

After the pre-run, the initial response measurements are made. These include a considerable number of response measurements: strain, stress and deflection measurements at different wheel loads, tyre inflation pressures, lateral positions and pavement temperatures. In most cases, both wide-base tyres (super single) and twin tyres are used.

The response measurements are made at five temperatures of 0°, +5°, +10°, +15°C and +20°C. The change in the temperature by five degrees takes about half a day.

Six wheel loads from 30 kN to 80 kN are normally used and five tyre inflation pressures from 500 to 900 kPa are used for each wheel load.

The speeds are 1, 4, 7, 10 and 12 km/h. Two transverse positions are used for measurements with dual wheels: sensors under the tyres and between the tyres. For the single wheel, only one transverse position was used: sensors under the tyre. The effects of transverse position are measured with one axle load and one tyre inflation pressure.

After the initial stage of the test, the pavement responses due to a moving wheel load are measured during testing twice a week or more often. The measurements are made only with constant test parameters.

Typical signals of different responses, stress, strain and deflection due to a moving wheel are presented in Figure 3 - Figure 6.

The soil pressure cell measures the vertical component of stress even though horizontal component may have some minor indirect effect. The signal is symmetrical, as can be seen in Figure 3.

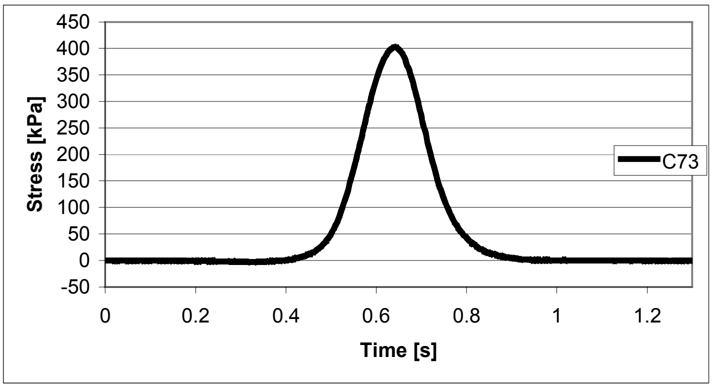

Figure 3. Typical signal of the soil pressure cell.

As the wheel of a vehicle approaches a sensor, the bituminous layer bends and causes compression at the bottom of the bituminous layer, as can be seen in Figure 4 (strain is negative). As the wheel is on the sensor, the strain at the bottom of the bituminous layer measures the tensile or positive value. Typically, compression is about one third of tension in the initial stage, depending on the pavement temperature AC modulus, etc. After the wheel has passed the sensor, there is once again compression, which is, however, smaller than the first compressive strain. In certain cases it may be nearly non-existent. However, the strain will always be on the same level (or zero) after the wheel has passed the sensor.

The basic shape of the longitudinal signal is the same even if a wheel does not overpass the sensor; the peaks are only smaller.

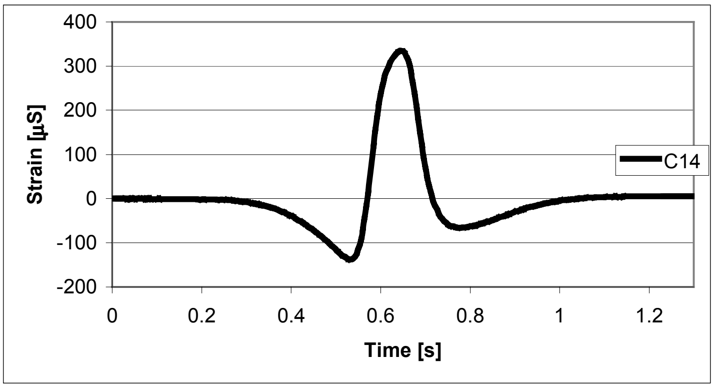

Figure 4. Typical signal of the longitudinal strain gauge at the bottom of the bituminous layers.

As the wheel passes the transverse sensor, the strain at the bottom is tensile all the time, as can be seen in Figure 5. The signal is not symmetrical, but owing to the viscoelastic nature of the bituminous materials the stresses relax at a speed that is dependent on the temperature and the properties of the bituminous layers. The relaxation may take a long time, and if the next wheel comes before relaxation has occurred, there may be an accumulation of strains. This phenomenon has been covered in References 8 and 9.

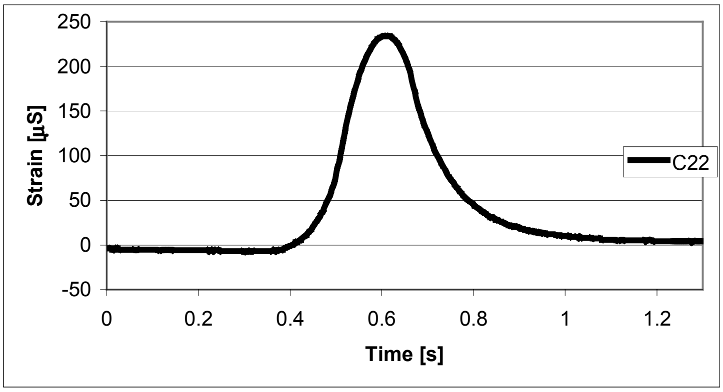

Figure 5. Typical signal of the transverse strain gauge at the bottom of the bituminous layers.

Transverse strains are very sensitive to the lateral position of the wheel. They are tensile under the tyre but immediately outside the tyre imprint they are compressive. If there are several axles, the shape of signal may be very complicated because of accumulation, as shown in Reference 8.

All these signals are from the same situation. The peak of the transverse signal is smaller (230 µS) than that of the longitudinal signal (330 µS). This relation depends on the thickness of the bituminous layer and the temperature; often transverse signals are greater.

Deflection of the whole pavement is measured with a sensor attached to a steel rod going through the pavement down to the bottom of the test pit. The signal is not symmetrical, as can be seen in Figure 6.

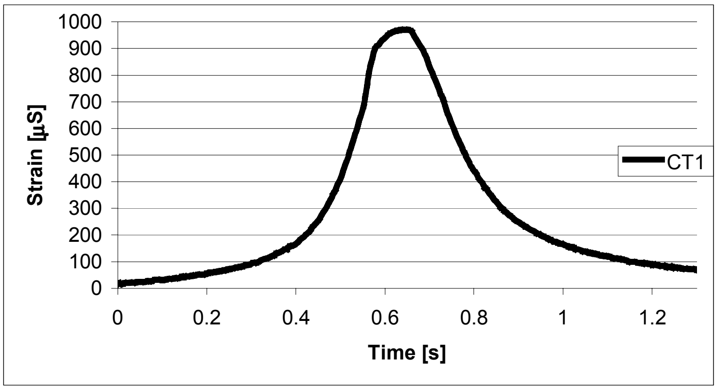

Figure 6. Typical signal of the deflection strain gauge.

The zero of the signals is taken about 0.6 seconds before the loading and thus no permanent strains or stresses are measured. The peaks are measured automatically and only a few signals are viewed.

However, seeing a signal may reveal something about the condition of the sensor or pavement. For instance, cracking close to the sensor can be found by comparing the actual strain signal at the bottom of the bituminous layers to that measured in the initial stage. Generally, comparing signals in different stages of the test is an important part of analysis.

1.5.4 Other measurements and observations

Rutting is observed visually every morning and if needed measured with a laser profilograph. In the beginning of the test and at least once a week, transverse profiles at five locations are measured with a laser profilograph.

Cracks are drawn on paper with the aid of 1*1 m grid that is divided into 100*100 mm squares. The cracks are digitized to a database with coordinates of the start point and end point, and where a crack is not straight, also coordinates of the middle point.

1.5.5 Post-mortem sampling and testing

When the test is completed, samples are taken from both the unbound and bound materials. Both the loaded and unloaded areas are sampled. The samples are tested in the laboratory according to the "basic minimum" program /10/.

2 BASE COURSE TESTS (TESTS 01 AND 02)

2.1 Aim

The research idea of tests 01-02 was to test two kinds of base course material. These pavements would also serve as reference pavements for the HVS-NORDIC research programme. These tests were also the first tests, and thus they served as a performance test for the facility.

Test pavement 01 consisted of 50 mm asphalt concrete with bitumen of penetration 80, a 250 mm unbound base layer and subgrade of fine sand (2300 mm). The second test structure was also constructed in test pit no. 1. The only difference was that the quality of the base course material was better. Both base course materials are reference materials in the Finnish national research programme (TPPT). The structures of tests 01 and 02 are presented in Figure 7 and Figure 8.

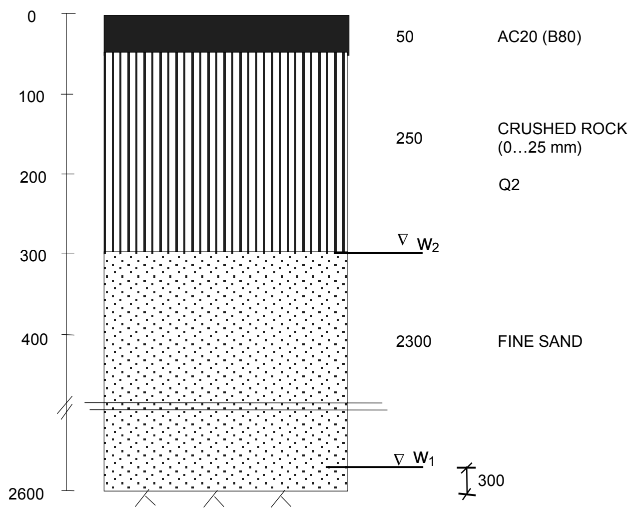

Figure 7. The structure of test 01.

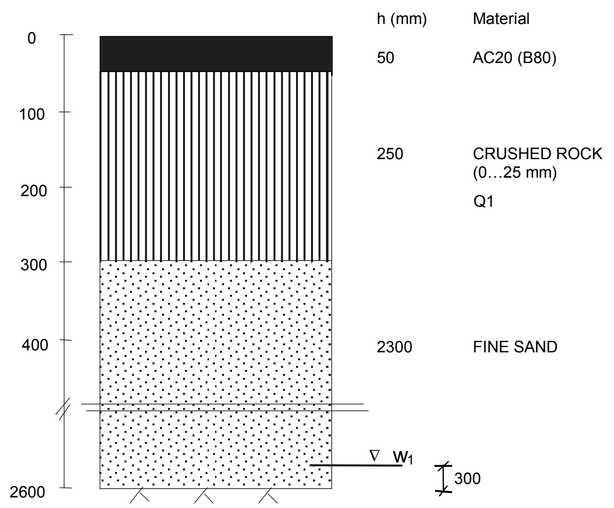

Figure 8. The structure of test 02.

2.2 Materials

The test structures were built in a concrete pool. In tests 01 and 02 there was 2300 mm of subgrade on the concrete. This layer was made of fine sand. In both tests 01 and 02 above the sand there was 250 mm of base course, which was made of crushed rock 0...25 mm. On these unbound layers there was one bound layer (50 mm), AC (B80).

The results of the base course material tests and the composition of rock of tests 01 and 02 are shown in Table 1 and Table 2.

Table 1. Base course material (Lusi-aggregate) in test 01 and results of material tests.

Rock type	Micagneiss
Minerals	Mica 39%, Quartz 30%, Cordiorite 12%, Plagioclase 10%.
Specific gravity	2.80 g/cm^3
Los Angeles	27 %
Max dry unit weight	22.3 kN/m^3
Optimum water content	7.2 %
Nordic Abrasion Value (Ball Mill Value)	21 %

Table 2. Base course material (Teisko-aggregate) in test 02 and results of material tests.

Rock type	Granodiorite
Minerals	Quartz 26%, Potash Feldspar 17%, Plagiollase 48%.
Specific gravity	2.67 g/ cm^3
Los Angeles	24 %
Max dry unit weight	21.2 kN/m^3
Optimum water content	4.4 %
Swedish Impact Value (Modified)	27 %
Abrasion Value	1.8 cm^3
Tröger Value	12 cm^3
Point Load Strength Index (IS_{50})	10.4 Mpa
Nordic Abrasion Value (Ball Mill Value)	11 %

Teisko-aggregate is from Central Finland near Tampere. It has been the reference material in the ASTO research programme and is the good-quality reference material in the TPPT research programme. Lusi-aggregate is from the South-eastern part of Finland and is the low-quality reference material from TPPT. Both are crushed rock and one reason for their selection was that they will be available for a reasonably long time.

Teisko-aggregate is typical Finnish granitic rock consisting of hard but brittle minerals. Its Los Angeles value is reasonably good (24 %) as is its Nordic Abrasion Value, which is 11 %. The Nordic Abrasion Value (Ball Mill Value) is a CEN test for determining the stud-resistance properties of aggregates used in bituminous wearing courses. It is often used for base course materials in Finland, too.

Lusi-aggregate has plenty of mica, which is a soft but tough mineral. Its toughness can be seen in the Los Angeles value (27 %), which is reasonably good for a base course material, or it resists impacts well. The Nordic Abrasion Value of 21 % shows that it is not good against grinding. As aggregate crushes during compaction or later because of traffic loadings, mica concentrates in the fine aggregate and the aggregate becomes more susceptible to water.

The grading curves of both the base course materials are shown in Figure 9. They are nearly identical.

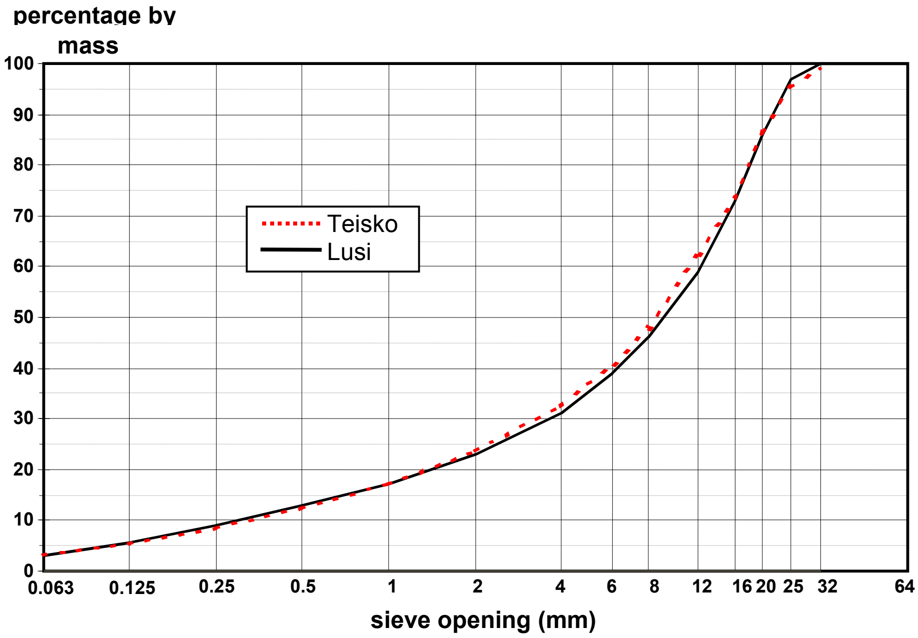

Figure 9. The grading curves of the crushed rock (Teisko-aggregate, base course material in test 02 and Lusi-aggregate, base course material in test 01 (and 03-05)).

The grading curves of the subgrade material are shown in Figure 10. The result of the proctor compaction test is shown in Figure 11.

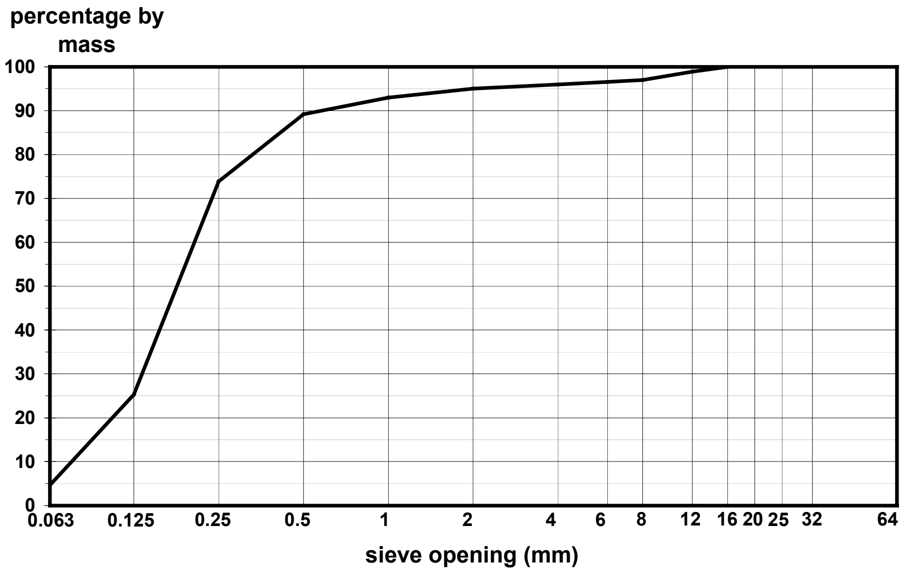

Figure 10. The grading curve of the sand, subgrade material in test 01 - 05.

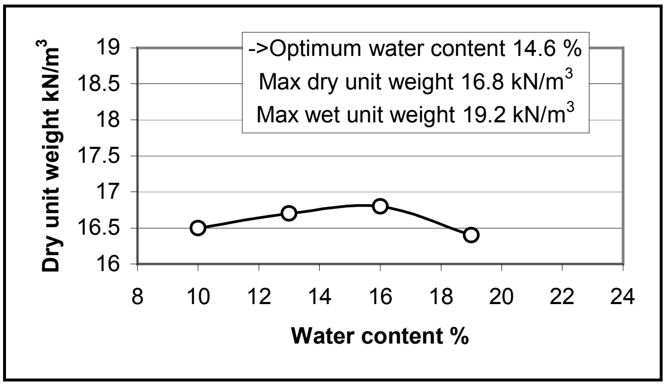

Figure 11. Result of proctor compaction test from subgrade material.

2.3 Construction

The test construction was made with usual road-building machines. Test pavements 01 and 02 were constructed in test pit no. 1, which is made of concrete. The water-table level was about 2.3 m from the road surface in both tests but was raised at the end of test 01 to 0.3 m from the surface.

The actual thickness of the bituminous layers is not always exactly similar to the designed structures. This may make comparison of the tests more complicated. The actual thickness of the bound layers of tests 01 and 02 are shown in Figure 12 and Figure 13. The thickness was determined from core samples.

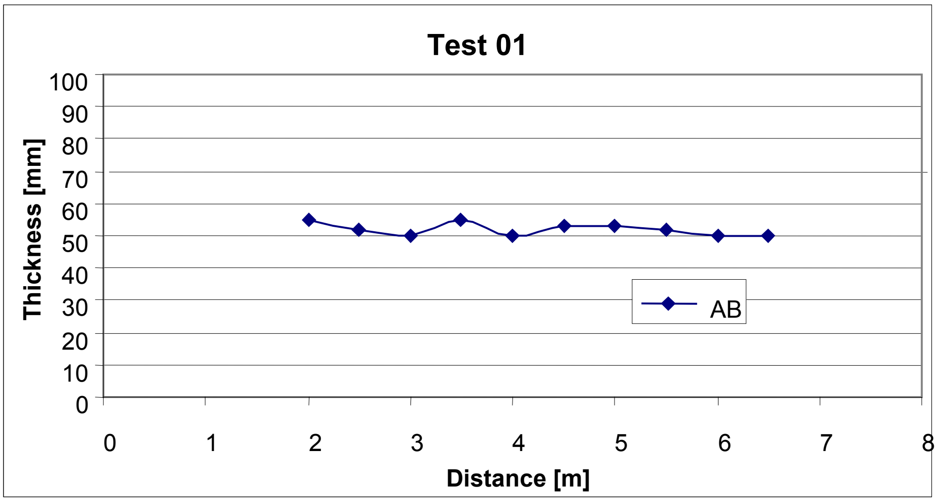

Figure 12. The actual thickness of the bituminous layer in test 01.

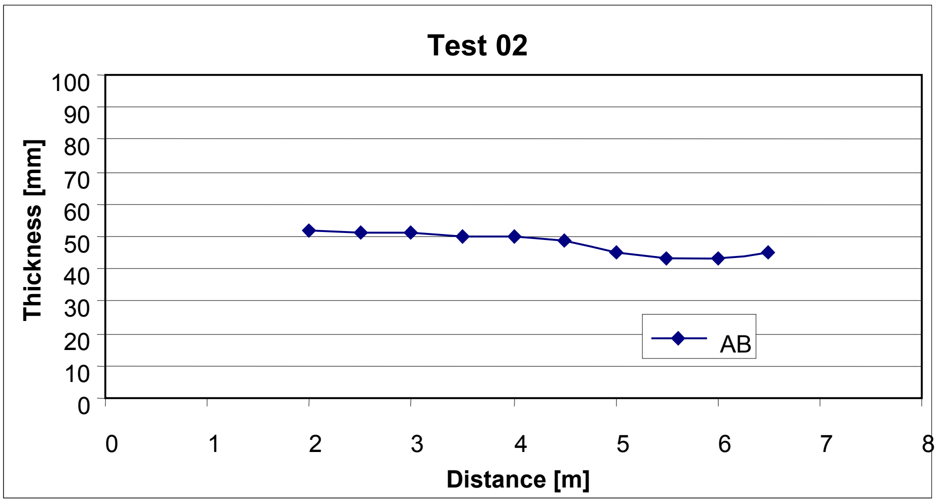

Figure 13. The actual thickness of the bound layer in test 02.

The actual thickness of the layers in tests 01-02 is shown in Table 3.

Table 3. Structures in tests 01 and 02.

Test	Structure
Test 01	52 mm AC20(B80) 250 mm crushed rock Q1 2300 mm fine sand
Test 02	48 mm AC20(B80) 250 mm crushed rock Q1 2300 mm fine sand

Quality control tests were carried out on the subgrade, for instance, water volumeter tests. The results of the water volumeter tests are given in Table 4.

Table 4. Results of water volumeter tests on the subgrade in tests 01 and 02.

Surface of the subgrade +16.10 Optimum water content 14.6 % Max dry unit weight 16.8 $\gamma_d kN/m^3$															
Ground level	15.2	15.2	15.2	15.4	15.4	15.4	15.6	15.6	15.6	15.9	15.9	15.9	16.1	16.1	16.1
Water content %	5.5	8.5	6.8	7.0	6.8	6.6	7.1	7.0	6.8	5.9	6.5	6.3	7.0	7.2	6.8
Dry unit weight $\gamma_d kN/m^3$	15.0	15.5	14.9	15.0	15.1	14.9	15.1	16.0	15.0	15.1	15.1	15.2	15.1	15.5	15.1
Compaction index %	89.3	92.3	88.7	89.3	89.9	88.7	89.9	95.2	89.3	89.9	89.9	90.5	89.9	92.3	89.9

The falling weight deflectometer measurement was conducted in two phases. The first measurement was taken from the surface of the base course, before asphalt. The second measurement was taken from the surface of the asphalt layer. Deflections were measured at three load levels. These falling weight deflectometer measurements cannot be done under HVS machine.

The FWD deflections of the test constructions 01 and 02 are shown in Figure 14.

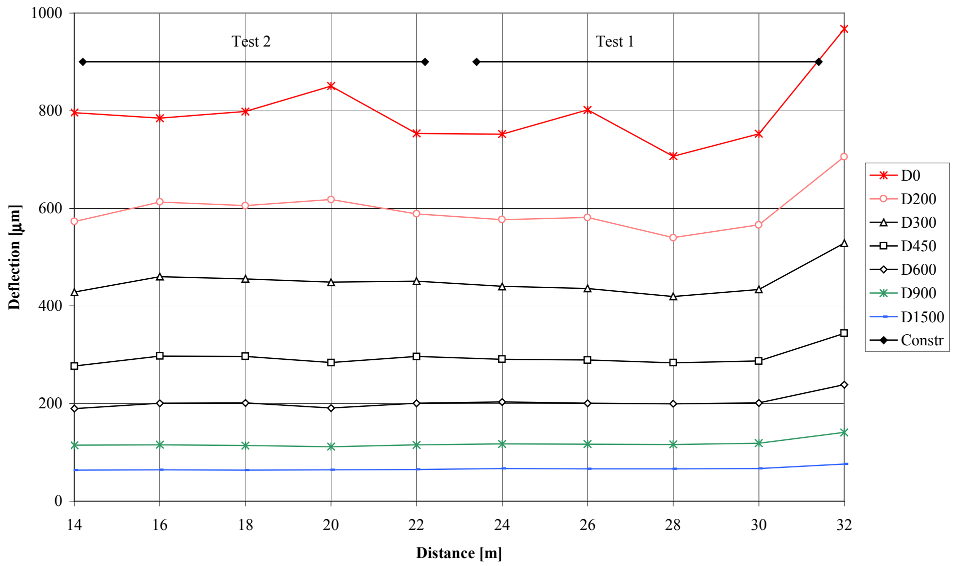

Figure 14. Deflections from FWD measurements (50 kN) in tests 01-02. Pavement temperature was 4°C.

The deflection bowl of the FWD measurements in tests 01 and 02 is shown in Figure 15. The structure of test 02 seems to be a little bit stiffer, especially the upper part.

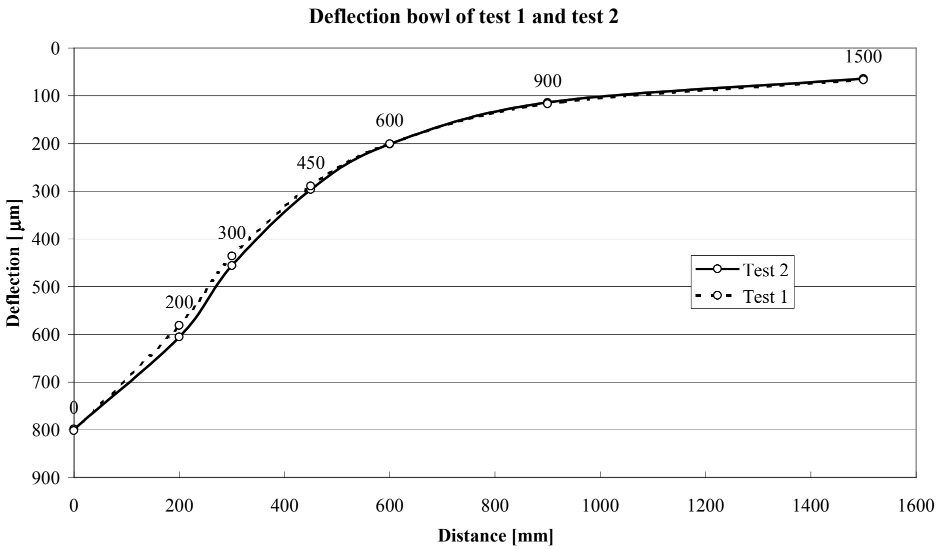

Figure 15. Deflection bowl of FWD measurements (50 kN) in tests 01-02. Pavement temperature was 7°C.

The back-calculated moduli of pavement materials are presented in Table 5. The back calculation was made with the Modcomp 3 program, which is based on the linear elastic multilayer theory.

The first row of each case is the result of the free modulus calculation. The results are unreasonable because the thickness of the bituminous layer was only 50 mm. For this reason, the calculations were made with a fixed bound material modulus. The fixed values were chosen based on the stiffness modulus tested in the laboratory and on the pavement temperature during measurements. With fixed values (*), the calculated moduli are more reasonable.

Table 5. The results of back calculation of FWD measurements (=fixed value).*

Test 01 (Resilient Modulus, Modcomp3)				
Distance (m)	AC (7 °C)	Base course	Subgrade	Bedrock
2	37300	29	204	*10000
	*8000	251	73	*10000
4	24500	82	98	*10000
	*8000	238	74	*10000
6	16900	96	96	*10000
	*8000	184	81	*10000
Test 02 (Resilient Modulus, Modcomp3)				
Distance (m)	AC (7 °C)	Base course	Subgrade	Bedrock
2	13700	161	83	*10000
	*8000	224	77	*10000
4	25000	139	89	*10000
	*8000	315	75	*10000
6	21300	129	89	*10000
	*8000	268	76	*10000

2.4 Test

In test 01 the loading mode was bi-directional, the dual-wheel load was 60 kN with the tyre inflation pressure at 800 kPa. In test 02 the loading mode was also bi-directional, and the other parameters were 80 kN and 800 kPa, similarly. In test 01 the speed of the loading tyre was 10 km/h and in test 02 the speed was 10-12 km/h. The test parameters of tests 01 and 02 are shown in Table 6.

Table 6. Test parameters in tests 01 and 02.

Test	Test parameters
Test 01	60 kN, 800 kPa, bi-directional, Dual, 10 °C, 10 km/h
Test 02	80 kN, 800 kPa, bi-directional, Dual, 10 °C, 10-12 km/h

2.5 Results

2.5.1 Rutting

The development of rutting in test 01 can be seen in Figure 16. Test structure 01 lasted for 1.5 million load repetitions without cracks, and only 25 mm rutting was found.

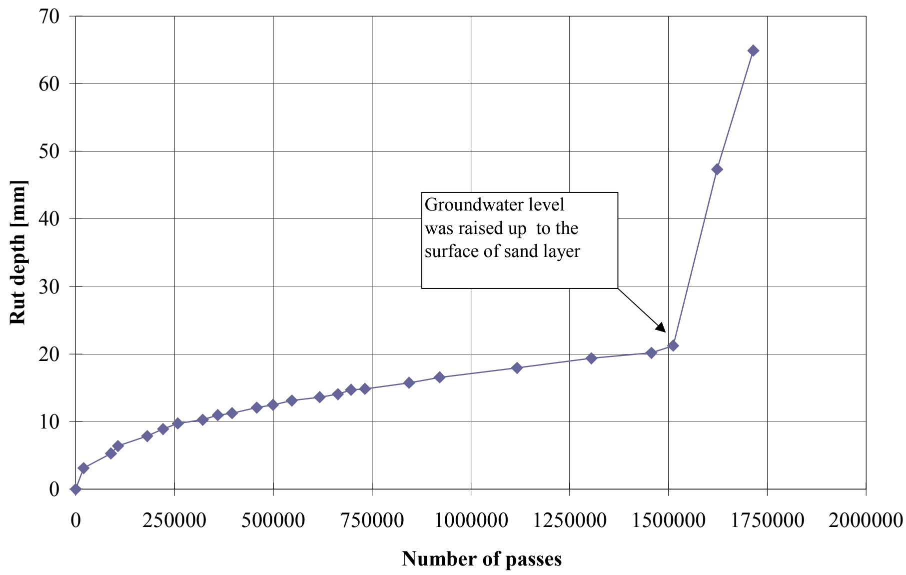

Figure 16. Rut depth vs. number of passes during test 01 (low-quality base, 60 kN dual-wheel load).

The water-table level was raised from the original level (2.3 m from the road surface) to 0.3 m from the road surface (at the bottom of the base course) in order to see the effect of moisture, and in order to accelerate the test after 1.5 million load repetitions. Only 0.2 million load repetitions were needed to increase rutting from 21 mm to 65 mm.

Since it was decided to carry out thawing tests during the first winter, there were only two weeks in which to load test 02. Therefore, it was decided to use 80 kN (instead of 60 kN) dual-wheel load after the initial response measurements. Owing to the lack of time, only 0.17 million load repetitions could be made.

In test 02 after 0.17 million passes, only 18 mm rutting could be seen, but no cracks. After these loadings, the test had to be stopped because thawing test constructions were made in the same test pool. The development of rutting in test 02 (as well as test 01) can be seen in Figure 17.

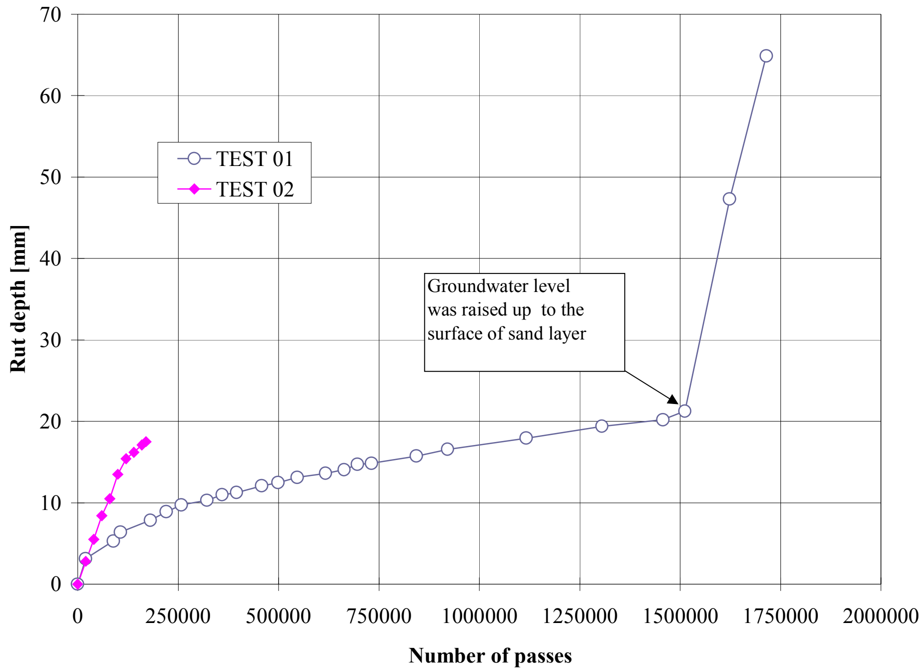

Figure 17. Rut depths vs. number of passes during test 01 (low-quality base, 60 kN dual-wheel load) and test 02 (high-quality base, 80 kN dual-wheel load).

2.5.2 Cracking

No cracks were found on the road surface before raising the water-table level up. With this water level, after 0.2 million load repetitions ten 0.4 m long transverse cracks were found on the road surface at the end of the test. In test 02 no cracks were found.

2.5.3 Deflection measurements

Deflections due to different wheel loads were measured during the test with a benkelman beam at three locations.

The results of the benkelman beam measurements in test 01 are shown in Figure 18.

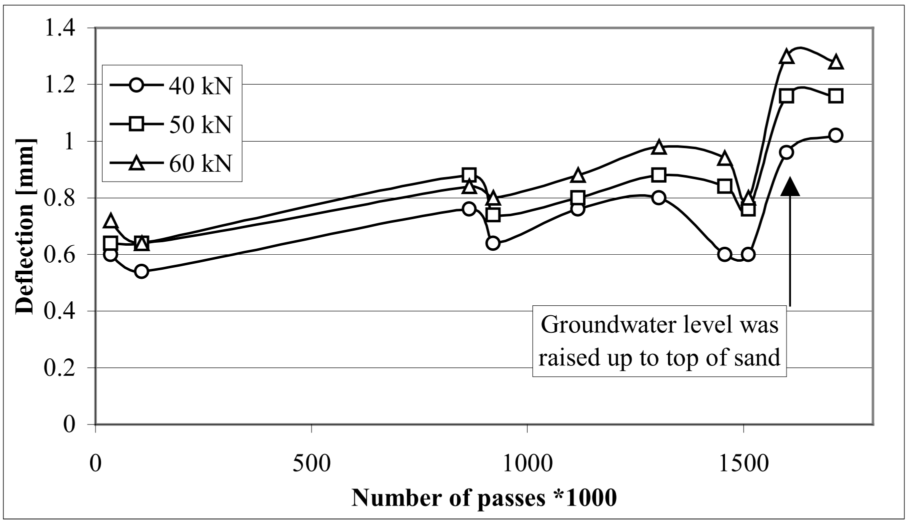

Figure 18. Benkelman beam measurements of test 01.

2.5.4 Response measurements

Initial response measurements were made after the pre-run before the actual test loading. A huge number of different parameter combinations were used in the measurements.

This chapter only deals with the typical behaviour of strain and stress in every measured layer vs. speed and temperature because these structures, 01-02, were alike. There was minor variation in the level of strain/stress in each structure but the behaviour in general was similar. The basic levels of the sensors are shown in Appendix 3 and 4.

Figure 19 shows how the longitudinal strain at the bottom of the bituminous layer changes versus the speed of the loading tyre. The effect of a low speed is significant, but when the speed increases the effect decreases.

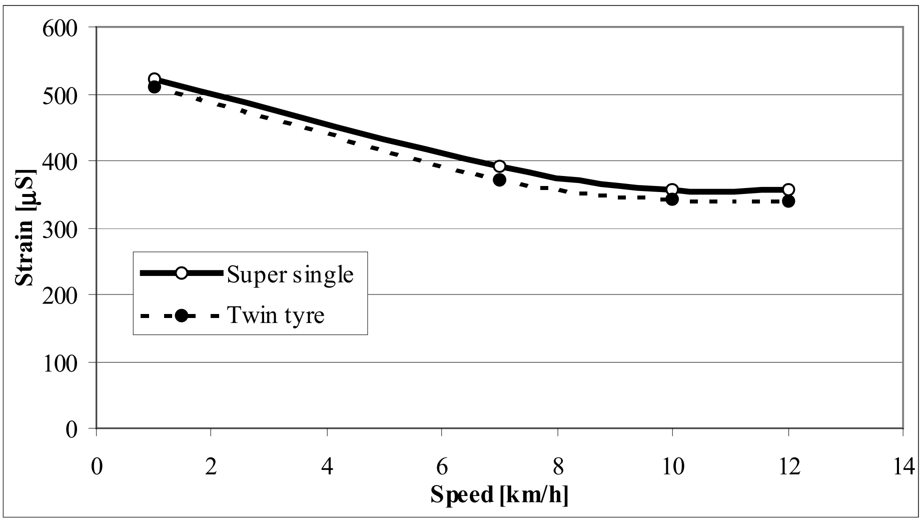

Figure 19. Typical behaviour of longitudinal strain at the bottom of bituminous layer vs. speed of loading tyre (test 01, sensor A12).

The effect of temperatures on the strain at the bottom of the bituminous layer is shown in Figure 20. The effect of pavement temperature on asphalt strains is enormous. When the temperature of the bituminous layers increases from 1°C to 15 °C the strain increase 75 % (from 210 μS to 360 μS).

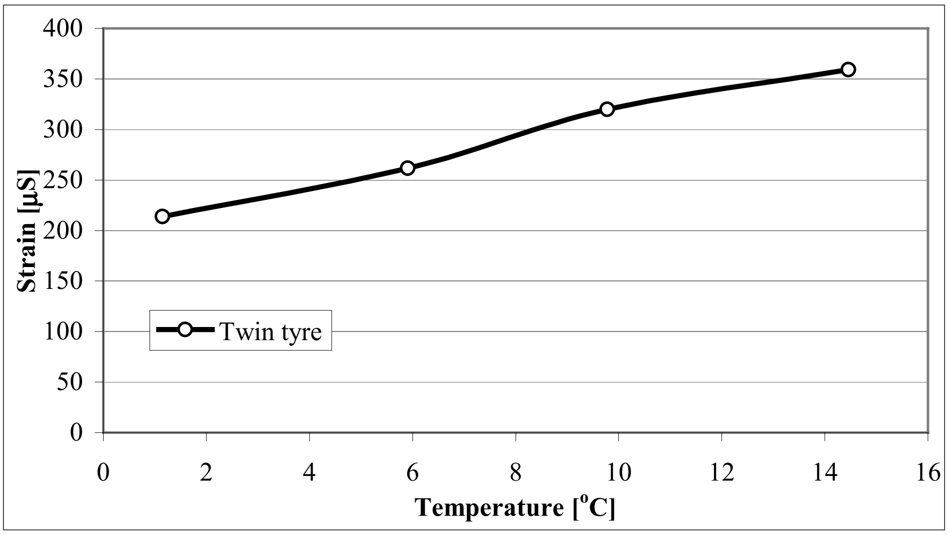

Figure 20. Typical behaviour of longitudinal strain at the bottom of bituminous layers vs. pavement temperature (test 02, sensor BB5).

Figure 21 show how the stress in the middle of the base course (test 01, sensor A72) changes versus the speed of the loading tyre. Strain caused by the super single tyre is 35 % higher than that caused by the twin tyre.

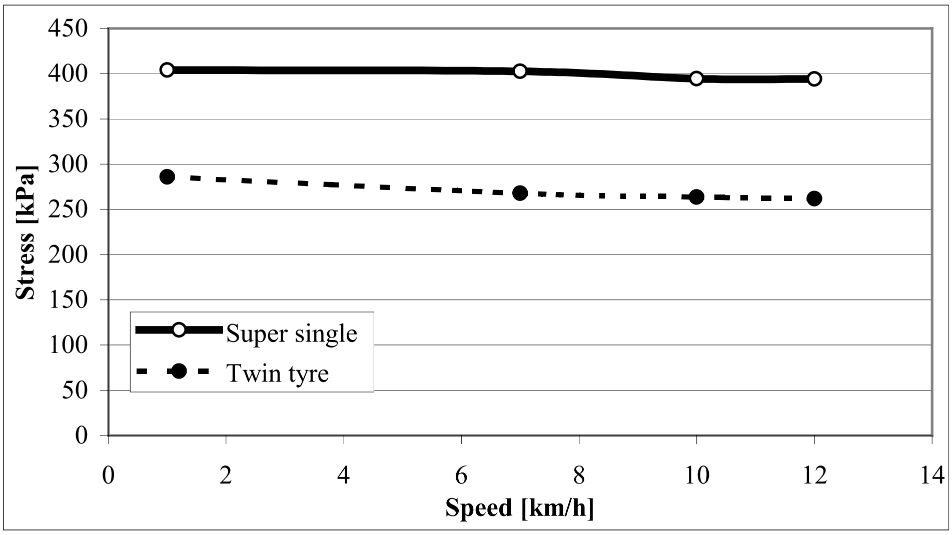

Figure 21. Typical behaviour of stress in the middle of base course (175 mm from the road surface) vs. speed of loading tyre (test 01, sensor A72).

The effect of temperatures on stress in the middle of the base course (test 01 and sensor A72) is shown in Figure 22. When the pavement temperature increases from 10 °C to 20 °C the strain increases 20 %.

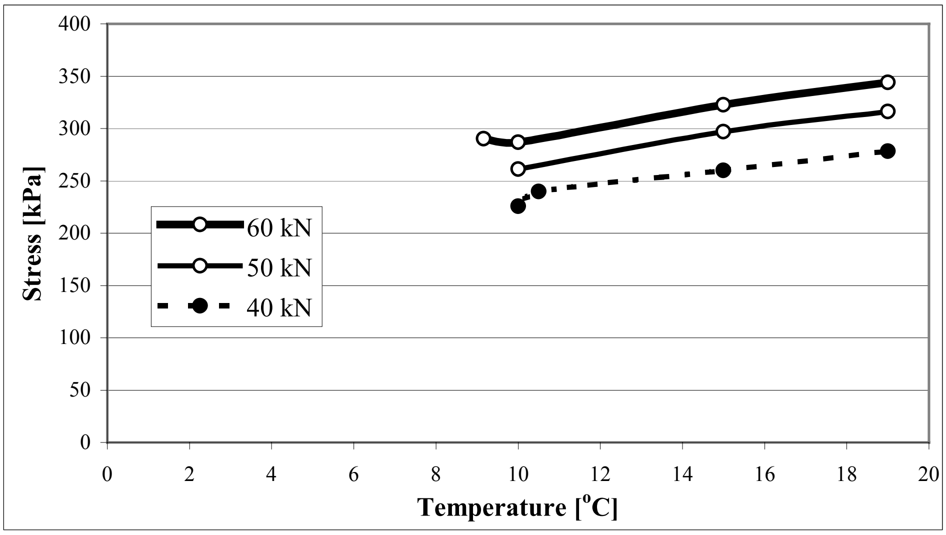

Figure 22. Behaviour of stress in the middle of base course (175 mm from the road surface) vs. pavement temperature (test 01, sensor A72).

Figure 23 shows how the stress on the top of the subgrade changes versus speed of the loading tyre. The effect of temperatures on stress on the top of the subgrade is shown in Figure 24.

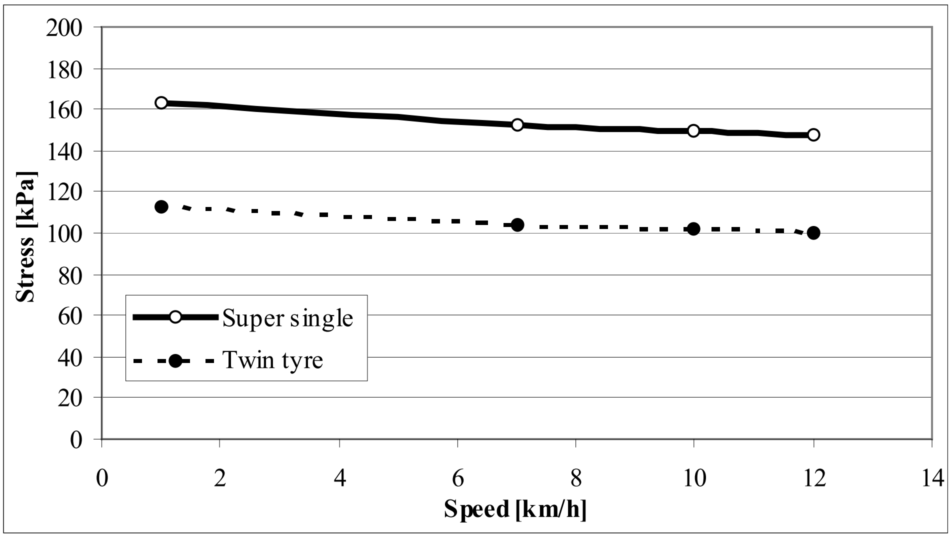

Figure 23. Behaviour of stress on the top of the subgrade (300 mm from the road surface) vs. speed of loading tyre (test 01, sensor A91).

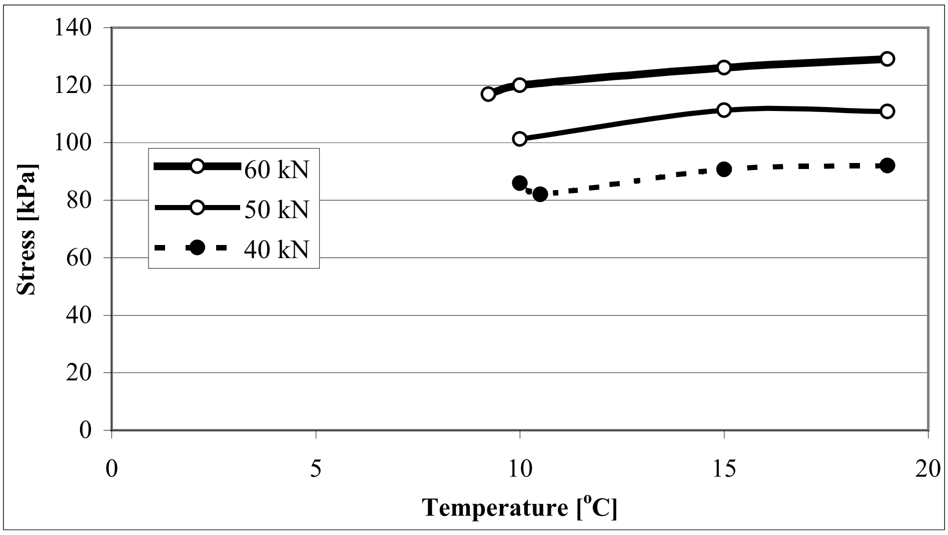

Figure 24. Behaviour of stress on the top of the subgrade (300 mm from the road surface) vs. pavement temperature (test 01, sensor A91).

Figure 25 shows how the stress in the subgrade (600 mm from the road surface) depends on the speed of the loading tyre. The effect of pavement temperatures on the stress in the subgrade is shown in Figure 26.

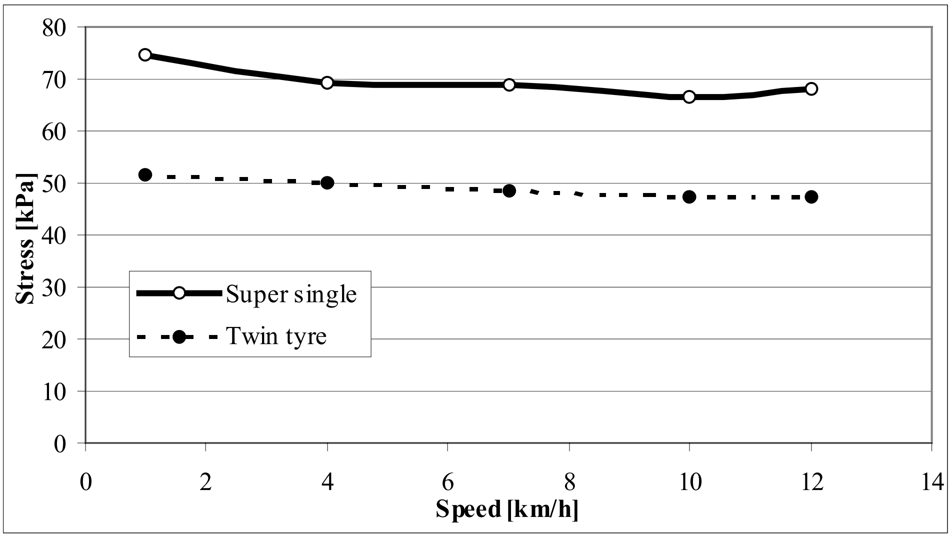

Figure 25. The behaviour of stress in the subgrade (600 mm from the road surface) vs. speed of loading tyre (test 02, sensor B98).

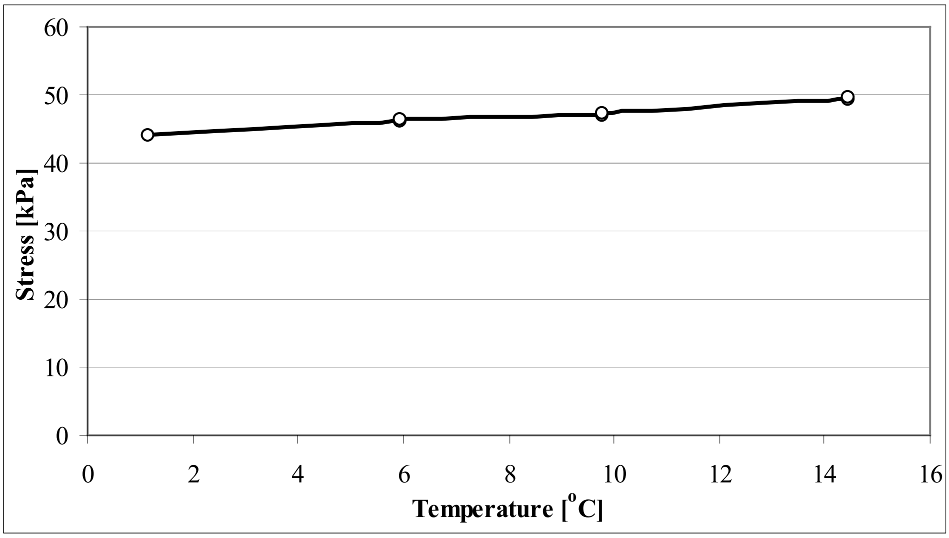

Figure 26. The behaviour of stress in the subgrade (600 mm from the road surface) vs. pavement temperature (test 02, sensor B98).

The relationship between wheel load and longitudinal strain at the bottom of the bituminous layers in test 01 and 02 is shown in Figure 27 (sensor B14) and Figure 28 (sensor A12). The measurements show that when the wheel load is low (30 - 40 kN), the strain caused by the super single tyre is about 20 % greater than strain caused by the twin tyre. When the wheel load increases, the difference between strains falls.

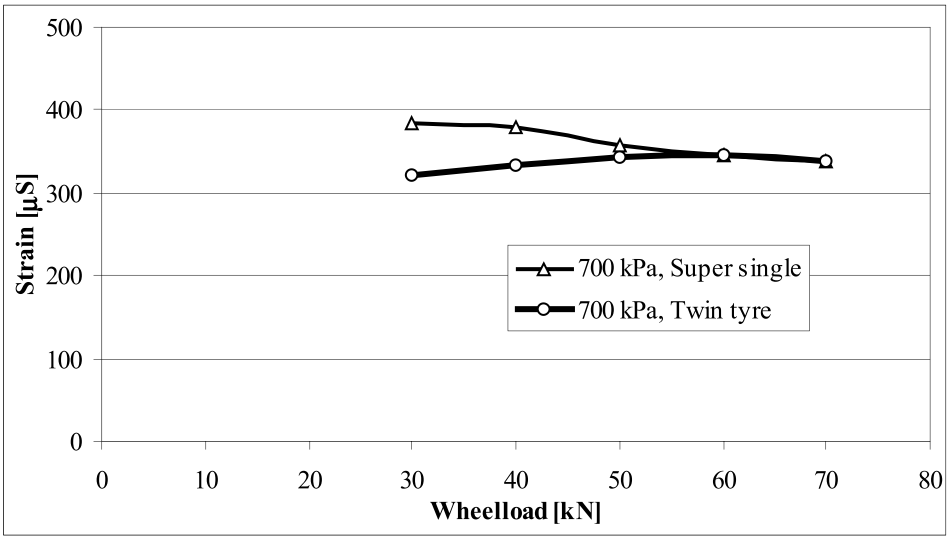

Figure 27. Measured longitudinal strain at the bottom of the bituminous layer vs. wheel load with twin and super single tyres (test 01, sensor A12).

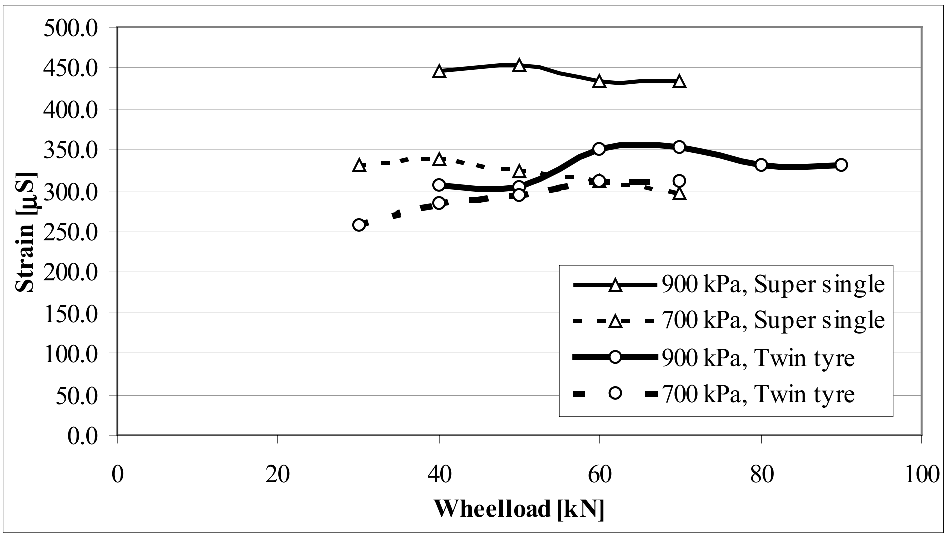

Figure 28. Measured longitudinal strain at the bottom of the bituminous layer vs. wheel load with twin and super single tyres at the same tyre pressure (test 02, sensor B14).

Figure 29 (sensor BB1) shows how the tyre pressure effects strain at the bottom of the bituminous layer with different wheel loads in test 02. When the tyre pressure is increased from 500 kPa to 900 kPa, the strain at the bottom of the bituminous layer increases from 290 µS to 380 µS (50 kN). The increase seems to be linear. Figure 30 (sensor B71) shows how the wheel load affects stress in the middle of the base course with different tyre pressures.

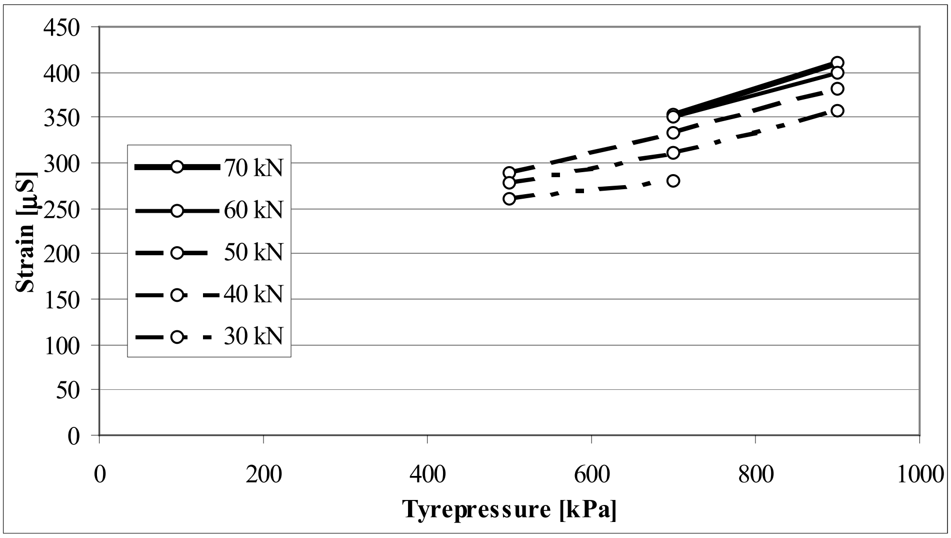

Figure 29. Measured longitudinal strain at the bottom of the bituminous layer vs. tyre pressure with twin tyre (test 02, sensor BB1).

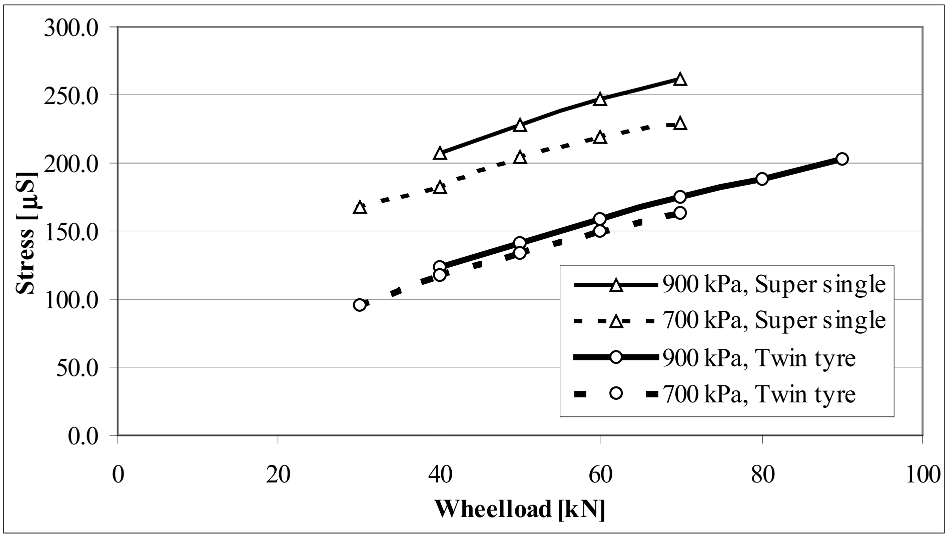

Figure 30. Measured stress in the middle of the base course vs. wheel load with twin and super single tyres (test 02, sensor B71).

Figure 31 shows the effect of tyre pressure with different wheel loads on stress in the middle of the base course in test 02.

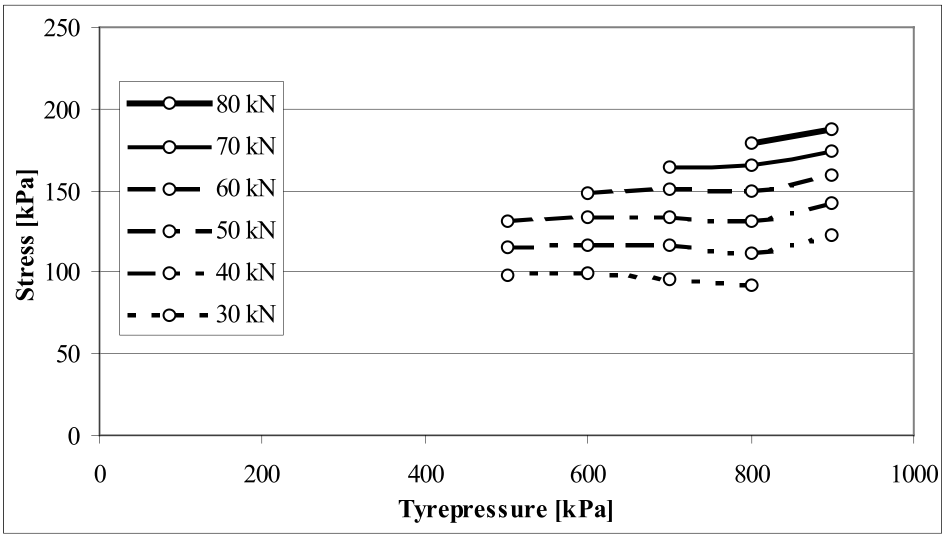

Figure 31. Measured stress in the middle of the base course vs. tyre pressure with twin tyre (test 02, sensor B71).

Figure 32 shows the difference between the super single and twin tyre effect on stress on the surface of the subgrade. The example is from test 02 and the sensor is B91.

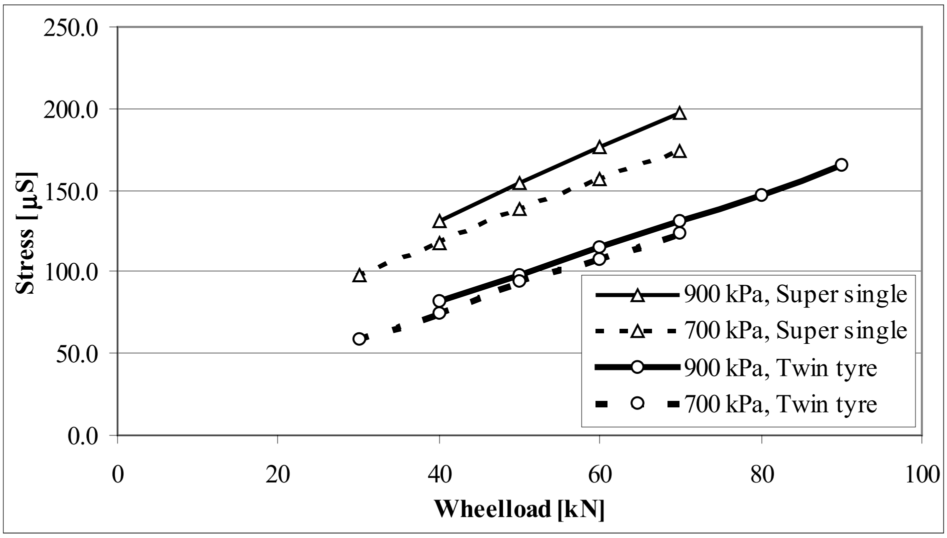

Figure 32. Measured stress on the top of the subgrade vs. wheel load with twin tyre and super single tyre (test 02, sensor B91).

Figure 33 shows the effect of tyre pressure with different wheel loads on stress on the top of the subgrade in test 02.

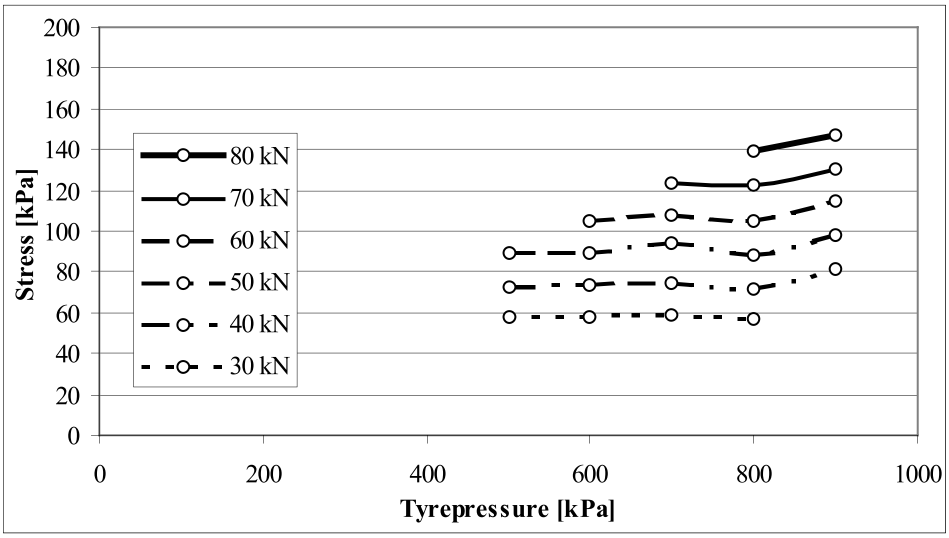

Figure 33. Measured stress on the top of the subgrade vs. tyre pressure of twin tyre (test 02, sensor B91).

Figure 34 shows the difference between the super single and twin tyre effect on stress on the surface of the subgrade. The example is from test 02 and the sensor is B98.

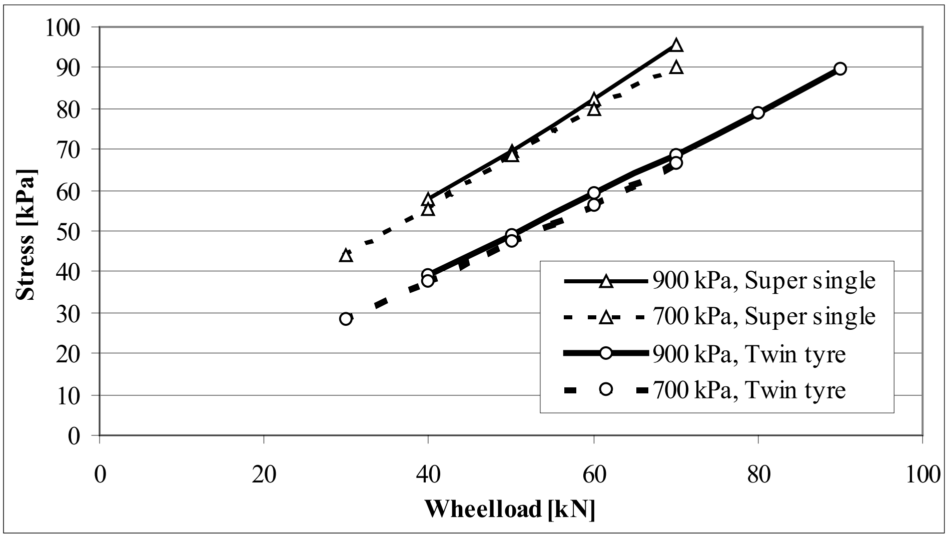

Figure 34. Measured stress in the subgrade (600 mm from the road surface) vs. wheel load with twin tyre and super single tyre (test 02, sensor B98).

Figure 35 shows that the effect of tyre pressure on stress in the subgrade is only nominal.

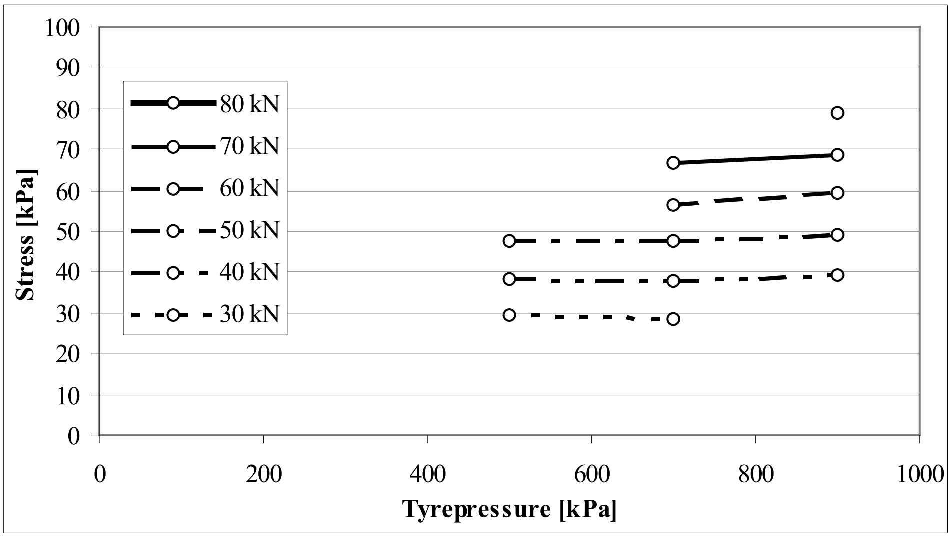

Figure 35. Measured stress in the subgrade (600 mm from the road surface) vs. tyre pressure of twin tyre (test 02, sensor B98).

The pavement response due to the HVS wheel load was measured during testing with test parameters only. The different responses, strain, stress and deflection vs. number of passes are presented in Figure 36 to Figure 41. These figures are from the sensors in test 01. The measurements were done with the twin tyre (60kN, 800 kPa, 10km/h).

Longitudinal strain on the surface of the bituminous layer is shown in Figure 36. There has been only a slight change in the level of strain.

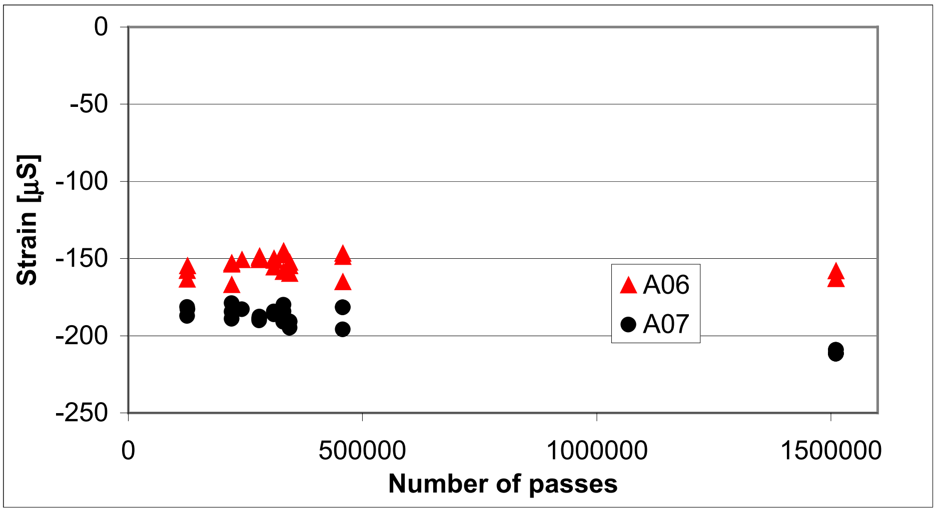

Figure 36 Development of longitudinal strain on the surface of the bituminous layer vs. number of passes in test 01.

The development of longitudinal strain at the bottom of the bituminous layer is shown in Figure 37. According to both sensors (A11, A12), the strain has decreased during the loading.

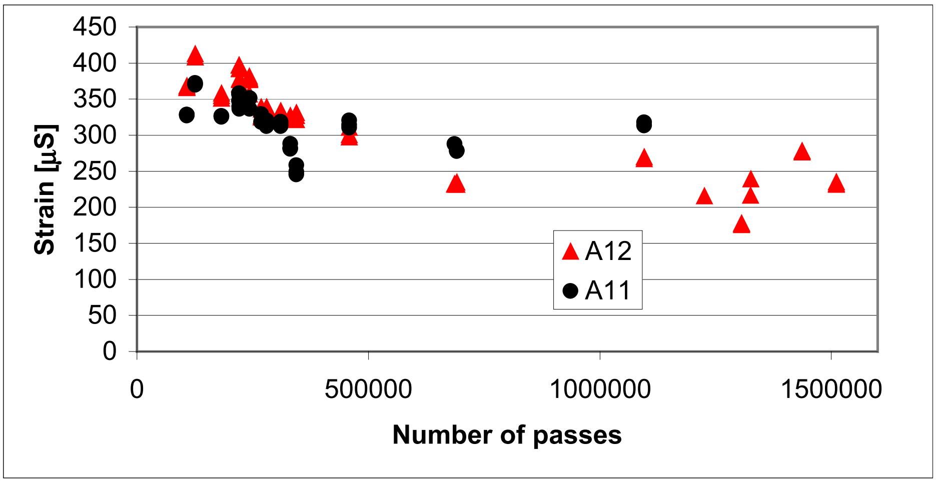

Figure 37. Development of longitudinal strain at the bottom of the bituminous layer vs. number of passes in test 01.

The development of transverse strain at the bottom of the bituminous layer is shown in Figure 38. The changes in strain are very small.

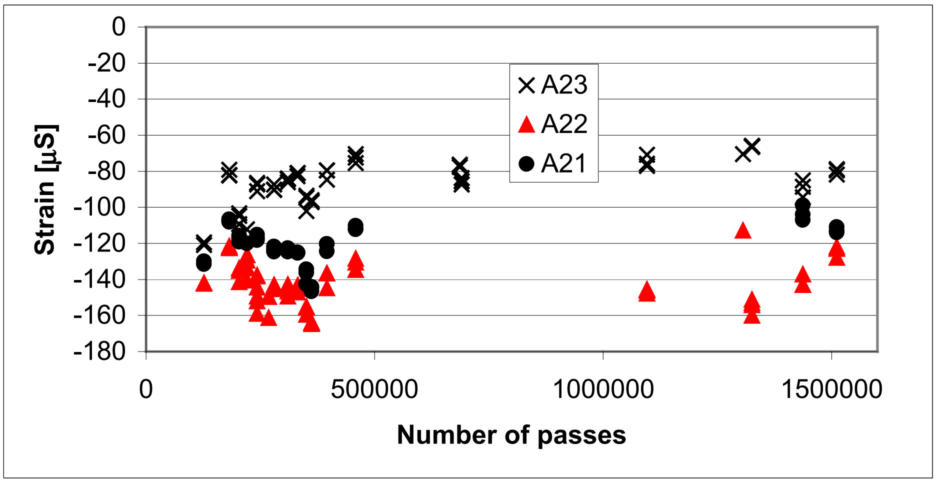

Figure 38. Development of transverse strain at the bottom of the bituminous layer vs. number of passes in test 01.

The development of stress in the middle of the base course in test 01 is shown in Figure 39. According to the sensors A72 and A73, the stress has increased during the loading. There could be some fault in the functioning of sensor A71. This can be detected from signal forms.

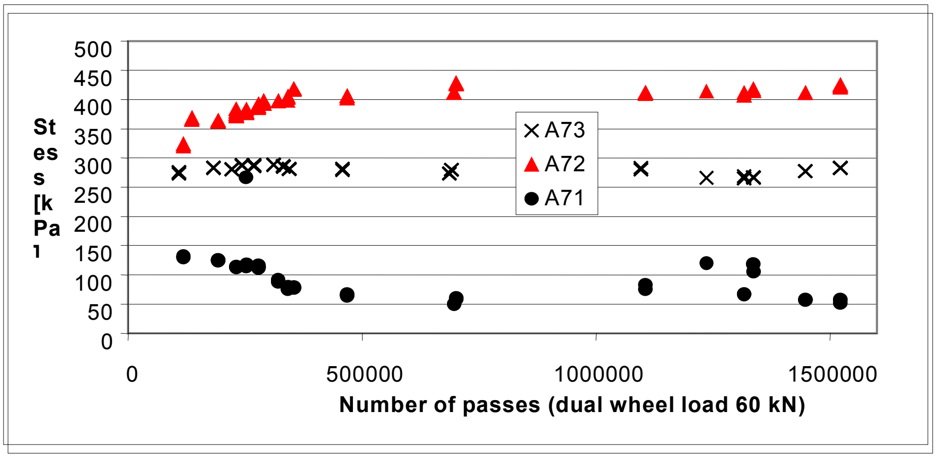

Figure 39. Development of stress in the middle of the base course (175 mm from the surface) vs. number of passes in test 01.

The development of stress on the surface of the subgrade is shown in Figure 40. According to all the sensors (A91-A93), the stress has increased during the loading.

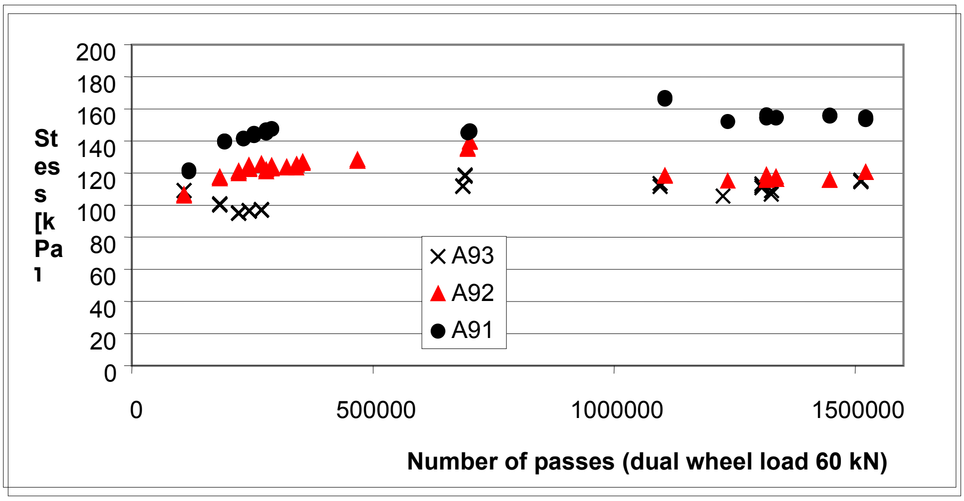

Figure 40. Development of stress on the surface of the subgrade (300 mm from the road surface) vs. number of passes in test 01.

The development of stress in the subgrade is shown in Figure 41. Generally, stress is increasing during loading. According to sensor A96, the stress has increased during the loading. There could be some fault in the functioning of sensor A97. This can especially be detected from the form of the top of the signal. In the latter signal, the top is slightly collapsed and the loading time has decreased.

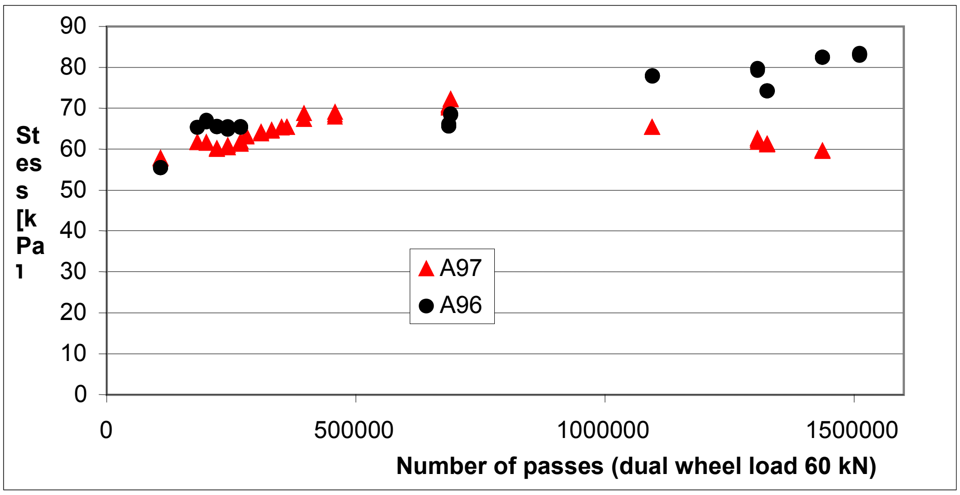

Figure 41. Development of stress in the subgrade (600 mm from the surface) vs. number of passes in test 01.

2.6 Testing after loading

2.6.1 Post mortem

Figure 42 shows how the grading curve of the crushed rock has changed during the loading in test 01 (Lusi-aggregate). The change is very small although the test pavement carried more than 1.7 million passes with 60 kN wheel load. This is caused by tough stone material.

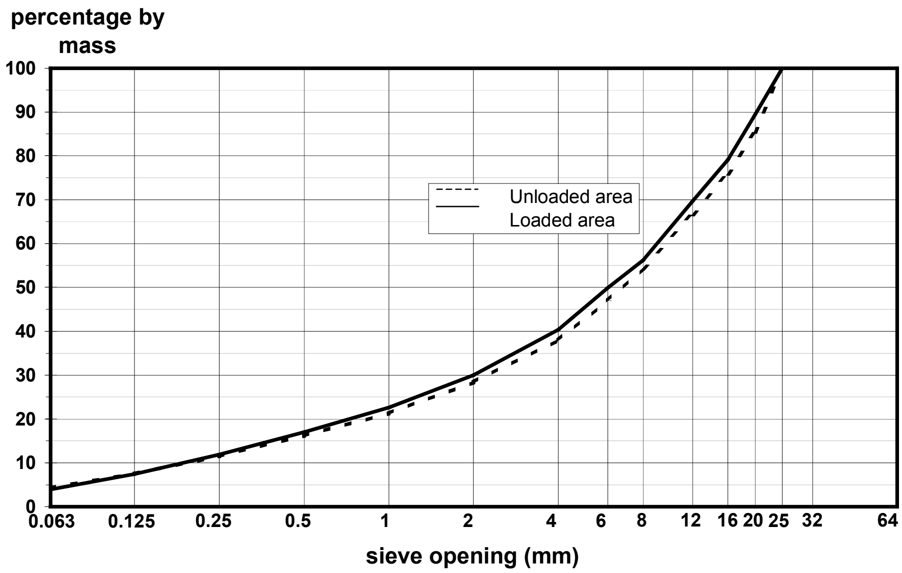

Figure 42. The grading curve from loaded area and from unloaded area in test 01.

Figure 43 shows how the grading curve of the crushed rock has changed during the loading in test 02 (Teisko-aggregate). There has not been any grinding as the fine-content has not increased but aggregates have broken because minerals are brittle (see point 2.2). The wheel load was very high, 80 kN dual wheel load. It is difficult to estimate the change in grading with normally used wheel load 60 kN.

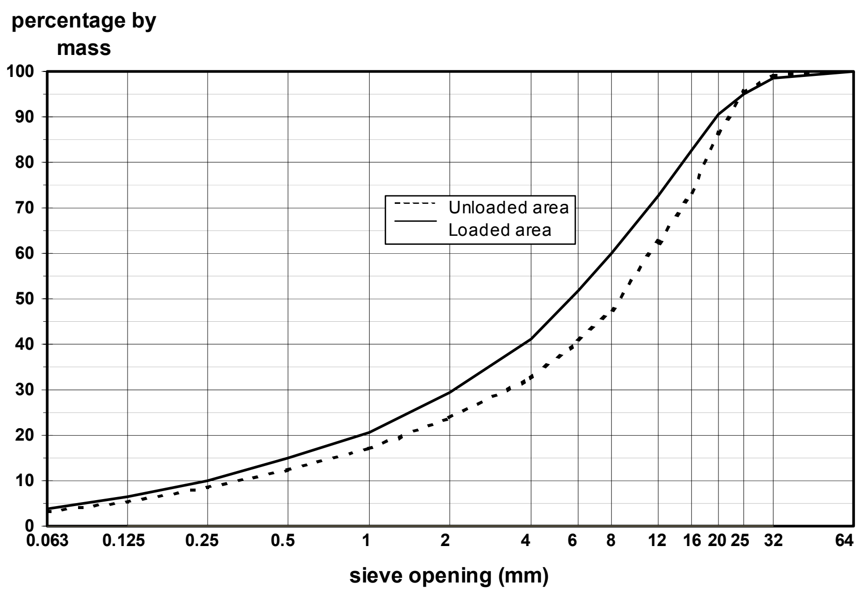

Figure 43. The grading curve from the loaded area (0.17 million passes with 80 kN dual wheel load) and from the unloaded area in test 02 (Teisko-aggregate).

3 LOADING MODE TESTS (TESTS 03-05)

3.1 Aim

The research idea was to test the effect of loading mode (bi-directional, single-wheel) and compare the results to those obtained from loading with different modes (uni-directional, single-wheel -test 04) and (bi-directional, dual-wheel -test 05).

It was decided to test section 03 with 70 kN single-wheel bi-directional, section 04 with 70 kN single-wheel uni-directional, and section 05 with 70 kN dual-wheel bi-directional, to study the effect of loading mode (uni-/bi-directional) and the effect of wheel type (dual/single).

One aim of the uni-directional test was to test the HVS-NORDIC itself in the warranty period.

Test structures 03-05 were built on hard rock. There is a 1300 mm blanket course on the rock, which is made of fine sand. Above the sand there is a 250 mm base course, which is made of crushed rock. On these unbound layers there is one bound layer, AC (B80). The structure of test 03-05 is shown in Figure 44.

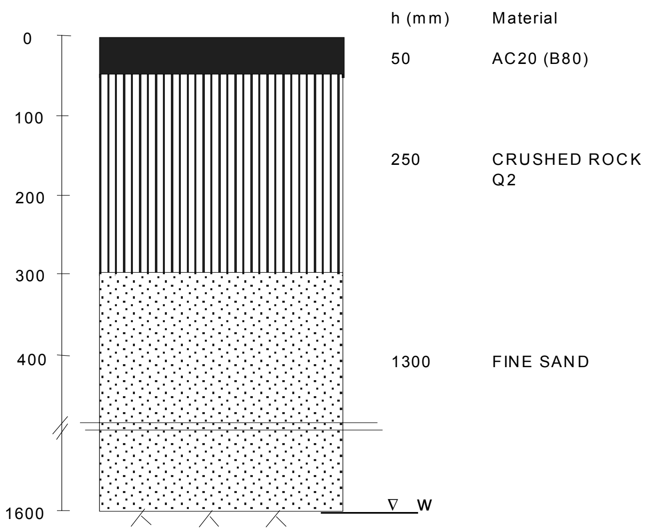

Figure 44. The structure of test 03-05.

3.2 Materials

The test structures are built on concrete. In tests 03-05 there is a 1300 mm blanket course on the rock. This layer is made of fine sand. In all tests, 03-05, above the sand there is a 250 mm base course made of crushed rock. On these unbound layers there is one bound layer, AC (B80).

Table 7. Base course material (Lusi-aggregate) in tests 03-05 and results of material tests.

Rock type	Granodiorite
Minerals	Micagneiss 39%, Quartz 30%, Cordiorite 12%, Plagioclase 10%.
Solid density	2.80 g/cm^3
Los Angeles	26.8
Max dry unit weight	22.27 kN/m^3
Optimum water content	7.22 %
Nordic Abrasion Value (Ball Mill Value)	21 %

The grading curve of the crushed rock in tests 03 and 04 is shown in Figure 45.

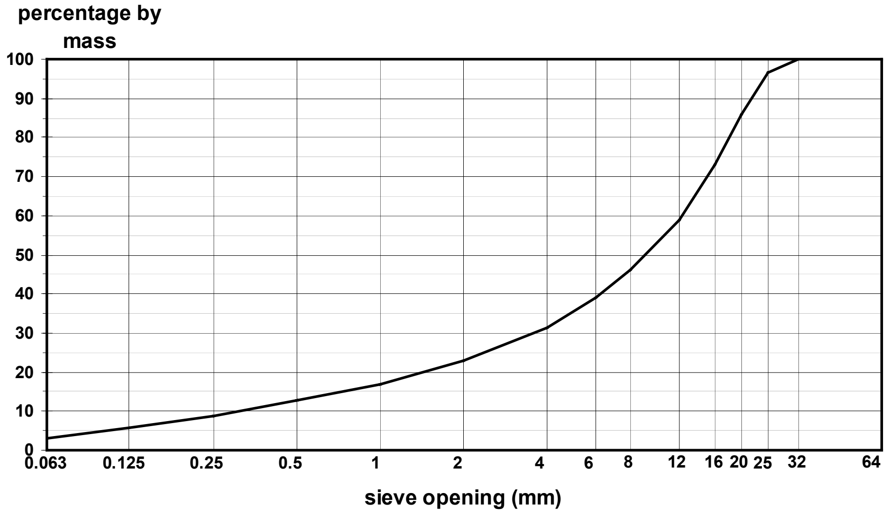

Figure 45. The grading curve of the crushed rock (Lusi-aggregate, base course material in tests 03 and 04).

The subgrade material is the same as in tests 01 and 02. The grading curves of the subgrade material are shown in Figure 10, and the result of the proctor compaction test is shown in Figure 11.

3.3 Construction

Tests 03-05 were built in a test pit that is excavated mainly in rock (rock pool) and it has no water-table regulation. The actual thickness of the bound layers of tests 03 and 04 are shown in Figure 46 and Figure 47. There was little variation in thickness; the thickness was defined from core samples.

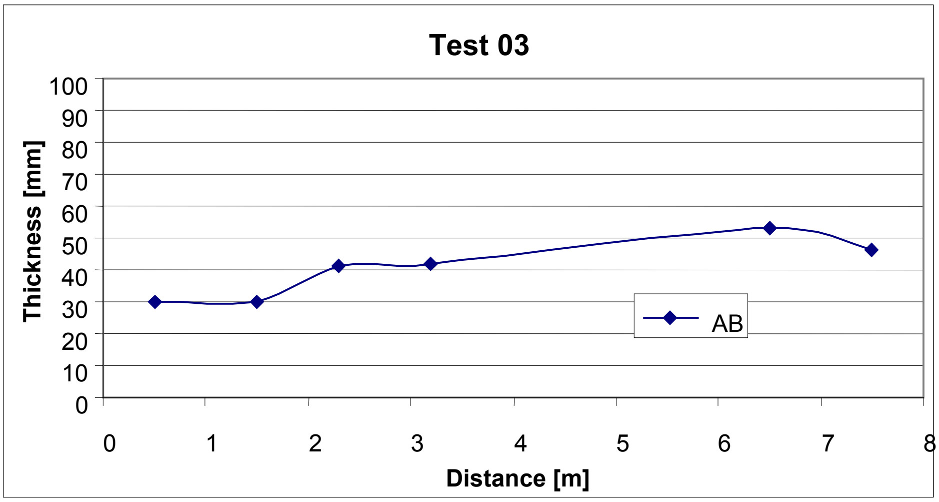

Figure 46. The actual thickness of the bound layer in test 03.

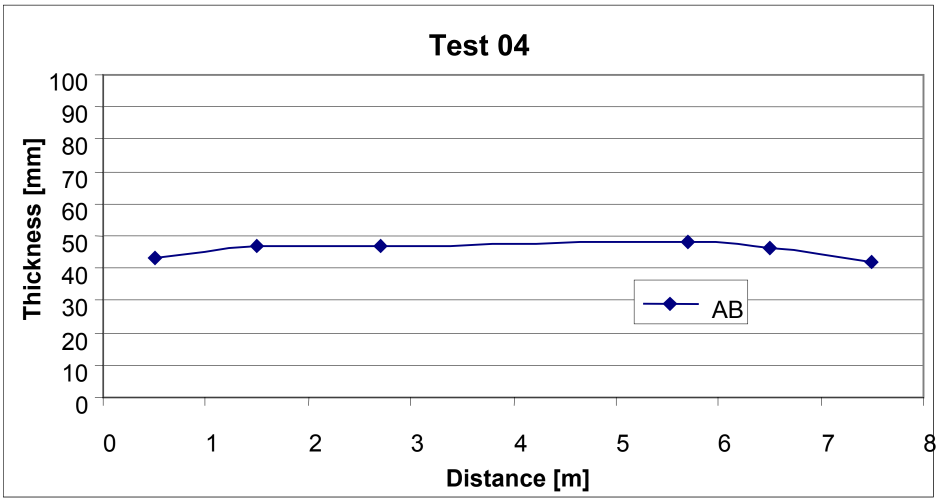

Figure 47. The actual thickness of the bound layer in test 04.

Table 8. Structures in tests 03 and 04.

Test	Structure
Test 03	40 mm AC20(B80) 250 mm crushed rock Q1 2300 mm fine sand
Test 04	46 mm AC20(B80) 250 mm crushed rock Q1 2300 mm fine sand

Quality control tests were carried out on the subgrade, for instance, water volumeter tests. The results of the water volumeter tests are given in Table 9.

Table 9. Results of water volumeter tests from subgrade in tests 01 and 02.

Surface of the subgrade +16.10 Optimum water content 14.6 % Max dry unit weight 16.8 γ_dkN/m^3															
Ground level	15.0	15.0	15.0	15.3	15.3	15.3	15.6	15.6	15.6	15.9	15.9	15.9	16.1	16.1	16.1
Water content %	6.8	6.7	7.5	7.6	7.0	6.8	6.9	7.3	7.4	7.3	6.4	6.8	6.6	7.0	7.5
Dry unit weight γ_dkN/m^3	14.9	15.0	15.2	14.6	14.9	15.0	14.9	15.3	15.1	14.7	14.9	14.9	15.5	15.4	15.0
Compaction index %	88.7	89.3	90.5	86.9	88.7	89.3	88.7	91.1	89.9	87.5	88.7	88.7	92.3	91.7	89.3

Falling Weight Deflectometer measurements were done after the asphalt was laid. The measurements show how homogeneous structures 03-05 were. The FWD deflections of the test constructions 03 and 04 are shown in Figure 48, and the deflection bowl in Figure 49.

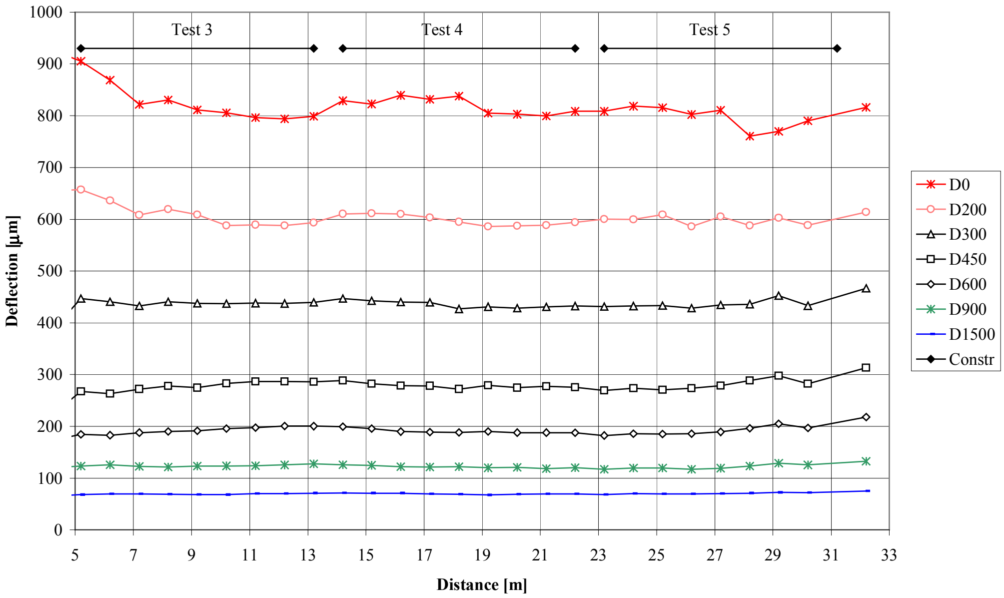

Figure 48. Deflections from the FWD measurements (50 kN) of tests 03-05. Pavement temperature was 7°C.

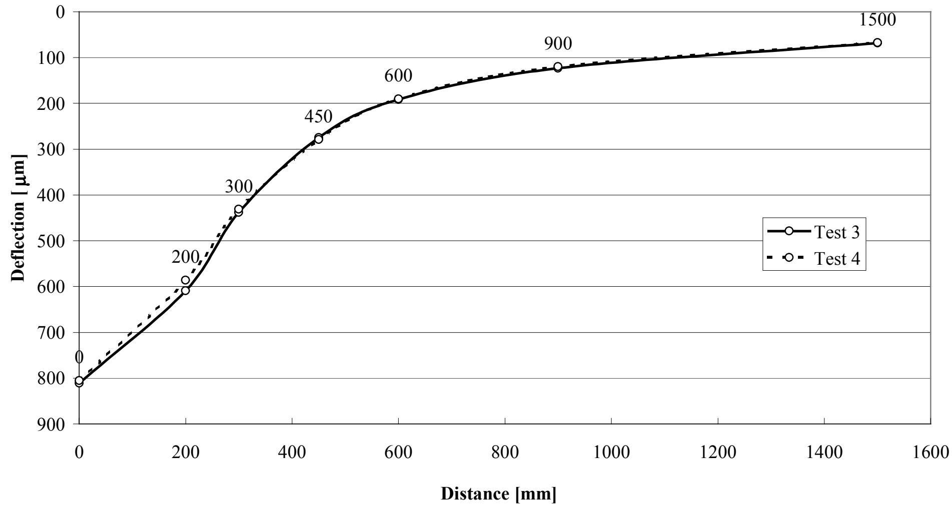

Figure 49. Deflection bowl in FWD measurements of tests 03-04. Temperature was 7°C.

The back calculation of the moduli of the pavement materials is presented in Table 10. The back calculation is made with the Modcomp 3 program, which is based on the linear elastic multilayer theory.

The first row of each case is the result of the free modulus calculation. The results are unreasonable because of thin asphalt layer. For this reason, the calculations have been made with a fixed bound material modulus. The fixed values are chosen based on the stiffness modulus tested in the laboratory and on the measuring pavement temperature. With fixed values (*), the calculated moduli are more reasonable.

Table 10. The results of back calculation (=fixed value).*

Test 03 (Resilient Modulus, Modcomp3)				
Distance (m)	AC (7 °C)	Base course	Subgrade	Bedrock
2	21700	75	93	*10000
	*8000	205	68	*10000
4	25600	58	104	*10000
	*8000	217	66	*10000
6	17800	139	70	*10000
	*8000	244	61	*10000
Test 04 (Resilient Modulus, Modcomp3)				
Distance (m)	AC (7 °C)	Base course	Subgrade	Bedrock
2	16700	110	78	*10000
	*8000	200	66	*10000
4	12700	142	74	*10000
	*8000	194	68	*10000
6	18400	126	76	*10000
	*8000	223	66	*10000

3.4 Test

In test 03, the loading mode was bi-directional, single-wheel (70 kN), and in test 04 the loading mode was unidirectional, single-wheel (70 kN). In tests 03 and 04 the speed of the loading tyre was 12 km/h (see Table 11).

Table 11. Test parameters in tests 03 and 04.

Test	Test parameters
Test 03	70 kN, 800 kPa, bi-directional, single, 10 °C, 12 km/h
Test 04	70kN, 800 kPa, uni-directional, single, 10°C, 12 km/h

3.5 Results

3.5.1 Rutting

After the initial response measurements, 1.4 million loadings in test 03 and 0.32 million loadings in test 04 were made. In test 03, about 34 mm and in test 04 about 22 mm of rutting could be seen on the road surface. The rut depths are shown in Figure 50.

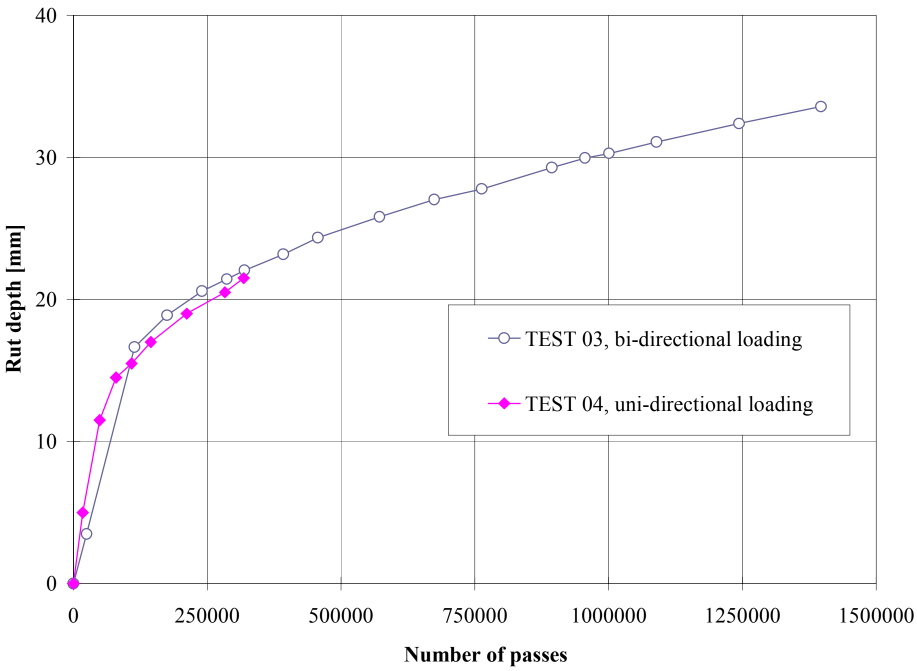

Figure 50. Rut depths vs. number of passes, test 03 (70 kN single-wheel load, bi-directionally) and test 04 (70 kN single-wheel load, uni-directionally).

3.5.2 Cracking

The first crack was found after 890 000 passes in test 03. Overall, seven transversal cracks were found in test 03. In test 04, no cracks were found.

3.5.3 Response measurements

Initial response measurements were made after the pre-run before the actual test loading. A huge number of different parameter combinations were used in the measurements.

Typical behaviour of the strain and stress in every measured layer vs. the speed and temperature is dealt with in Section 2.5.4. (Response measurements). The figures and texts therein also apply to tests 03 and 04 because structures 01-05 were alike. There was some minor variation in the level of strain/stress in each structure but the behaviour in general was similar.

The pavement response due to the HVS wheel load was measured during testing with test parameters only. Figure 51 to Figure 54 show the development of sensor values in test 03. There are only slight changes in the strain /stress values during the test. These tests were done with the single tyre (70 kN, 850 kPa, 10 km/h, side location 0).

The development of longitudinal strain at the bottom of the bituminous layer is shown in Figure 51. According to sensor C11, the strain has slightly increased during the loading.

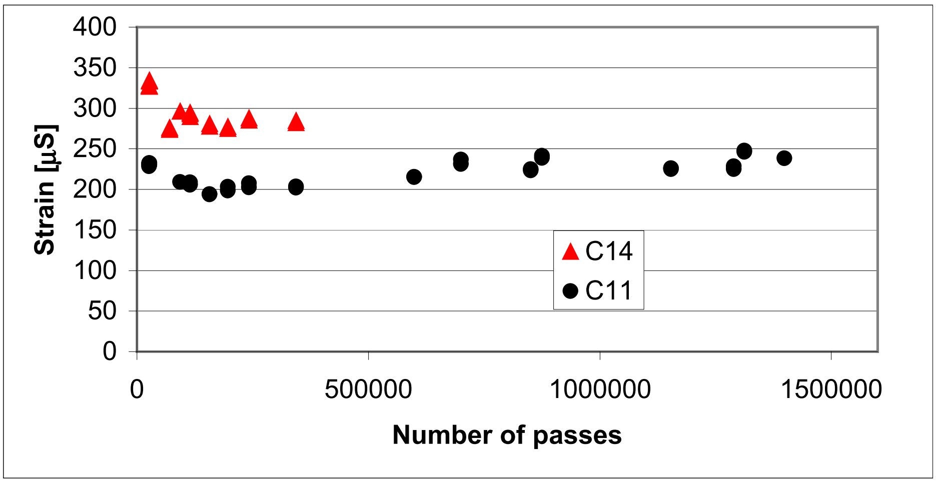

Figure 51. Development of longitudinal strain at the bottom of the bituminous layer vs. number of passes in test 03.

The development of stress in the middle of the base course is shown in Figure 52. According to sensor C73, the stress has increased during the loading, and according to sensors C71 and C72 the stress level has not changed much.

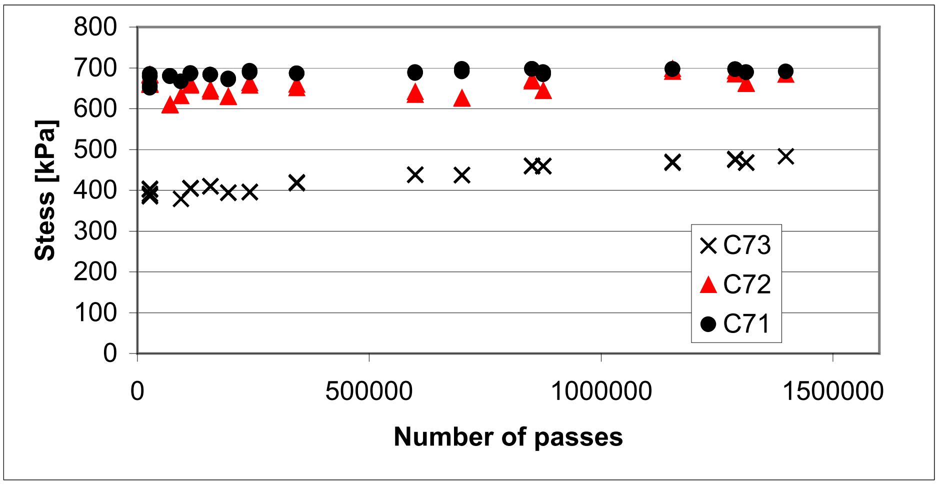

Figure 52. Development of stress in the middle of the base course (17.5 cm from the surface) vs. number of passes in test 03.

The development of stress on the surface of the subgrade is shown in Figure 53. According to sensor C93, the stress has increased during the loading, and according to sensors C91 and C92 the stress level has not changed much.

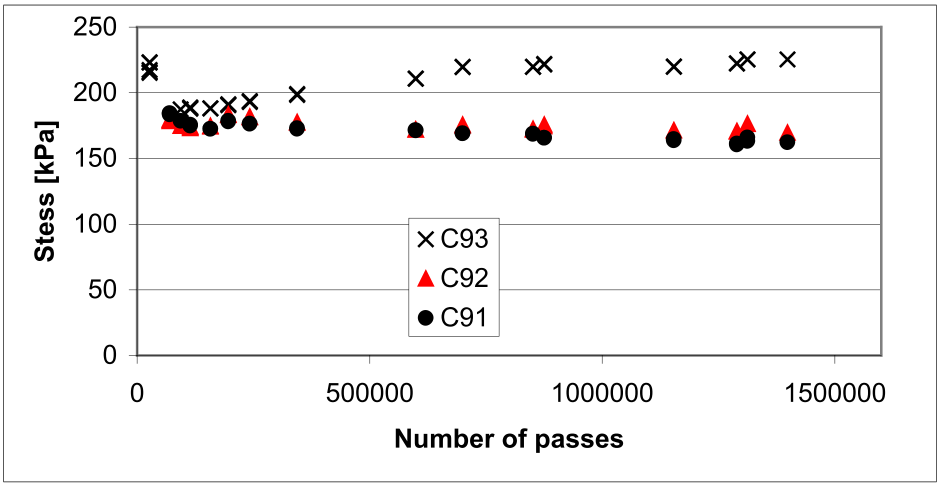

Figure 53. Development of stress on the surface of the subgrade (30 cm from the surface) vs. number of passes in test 03.

The deflection of pavement surface is presented in Figure 54. Deflection has increased slightly in the end of test.

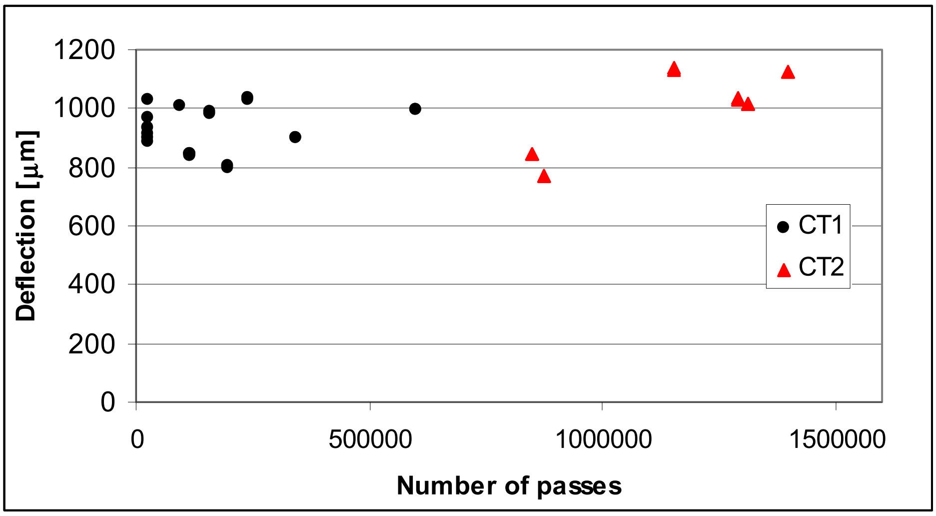

Figure 54. Deflection vs. number of passes during test 03.

3.6 Testing after loading

3.6.1 FWD testing

After loading, the HVS was removed from the test sections and FWD measurements were made on both sections. Figure 55 shows how the deflection (measured with Falling Weight Deflectometer) values change during the HVS loading. The values after the loading are much lower than the values before loading, even though the temperature was higher in the measurements after loading (bullets without line are the values before loading and bullets with line are values after loading). This could be the consequence of condensation of the layers. Test area 05 was not under HVS loading.

Figure 56 shows how the deflection bowl has changed during the HVS loading. The change in the deflection bowl was nearly equal in both tests 03 and 04, although there was a vast difference in number of passes.

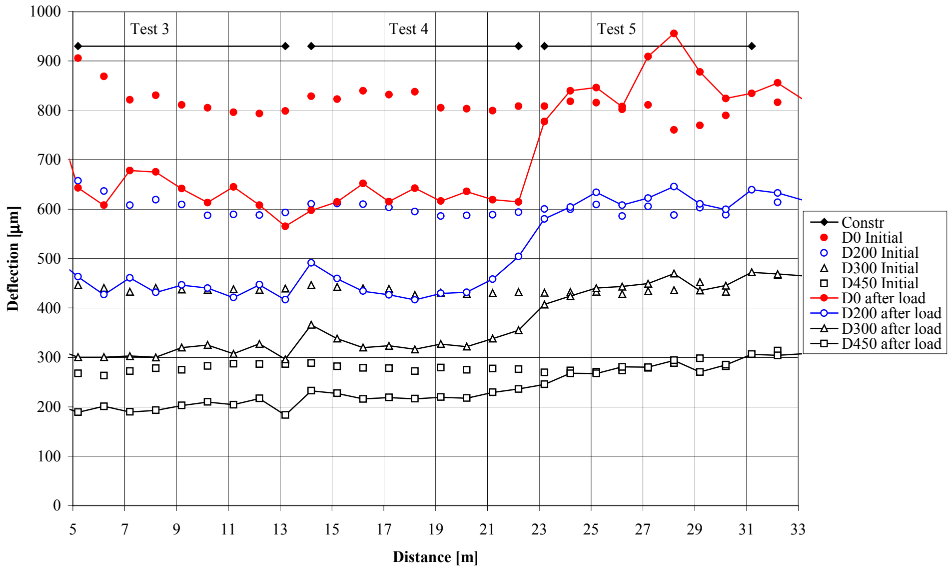

Figure 55. Deflections from FWD Measurements (50 kN) of tests 03-05 (before and after loading). Temperature was 7°C in the measurements before loading and 15°C after loading.

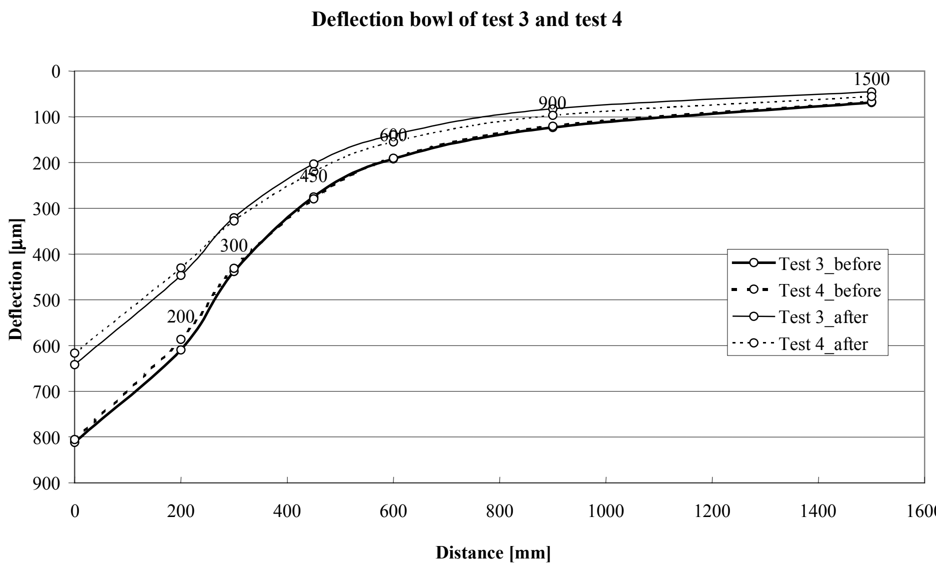

Figure 56. The deflection bowl of the FWD measurements in test 03 and 04 before and after loading. Temperature was 7°C in the measurements before loading and 15°C after loading.

3.6.2 Post mortem

When the test was completed samples were taken from both unbound and bound materials. Both loaded and unloaded areas were sampled, too. The samples were tested in laboratory according to "basic minimum" program.

When structures 03 and 04 were broken up, the thicknesses and the form of the bound layer and base course were measured. These measurements were made for research of contraction. The measurement results are shown in Figure 57 and Figure 58. Because its surface seems to be a little higher outside the rut, this deformation is not only due to direct compaction but also because of shear movements. Deformation has occurred mainly in the base course. The asphalt concrete comprised a few millimetres and perhaps subgrade, too.

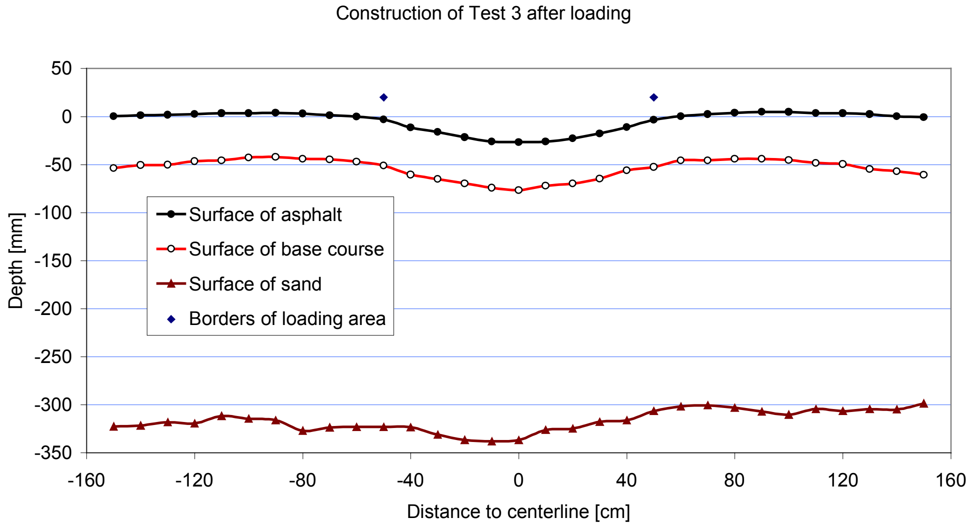

Figure 57. Form of construction 03 after loading.

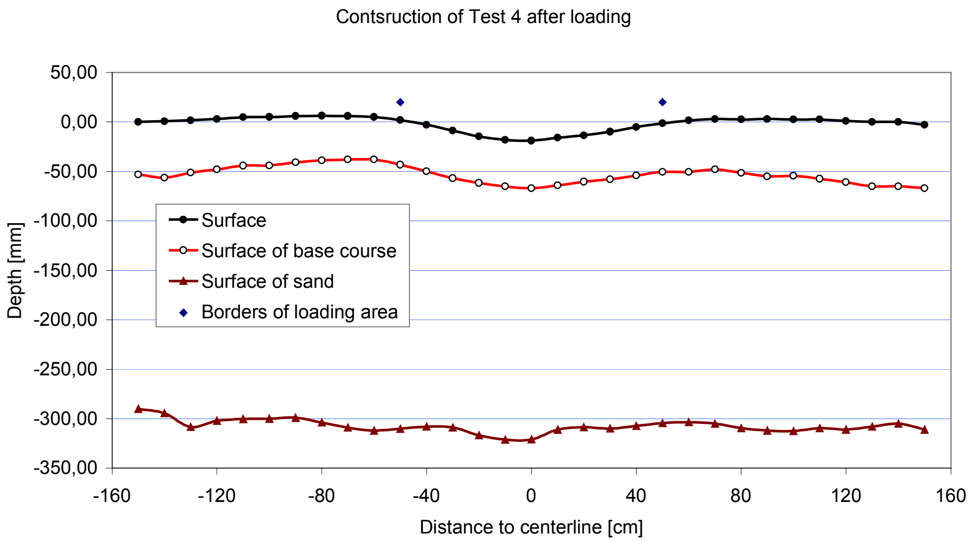

Figure 58. Form of construction 04 after loading.

Figure 59 shows how the grading curve of the crushed rock has changed during the loading in test 03. The material has crushed about 5 %-unit between 1 and 16 mm.

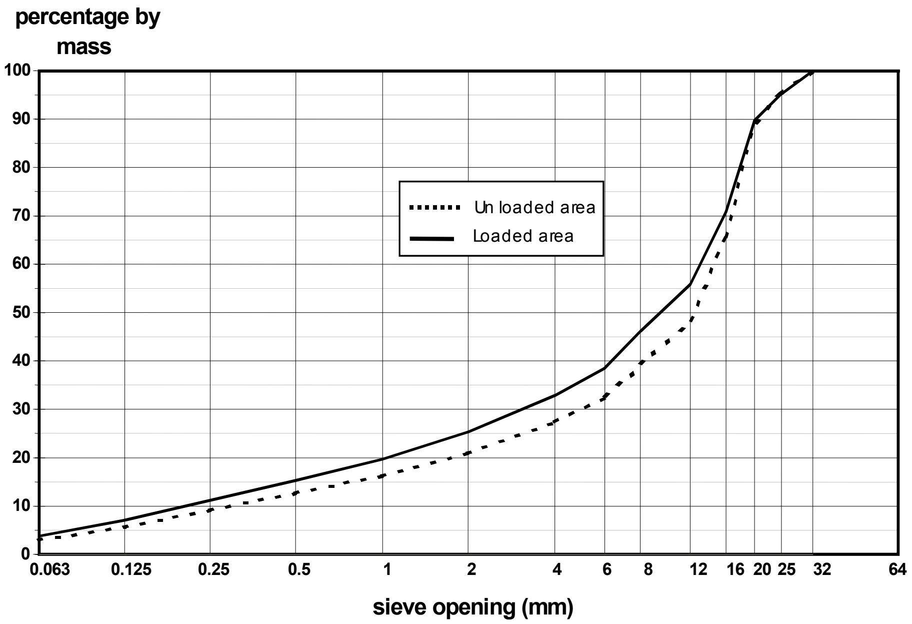

Figure 59. The grading curve from the loaded area and from the unloaded area in test 03, 1.4 million passes with 70 kN single wheel load.

Figure 60 shows how the grading curve of the crushed rock has changed during the loading in test 04. The material has crushed about 3.5 %-unit between 1 and 16 mm.

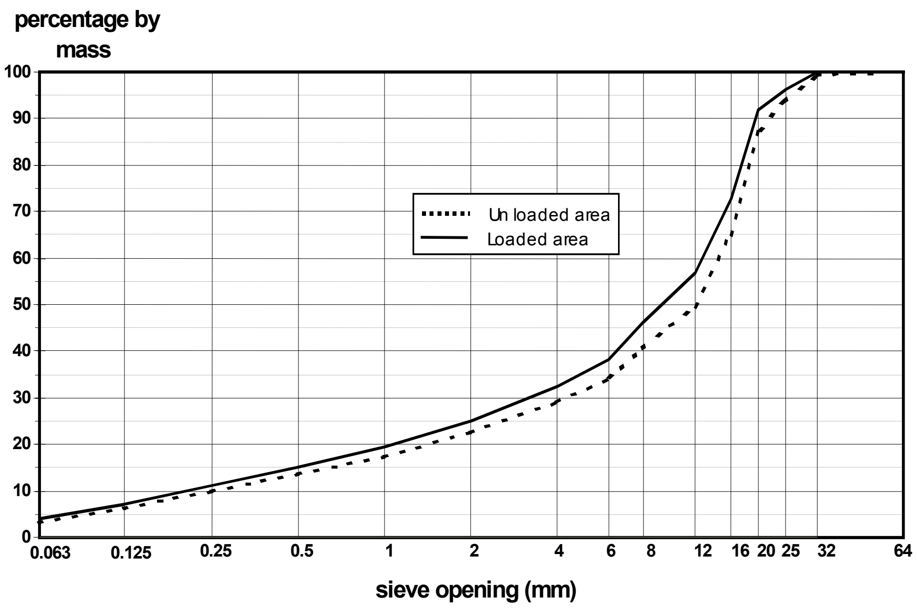

Figure 60. The grading curve from the loaded area and from the unloaded area in test 04.

4 DISCUSSION

4.1 Aims

This report covers the first five tests made with the HVS-NORDIC. The main aims of the tests were to

- test pavements, which can be used later as a reference
- see the difference between two unbound granular base courses
- see the effect of the water–table level
- see if there is any difference between uni- and bi-directional loading modes
- test the general properties of the HVS-NORDIC more widely after the official acceptance test
- gain experience in running HVS-NORDIC
- gain general experience of the linear ALT facility
- see if there are any technical problems during the warranty period.

4.2 Lifetime

The pavements lasted longer than expected. It is a well-known fact that very often pavements last longer in Accelerated Loading Tests than estimated. VTT has previous experience of this from the dynamic axle load test at CAPTIF in New Zealand /15/ where pavements lasted much longer than expected. VTT also has experience from the Manége de fatigue of LCPC in Nantes /16/ where pavements lasted roughly according to expectations. Both were circular, the first one with a relatively small radius (7 m) and speed (45 km/h) and sheltered from rain and sunshine but no temperature control, and the second one with greater speed (72 km/h) and radius (40 m). No special reasons were found for good pavement performance at CAPTIF.

With HVS-NORDIC, the pavement temperature and the water-content of the unbound layers and subgrade can be kept constant. Thus, a good comparison between tests can be attained, which is important because only in certain cases can two pavements be tested simultaneously. In reality, especially in the Nordic countries, the pavement temperature and water content vary considerably and the surfacing is also subject to rain and sunshine.

The cracking of bituminous material and rutting are the main types of distress caused by traffic loading. Such cracking is due to fatigue in the bituminous layer. It is typical of the fatigue in any material that at first, during many loadings, the damage is invisible and cannot be detected by any normal means. After that phase though, cracking usually develops reasonably fast. Ruts are formed in the beginning more rapidly (initial rutting) and after this phase much slower. Because the nature of these two phenomena is different, it is not plausible to change the conditions during a test because distresses are in different phases and thus not comparable to other tests.

Both temperature and water content vary in real road pavements. In these tests, the water-content was first close to the optimum water content that was used during construction. Because the test pits are closed and there is drainage (water table at –2.4 m in test pit 1), the water content will decrease, perhaps 2-3 percentage points, as the water sinks down because of gravity /11/. The water table in Finland is very seldom as deep as it was in the test. Basically the same happens also in real pavements, but some more water penetrates into the pavement structure through the shoulders, and if there are cracks, through them. There is not much data about seasonal variations of water content in pavements, but usually the thaw period and autumn rains can be seen as peaks in water content (see also 4.1.4 Rutting).

Distress will usually occur as the road is in its weakest condition, which means that the water content is great (usually because the water table is high) and the temperature is relatively low. Testing in absolutely the weakest condition is, however, seldom plausible.

It was decided after tests 01-05 that the water table will be normally 300 mm from the surface of the subgrade sand. Test pit 2 was made waterproof in order to make that possible there, too. There is no experience yet of whether this change will solve the problem.

This lack of seasonal variation may also explain why the OECD test at CAPTIF, New Zealand lasted longer than predicted.

4.3 Cracking

No cracking could be found in test 01 even after 1.5 million loadings. After the water table was raised, rutting increased dramatically and the first cracks (transverse) could be found after 0.1 million more loadings. Cracks (transverse) were found in test 03 only after 890 000 loadings even though the single wheel load was 70 kN. Rough estimates were made before the test and cracks were estimated to appear much earlier.

Figure 61 presents the rutting of two tests. The road structures are similar but test 01 was loaded with a 60 kN dual wheel and test 03 with a 70 kN single wheel. In the latter, rutting is about twice that of the former. In test 03 a few cracks were also found after 0.8 million loadings.

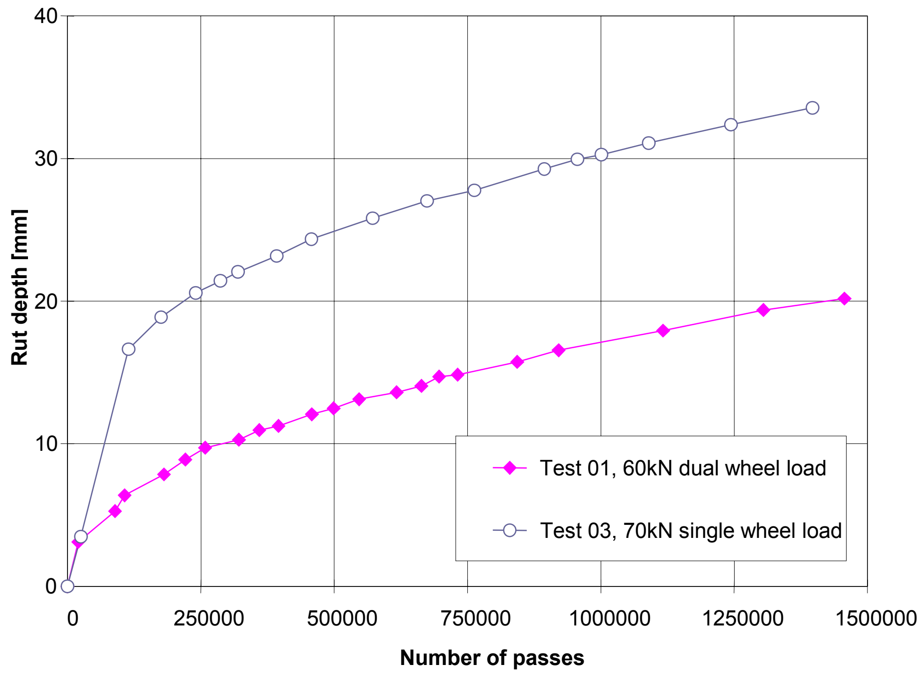

Figure 61. Rut depths vs. number of passes, test 01(60 kN dual-wheel load) and test 03 (70 kN single-wheel load).

The facility to measure the fatigue properties of bituminous mixtures at VTT uses a specimen (420 mm x 100 mm x 50 mm) on a rubber base. Force is applied in the middle of the beam sample thus bending the beam. The duration of the force is 0.05 seconds and the rubber bed bounces the specimen back. Next, loading occurs after 1 second. The strain is measured at the bottom of the sample and it is used in order to control the force actuator. Both stress and strain control can be used. Cracks are detected by strain gauges and as soon as two strain gauges have been broken the test is finished. Six samples can be tested independently at the same time.

Similar asphalt concrete samples have been tested in the fatigue facility at a temperature of 15 °C, or at 5°C warmer temperature than in HVS-NORDIC. Test 01 was finished after 1.5 million passes. According to the fatigue test in the strain control test, this number is attained at a strain level of 330 µS, and in the stress control test at 230 µS strain level. As the bituminous layer is thin, only 50 mm, the strain control is more realistic.

The strains in the HVS-NORDIC test are about 300 □S thus the first cracks could have been expected only soon after the test had been finished.

There are certain factors that slightly modify the conclusion above:
– crack growth
– transverse wander of the wheel in HVS-NORDIC
– temperature 10°C instead of 15°C at the fatigue facility
– healing of bituminous materials in HVS-NORDIC.

The criterion for the fatigue failure in the fatigue testing facility of VTT is the breaking of two strain gauges at the bottom of the specimen. One very common criterion in the fatigue tests is that the modulus is half of the original value. The strain gauges usually break earlier. After the first cracking has been found, it will take some time before it can be seen on the surface. Thus, cracking on the surface can be seen later than estimated from the fatigue tests.

The strain values are measured as the wheel is on the sensor. The lateral wander of HVS-NORDIC simulates the lateral wander on a road and thus many wheel passes induce some smaller values. This effect delays, to some extent, the formation of cracks.

Fatigue testing has been conducted at slightly higher temperatures (15°C) than the test with HVS-NORDIC has been done (10°C) or the fatigue properties of HVS-NORDIC bituminous mixtures are somewhat worse. This has only a minor influence because the change in the stiffness has no effect as the results are based on strain measurements.

Cracks have been very thin and they have not widened later during the test. This may be due to the lack of water and dirt with it. It has also been found that thin cracks seem to disappear later, or healing effects exist especially as the crack will stay clean.

The pavements that were used in these tests are typical for low to medium-volume roads in Finland. They were selected to be weak enough in order to avoid the first tests lasting for too long. Thus, in our case, the bituminous courses were thin, only 50 mm.

The strain measurements revealed that the strains in thin bituminous pavement do not only increase with load but may also decrease (Figure 27). The reason is that the length of the contact interface between the tyre and pavement increases as the load increases at the same tyre inflation pressure. Another contributory reason is that, because of thin bituminous layers, the stress on the unbound layer is greater and consequently its modulus is greater, because?? the moduli of unbound granular materials are stress-dependent. This phenomenon has been found earlier in our response measurements at the Virttaa test site /15/.

The preceding phenomena mean that strains at the bottom of the bituminous layers do not increase proportionally to the load, but much less if the bituminous layer is thin. If the performance of bituminous material in accelerated loading is to be studied and cracking is the failure mode, the test must be planned very carefully. In order to get results in reasonable time, the bituminous layer should be thin. In order to get cracking as the failure mode, it may be necessary to have higher tyre inflation pressure than recommended by the manufacturer in order to obtain the tyre imprint smaller and strains in the pavement greater. The effect of base course and/or its water content must be carefully considered during the design of the test.

Cracks are seen on the surface and there are no means to see starting cracks in the bottom of the bituminous layer. Indirectly some indication of cracks can be attained from strain gauge measurements. This is covered later in 4.1.8 Analysis of strain signals.

4.4 Rutting

It can be seen from post-mortem Figure 57 and Figure 58 (tests 03 and 04) that rutting occurred mainly in the base course. Because there is some heave outside the loaded area in any case, some of the rutting is due to shear failure. Some post-compaction has occurred in the bituminous layers, but no plastic deformation because the temperature was relatively low, only 10C. Some deformation has also occurred in the subgrade.

During the test, unbound granular materials crushed, as can be seen in Figure 59 and Figure 60. This change in grading curves is due to the breaking up of aggregate and there is no grinding effect because there is only a minor increase in the amount of fines.

The rut depth at the end of test 01 was 65 mm. Strong rutting has occurred in the subgrade with clear shear effects. The rut was so deep that the whole structure deformed and it is impossible to judge if any part of the rutting has happened also in the base course. Because the water table was so high, it is understandable that deformation occurred in the subgrade during the last phase of the test. It is probable that rutting occurred in the base course in the earlier phase of the test, as the pavement was dry.

About the same rutting effect had 80 kN dual wheel load and 70 kN single wheel load (test 02 and 03).

One conclusion of these post-mortem results is that the tests should be finished early enough; ruts should not exceed 30…40 mm at the end of the test in order to get a realistic view of what has happened in the pavement structure during the test. In practice, the rut depth in the Finnish road network should not be more than 20 mm, but in order to clarify the picture, there is good reason to continue the tests.

According to these results, rutting may happen in the base course or in the subgrade; there exists no general rule but each case depends on the material properties, layer thickness and water contents.

Tests 01 and 02 measured mainly the properties of unbound granular materials, as was the aim, and not rutting in bituminous layers or in subgrade.

If the interest is to measure the plastic deformation in bituminous materials, the pavement temperature must be higher, preferably 40°C, which is the pavement temperature during a beautiful summer day in Finland. This kind of test was done very successfully later as a part of the EU V Framework Research Project, REFLEX, in Sweden.

4.5 Loading mode

Loading in the HVS-NORDIC can be in one direction only or in both directions. Loading on a real road is uni-directional but if bi-directional loading mode is used the capacity of HVS-NORDIC is double. There is also less stress on the facility as there is no need to hoist and lower the wheel.

As the wheel runs on the road the principal stresses in the unbound layers rotate. If there is rotation always in the same direction, as in uni-directional loading, the aggregates may find stable structure and may afterwards become more deformation-resistant material. If there is bi-directional loading the direction of rotation changes after each pass and there may not be such "locking" as in uni-directional loading.

The comparison was to be made in tests 03 and 04. Tests 03-04 (Figure 50) showed, however, no difference in rutting between both loading modes. Unfortunately, test 04 had to be finished prematurely because the HVS-NORDIC was needed for thaw tests that could not be postponed. There were probably enough loadings for rutting in test 03 but not in test 04, though in neither case enough for cracking. However, there is no indication in the figure that there would be any difference later.

The University of Oulu has compared the effect of uni- and bi-directional loading in their small-scale accelerated testing facility and they found that there was about 30 % more vertical deformation if bi-directional loading was used /14/. They use a very soft and thin bituminous layer, which is mainly for the protection of the unbound layer. Even though the wheel is smaller, the tyre inflation is nearly the same as in bigger tyres (600 kPa) in order to get about the same contact stress as in real vehicles although the load was only 10 kN. The stresses in the unbound layers are greater than in the HVS-NORDIC, and because the tyre imprint and the dimensions of the test pit are small the rotation of principal stresses occurs close to the surface and is greater than in the HVS-NORDIC and in a real road.

If the loading is uni-directional there may be an accumulation of residual stresses, and cracking may occur earlier than in the bi-directional loading, where opposite loading may relax the stresses during every load pass. Since the test had to be interrupted and there were no cracks, the effect of uni- and bi-directional loading modes in bituminous materials is only on the hypothesis level.

A conclusion from this loading mode test is that bi-directional loading can be used in the future tests, especially as the capacity of HVS-NORDIC will be double.

4.6 Possible effect of low speed

The moduli of bituminous materials depend on the speed, or the effective moduli are smaller than on the roads. This may increase rutting but should not effect cracking too much. This phenomenon could perhaps be avoided by using harder bitumen, which has been considered but not used.

As the speed is slower than on the road, the change in strain from compression to tension happens slower than on the road, or the strain gradient is smaller. The fatigue properties may depend on this but the authors have not found any research results.

4.7 Effect of new pavement

In HVS tests pavements were quite new less than one year old. In real life, pavements are during their life time older and get stiffer and more brittle because of ageing effect. It is difficult to evaluate the effect of ageing on HVS test results.

4.8 Analysis of strain sensor signals

The development of signal from sensor A12 (triangles in Figure 37) can be seen in Figure 62. The most likely explanation for the reduction in tensile strain and the form of the signal is that there is a crack close to the sensor. The crack is so thin that it transmits compression but not tensile strain totally. Compression is equal in both signals but strain is smaller in the later signal.

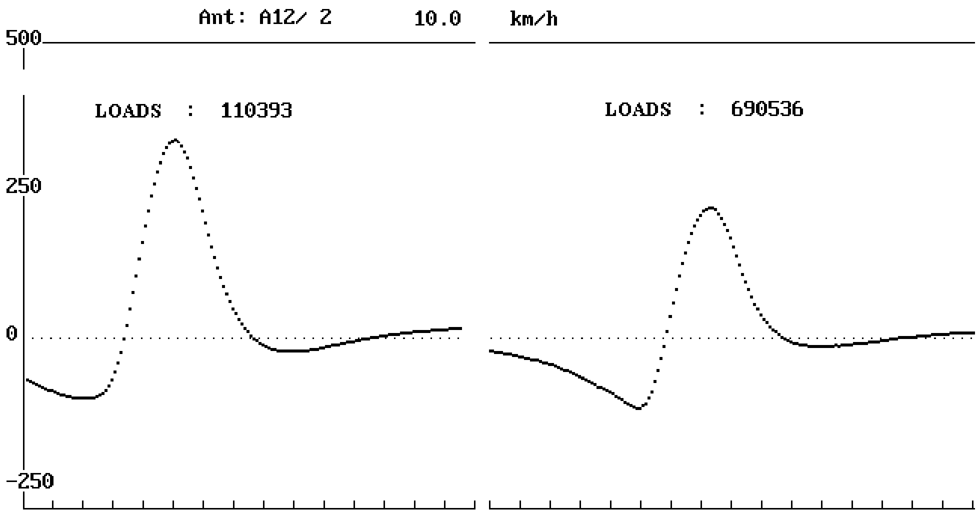

Figure 62. Longitudinal strain (µS) at the bottom of bituminous layer vs. time (s) in different stages of test 01 (sensor A12). Left from the beginning of the HVS loading, and right from the middle of loading.

The development of signal from sensor A11 (bullets in Figure 37) can be seen in Figure 63. The strain becomes slightly smaller when the number of loads increases. The reason for this may be an initial crack close to the sensor or deterioration of the base course.

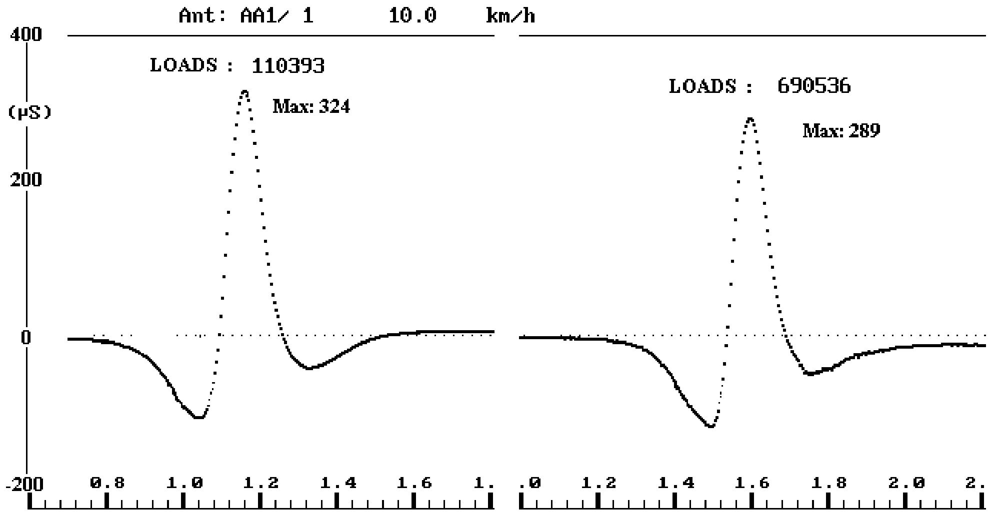

Figure 63. Longitudinal strain at the bottom of the bituminous layer vs. time (s) in test 01(sensor A11 (AA1)). Left from the beginning of HVS loading, and right from the middle of loading.

The development of signal from sensor A72 (triangles in Figure 39) can be seen in Figure 64. Stress increases during loading. The reason for this could be an initial crack in the asphalt or grinding of the base course material.

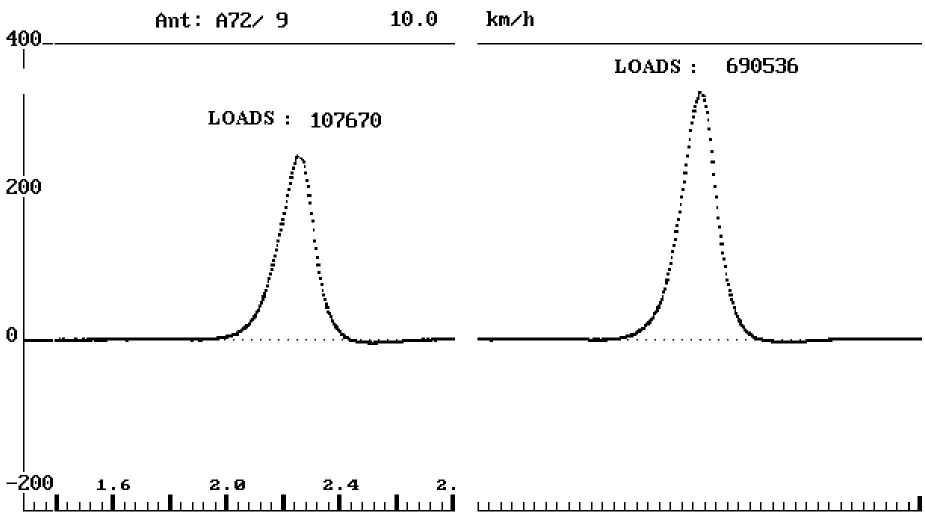

Figure 64. Stress (kPa) in the middle of base course vs. time (s) in test 01 (sensor A72). Left from the beginning of HVS loading, and right from the middle of loading.

5 CONCLUSIONS

Two tests were made on the test site close to the office of VTT's personnel in Espoo, Otaniemi. The structures of tests 01 and 02 were for base course tests, where the quality of base course material was different. These two test structures act as reference pavements for the research programme.

After the initial response measurements, over 1.7 million loadings were made in test 01. Structure 01 lasted for 1.6 million repetitions without cracks, only 25 mm rutting was found. When the water-table level was raised to 0.3 m from the surface, only 0.2 million load repetitions were needed to increase rutting to 65 mm, and ten transverse cracks were also found on the surface.

After the initial response measurements, only 0.17 million loadings with an 80 kN dual-wheel load were made on test 02. After 0.17 million passes only 18 mm rutting could be seen but no cracks. After these loadings, the test had to be stopped because thawing test constructions were made in the same test pool.

The next three tests 03-05 were loading mode tests. The research idea was to test, in the reference pavement, the effect of loading mode (bi-directional, single wheel) and compare the results to those loaded with different modes (uni-directional, single wheel; test 04) and (bi-directional, dual wheel; test 05).

A part of the testing was to test the properties of the HVS facility itself, especially in the guarantee period.

According to the experiences of the first tests, the following conclusions could be drawn:

1 The first result was that the pavements lasted for much longer than expected, when the water table level was more than 2 m below the road surface.

2 Deterioration was mainly rutting in dry circumstances, only a few cracks were found.

3 In dry circumstances, low quality base course material behaved well because it is tough. In wet circumstances the situation is probably unlike.

4 Under high wheel load (80 kN) high quality base course material crushed much. Under normal wheel load (60 kN) the situation could be otherwise.

5 Raising the water-table level up to the bottom of the base layer had a dramatic effect on rutting and cracking.

6 Initial strain measurements revealed that the strains in thin bituminous pavement not only increase with load, but may also decrease. The

reason is that the length of the contact interface between the tyre and pavement increases as the load increases at the same tyre inflation pressure.

7 According to the response measurements during testing, the observation response signals and top values in different stages of tests, some initial cracking was predicted to appear at the bottom of the bituminous layers.

8 No difference was found in rutting when the loading mode was uni- or bi-directional. Consequently, the bi-directional loading mode can be used to give HVS double efficiency compared to the uni-directional loading mode.

9 The personnel of VTT learned a lot of APT during the first tests in Finland

10 Despite the teething problems in the beginning, the HVS facility performed reasonably well during the first tests; the unscheduled down time was less than 20% of the total time.

Each test produces a considerable amount of data. The APT is neither simply a performance test, nor are only the lifetimes of different pavements compared. It is important to study the whole deterioration mode, too. The other measurements, especially the response measurements, are very important. All the data from constructions, response measurements and observations are strored into common data base for later analysis. The final comparison of test structures can be made only after careful analysis of that data.

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APPENDIX 1

Owerview of HVS tests and results.

Test	Objective	Structure	Pre-load parameters Passes Wheel load, kN Tyre type Temperature Speed	Test parameters Wheel load, kN Tyre pressure, kPa Tyre type Temperature Speed	Rut depth at		First crack at	Remarks
					Passes (pre-run included)	Rut, mm		
FIN01	Reference pavement, low-quality base material	50 mm, Asphalt 250 mm, Crushed rock 2300 mm Sand	0-72.000 40 kN, bi-directional Dual 10 °C 10 km/h	60 kN, bi-directional 800 kPa Dual 10 °C 10 km/h	100 000 200 000 400 000 800 000 1 200 000 1 700 000	6. 0 mm 8. 5 mm 11. 2 mm 15. 3 mm 18. 5 mm 62. 0 mm	-	GW in sand layer, N=1 510 000- 1 710 000
FIN02	Reference pavement, high-quality base material Wheel load	50 mm, Asphalt 250 mm, Crushed rock 2300 mm Sand	0-37.350 30 kN, bi-directional Single 10 °C 10 km/h	80 kN 800 kPa Dual 10 °C 10-12 km/h	20 000 40 000 100 000 140 000 170 000	2. 8 mm 5. 5 mm 13. 5 mm 16. 2 mm 17. 5 mm	-	
FIN03	Reference pavement, low-quality base material loading mode	50 mm, Asphalt 250 mm, Crushed rock 1300 mm Sand	0-25.000 30 kN, bi-directional Single 10 °C 10 km/h	70 kN, bi-directional 800 kPa, Single 10 °C 12 km/h	100 000 200 000 400 000 800 000 1 200 000 1 400 000	15. 0 mm 19. 5 mm 23. 0 mm 28. 0 mm 32. 0 mm 33. 5 mm	890 000	
FIN04	Reference pavement, low quality base material loading mode	50 mm, Asphalt 250 mm, Crushed rock 1300 mm Sand	0-17.400 50 kN, bi-directional Single 10 °C 10 km/h	70 kN, uni-directional 800 kPa Single 10 °C 12 km/h	100 000 200 000 318 000	15. 0 mm 18. 8 mm 21. 5 mm	-	unfinished
FIN05	Reference pavement, low-quality base material loading mode	50 mm, Asphalt 250 mm, Crushed rock 1300 mm Sand	-	-	0	-	-	not started

APPENDIX 2

Results of Benkelman beam measurements, Test 02.

Measured deflection in the middle of test section Tyre pressure 800 kPa, Asphalt temperature +10 oC										
Date	1.7. 1997	10.7. 1997	9.9. 1997	19.9. 1997	29.9. 1997	8.10. 1997	15.10. 1997	21.10. 1997	29.10. 1997	3.11. 1997
Number of passes	35000	107000	865800	920850	1116918	1305000	1456875	1511730	1600000	1714436
load (kN)										
30	0.52	-	-	-	-	-	-	-	-	-
40	0.60	0.54	0.76	0.64	0.76	0.80	0.60	0.60	0.96	1.02
50	0.64	0.64	0.88	0.74	0.80	0.88	0.84	0.76	1.16	1.16
60	0.72	0.64	0.84	0.80	0.88	0.98	0.94	0.80	1.30	1.28
70	-	-	1.00	0.94	0.96	1.02	0.98	0.96	1.48	1.36
80	-	-	-	-	-	-	-	-	1.54	1.34

APPENDIX 3

Sensors basic levels in initial measurements in tests 01 and 02.

Test 01	Pavement temperature 10 °C, Speed 10 km/h, Side location 0, Tyre pressure 700 kPa, Super-single				
Load [kN] Sensor	030.0	040.0	050.0	060.0	070.0
A01 [uS]	-258	-261	-251	-249	-241
A02 [uS]	-241	-242	-235	-230	-223
A03 [uS]	-215	-220	-206	-210	-225
A06 [uS]	-226	-232	-233	-247	-255
A07 [uS]	-231	-236	-237	-254	-257
A08 [uS]	-250	-254	-251	-262	-268
A11 [uS]	338	333	306	291	277
A12 [uS]	384	378	358	346	337
A14 [uS]	353	348	325	306	298
A15 [uS]	465	450	414	403	389
A21 [uS]	253	246	234	230	224
A22 [uS]	257	254	252	251	249
A23 [uS]	194	190	178	178	173
A24 [uS]	198	190	176	169	159
A25 [uS]	297	293	275	280	273
A71 [kPa]	147	158	174	191	206
A72 [kPa]	346	365	394	425	448
A73 [kPa]	311	324	351	374	397
A91 [kPa]	120	130	149	169	189
A92 [kPa]	98	106	126	143	162
A93 [kPa]	104	112	130	146	164
A96 [kPa]	55	61	72	82	91
A97 [kPa]	39	43	53	61	70
A98 [kPa]	44	49	59	68	78

Test 02	Pavement temperature 10 °C, Speed 10 km/h, Side location 0, Tyre pressure 700 kPa, Super-single				
Load [kN] Sensor	030.0	040.0	050.0	060.0	070.0
B01 [uS]	-231	-242	-244	-245	-240
B02 [uS]	-222	-226	-226	-220	-213
B06 [uS]	-255	-271	-276	-288	-303
B07 [uS]	-267	-273	-278	-293	-307
B08 [uS]	-295	-307	-322	-339	-355
B14 [uS]	332	336	325	311	296
B15 [uS]	340	341	338	325	312
B21 [uS]	253	250	247	244	245
B22 [uS]	245	238	232	228	227
B23 [uS]	218	210	206	203	202
B25 [uS]	243	232	226	222	222
B71 [kPa]	168	182	205	219	229
B72 [kPa]	184	207	229	245	257
B73 [kPa]	146	166	183	195	204
B91 [kPa]	99	117	139	157	174
B92 [kPa]	89	106	126	143	158
B93 [kPa]	-11	-15	-17	-19	-21
B96 [kPa]	35	43	53	62	70
B97 [kPa]	39	48	59	69	78
B98 [kPa]	44	55	69	80	90
BB1 [uS]	419	415	406	380	349
BP4 [uS]	275	267	262	259	258
BT1 [um/m]	633	735	829	906	978
BX3 [uS]	-207	-218	-221	-218	-214

APPENDIX 4

Sensors basic levels in initial measurements in tests 03 and 04.

Test 03	Pavement temperature 10 °C, Speed 10 km/h, Side location 0, Tyre pressure 700 kPa, Super-single				
Load [kN] Sensor	030.0	040.0	050.0	060.0	070.0
C02 [µS]	-172	-175	-178	-173	-173
C03 [µS]	-206	-215	-214	-213	-215
C11 [µS]	227	215	200	187	182
C12 [µS]	176	150	137	129	122
C13 [µS]	334	350	355	344	338
C14 [µS]	272	277	277	266	255
C21 [µS]	159	136	137	122	132
C22 [µS]	208	188	190	172	185
C23 [µS]	110	94	97	86	103
C71 [kPa]	378	430	468	497	536
C72 [kPa]	393	446	497	528	568
C73 [kPa]	167	197	230	255	290
C91 [kPa]	84	107	127	146	177
C92 [kPa]	80	102	121	139	164
C93 [kPa]	81	102	125	145	174
CT1 [µm/m]	898	883	945	940	1029

Test 04	Pavement temperature 10 °C, Speed 10 km/h, Side location 0, Tyre pressure 700 kPa, Super-single		
Load [kN] Sensor	050.0	060.0	070.0
D01 [µS]	-109	-107	-99
D02 [µS]	-105	-101	-96
D03 [µS]	-122	-120	-116
D11 [µS]	143	130	125
D13 [µS]	170	158	154
D14 [µS]	241	227	220
D71 [kPa]	343	371	388
D72 [kPa]	415	440	456
D73 [kPa]	381	413	434
D91 [kPa]	120	138	151
D92 [kPa]	100	114	124
D93 [kPa]	150	172	189
D96 [kPa]	50	60	66
D97 [kPa]	50	59	65
DD2 [µS]	-136	-139	-140
DT1 [µm/m]	1201	1311	1398