Long Term Bearing Capacity on a Secondary Highway

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ABSTRACT: Since 1993 county road W850 in Central Sweden has been subjected to falling weight deflectometer measurements on an annual basis. Road surface characteristics measurements were also carried out twice per year to monitor among other things seasonal variation. Other data collected include deflections from a fast rolling deflectometer, georadar data, material sampling from coring and visual inspections. The research from these data includes initial and permanent deformation determination; frost heave influence on roughness; and several follow ups on overlays and other maintenance actions. Two reports have been presented previous BCRRA conferences and have been published in the proceedings. The present paper is a synthesis of the vast data collected. First and foremost the long term change of the bearing capacity is presented. A comparison is done to the functional properties, collected by the surface characteristics tester. Then, some conclusions are made on the maintenance and rehabilitation efforts, which were decided by traditional means. For instance if bearing capacity data are used diligently before maintenance actions are taken, the deterioration rate could be lessened. If not, poor sections continue to perform poorly even after rather expensive actions. Overall, the as built condition plays an important part of the road performance even after fifty years after the construction date. Bearing capacity measurements are good investments also for the lesser categories of roads.

KEY WORDS: Rutting, deformation, roughness, unbound base, subbase, subgrade, gradation

1 INTRODUCTION

Spanning almost two decades, from 1993 to 2011 county road W850 in Central Sweden was subjected to frequent falling weight deflectometer measurements. Road surface characteristics measurements were carried out twice per year to monitor the effects of spring thaw and pavement deterioration. Other data assessed include deflections from a traffic-speed rolling deflectometer, ground penetrating radar data, material sampling from coring and visual inspections. The research from these data includes initial and permanent deformation determination; frost heave influence on roughness; and several follow up studies on overlays and other maintenance actions. Two reports have been published and presented at two previous BCRRA conferences and several university diploma works have been done based on the data, (Lenngren and Fredriksson, 1998) and (Lenngren and Fredriksson, 2002). Further, several courses in continuing education about road maintenance have been carried out with the data. The fine performance of bitumen grouted macadam also inspired the making of a new product based on this concept. The monitoring project received funding up until 2011. The present paper is the first to review the entire measuring period. Students and anyone interested to use the data for research projects are welcome to contact the authors. Note also that the data collected were not used to influence the maintenance actions taken during the period. Thus, they are independent from the decision making process.

2 BACKGROUND

Most long term pavement performance monitoring occurs on medium to high volume roads. However, the secondary county road W850, a 26 km long test road has been thoroughly examined by laser surface profilers and falling weight deflectometers many times over for almost twenty years. The site is located in central inland Sweden, at latitude 61 degrees North, see Figure 1. Winters do get cold with an average freezing index of about 1000 degree days Celsius. (1960-1990 data). The average annual daily traffic (AADT) is 2600 of which eleven percent are trucks on this two-lane road. Much traffic is generated by daily commuting from nearby towns to the regional capital Falun and there are about 35 buses on schedule every day. Quite a large proportion of the truck traffic is heavy, carrying timber to saw mills in the area. The width of the carriageway is varying from seven to nine meters and the maximum speed limit was originally set to 90 km/h, but was lowered to 80 km/h in 2011.

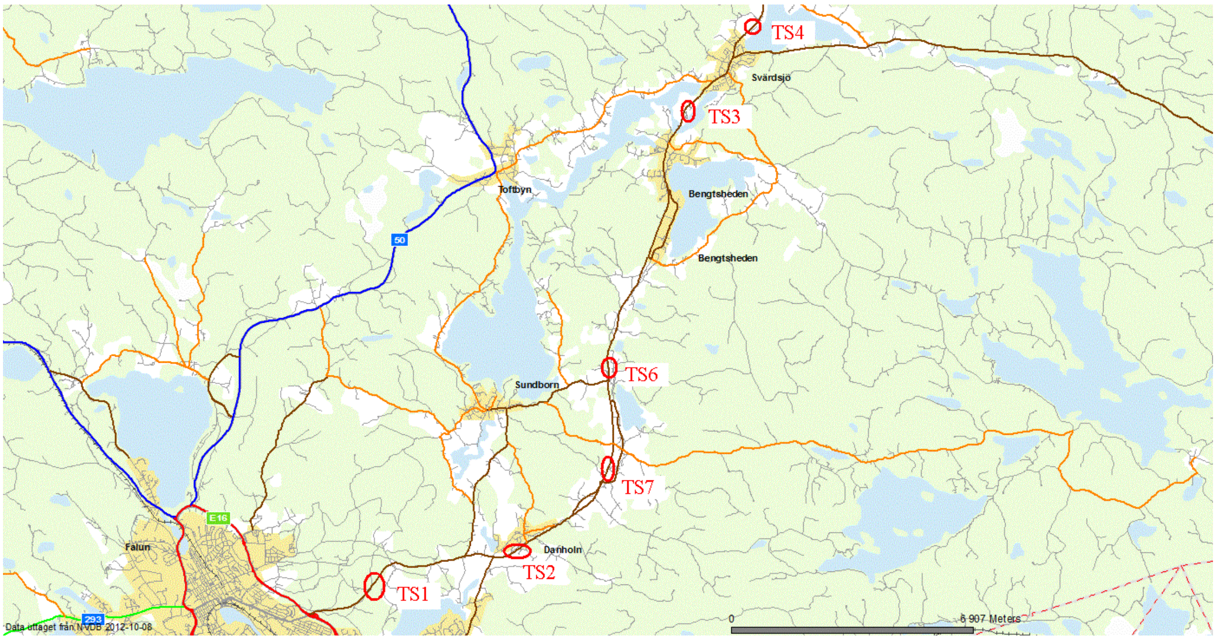

Figure 1: The test road is in the vicinity of regional center Falun

Surface characteristics testing was performed over the entire length of the road while FWD testing took place at designated test sections (TS), see Figure 1. Rolling deflection testers, ground penetrating radar and other test equipment have also been visiting the site occasionally. Thus, plenty of background material was at hand when some parts of the road were reconstructed in 1996. At TS 2 and 3 the rehabilitation included complete excavation of the roadway, insertion of an insulation layer, and a complete backfill with new granular material. As it were, the monitoring rut depth growth on these sections contributed to the understanding of early rut growth, (Lenngren and Fredriksson, 2002)

All in all, there were three sites that were excavated and backfilled with new material in 1996. Two of them were 50-meter long sections each on a strong support gravel subgrade interspersed with some silt lenses. There was also an 86-meter long section, on a clay subgrade.

The sections were originally constructed around 1960. All of them were subjected to uneven frost heave in the past, hence the call for removal of the in-place material. To reduce frost penetrating into the subgrade the new reconstructed sections were furnished with an insulation layer placed under the new backfill. The structures of the reconstructed sections are listed in Table 1.

Table 1: Reconstructed Road Sites Properties

Material	TS 2	TS 3
Asphalt Concrete Layers	Thickness 140 mm	Thickness 120 mm
Unbound Gravel Base	Thickness 700 mm	Thickness 500 mm
Frost Insulation Layer	Thickness 50 mm	Thickness 50 mm
Subgrade Material	Moraine Clay	Coarse Gravel Subgrade, with some silt pockets
Annual Average Daily Traffic	2620	2540
Road Width	7.4 m	8.0 m

TS 1 was originally constructed in 1984, TS 4 around 1960 and TS 6 and 7 around 1968. The structures of the common maintained sections are shown in Table 2.

Table 2: As built and maintained Road Sites Properties 1996-2009

Material	TS 1	TS 4	TS 6	TS 7
Asphalt Concrete Layer Thickness	125 -145mm	160-210 mm	165 mm	180-300 mm
Unbound Gravel Base Thickness	685 mm	540 mm	600 mm	600 mm
Subgrade Material	Moraine	Silt	Coarse Gravel	Silt
Annual Average Daily Traffic	3800	2100	2540	2540
Road Width	7.5 m	11 m	9,2 m	9,2 m

Material testing was done at the test sections in 1993, including the subgrade material. It consisted of fine soils at TS 2, 4, and 7, and coarse material at TS 1, 3, and 6. Intermittently some coarse soils were found at TS 2. TS 3 and 6 are cuts through an esker with gravel, but there are thin sediments of silt within the esker.

The subbase material is not up to par with current design code. The section built in 1984 is not even fulfilling the requirements at the time of construction. Typically, the material used consists of unsorted natural gravel and sand, with a smaller maximum particle size common today. The subbase at TS 4 has a lower layer of gravely sand and an upper part with coarse gravel. This is commonly found in roads built in the 1960:ies. Today, the subbase material preferred consists of crushed rock, providing a much better stability.

Unbound base courses containing too much fine material is susceptible to water. This is a prevailing situation for county road W850, except TS2. Typically, there is too much sand in the base course not providing shear resistance when subjected to high loads. At TS 3, 4, 6, and 7 there are some coarser fractions providing a varying strength.

Parts constructed in the 1950:ies and early 60:ies often had bituminous chip seals or alternatively bitumen bound macadam base layers, e.g. TS 2, 3, and 4. The latter material allowed construction traffic on the base course, which was the primary reason for using it. The bearing capacity was soon found to be improved also. Later on different hot mixes were used for the asphalt base as found in TS 1, 6, and 7. The 1995 reconstruction of parts of TS2 also incorporated a hot mix asphalt base. As by now the surface layer consists of a bituminous surface dressing in its entire length..

3 FIELD TESTING

As mentioned above the road has been measured on an annual basis, but there are years when the test occurred more often. As the area has quite cold winters, typically the surface tester usually measured before spring thaw and after the frost heave had settled in early summer. Just a month after the reconstructed parts had been opened for traffic the test road was measured with surface profiling equipment and a FWD in September 1996. The entire road was adjusted in 1996 and a new wearing course of single bituminous surface treatment was put on top of adjusted surface in the summer of 1997. The test equipment rendezvoused on the site for follow-ups of those activities. More tests were done after a year in traffic and later on measurements were made on an annual basis. By the end of 2011 the surface tester has made no less than 32 visits to the site and the FWD has visited eight times.

3.1 Surface Characteristics Measurements

The first surface characteristics visit occurred in 1992 by the VTI (Swedish Road and Transportation Research Institute) Laser Road Surface Tester, (L-RST). Originally, eleven sensors were used to make up the transverse profile. However, when dealing with shallow rutting, it is a good idea to employ as many sensors as possible. Miniscule rutting is difficult to detect, and driver variability is reduced the more sensors there are. Since 1997 and onward the vehicle was furnished with seventeen sensors for a better coverage and less operator error influence. With seventeen sensors it is still possible to make an eleven sensor evaluation. A regression between 11 and 17 sensors deemed the adjustment from 11 to 17 sensors to be within half a millimeter. The present report uses the higher number of sensors throughout.

All tests were repeated at least once. The crew used was trained for research work, and had over ten years of experience of this type of mission. The repeatability was very good for the sites investigated, except for two occasions. As the faulty runs were obvious they were excluded accordingly. The same driver was used at all times as to minimize the influence of driver idiosyncrasies.

In Figure 2 below the rutting is shown over the years. The average rut depth is about 11 mm in 1996 before the maintenance and reconstruction started. Rehabilitation with leveling layers was done 1995-1996, a new wearing course was laid 1997-07 and further leveling were done 2003-08 and 2008-09. As suggested by the 95-percentile line the variation along the road is large. The last maintenance prior to the test measurements was done in 1981 and the first 2.7 km was built in a new alignment in 1984, i.e. eight years before the test started.

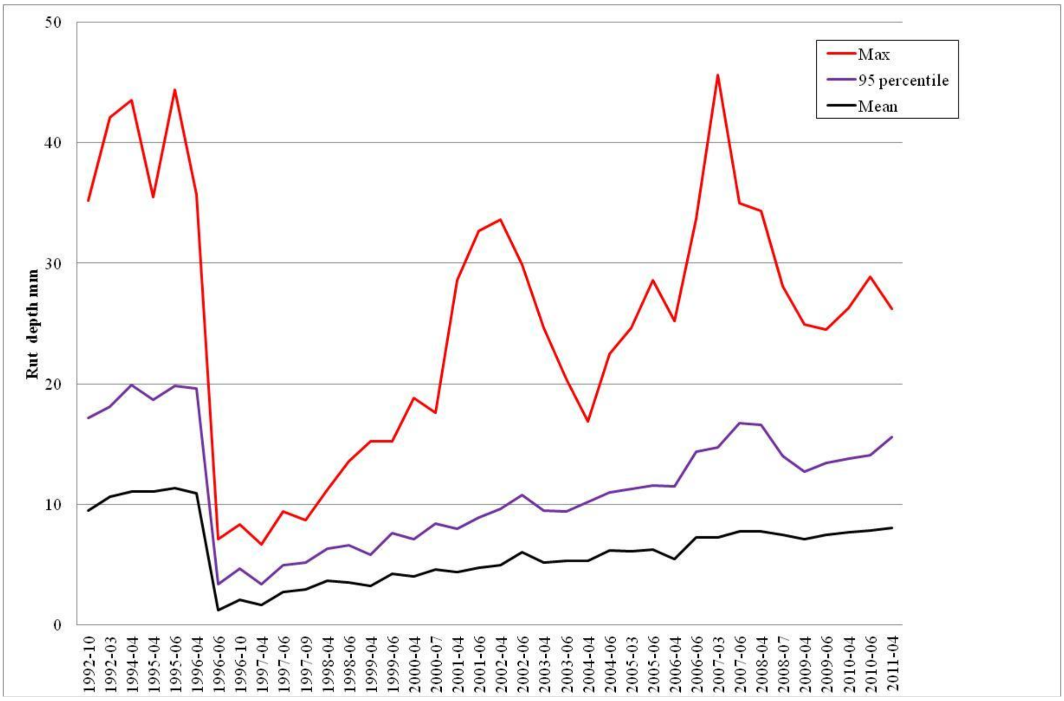

Figure 2: Rut development road 850 Falun – Svärdsjö 1992 – 2011. Routine maintenance occurred during 2003 and 2008.

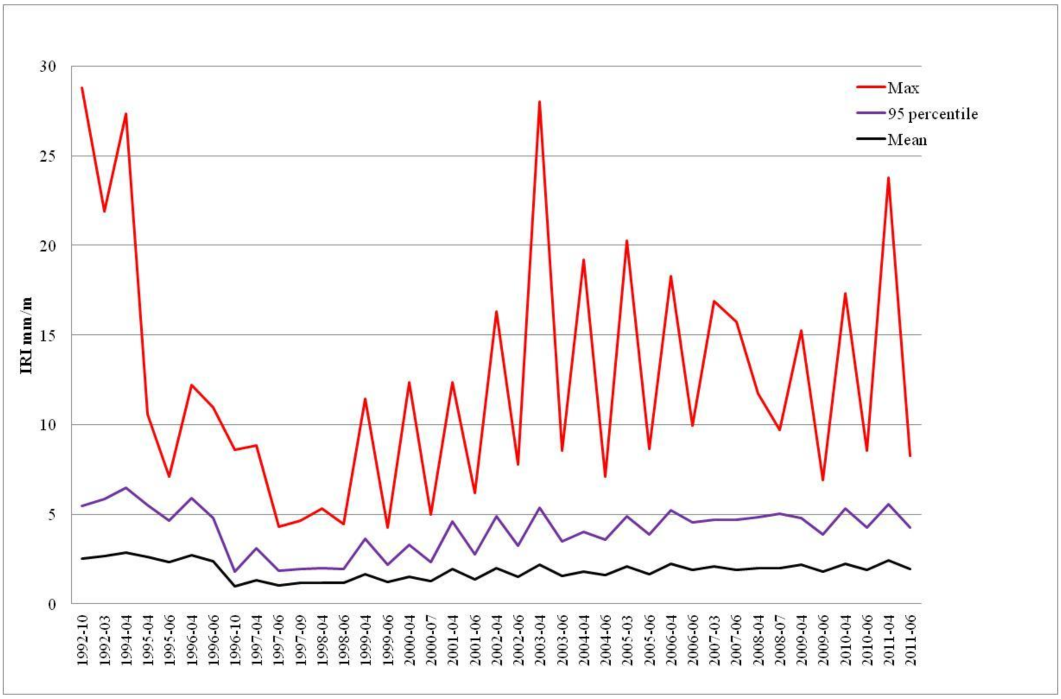

Figure 3: Roughness development road 850 Falun – Svärdsjö 1992 – 2011

In lieu of increasing traffic the deterioration rate is decreasing. A trend observed, not to be uncommon for this category of road. The improved bearing capacity is the primary reason. At the time of the design there was also a general overestimation of the distresses caused by heavy trucks. The design rutting rate is 0.8 mm per annum in addition to a 3 mm initial rut depth. (Older design assumed a 1 mm/year rate).

The roughness has a direct influence on the user costs and traffic safety, (VTI 2002). Figure 3 shows the International Roughness Index (IRI) over time in the northbound direction. The average is 2.7 mm/m before 1996, but the 95-percentile is 5.9 mm/m, in effect meaning that five percent has a roughness higher than 5.9. Note that the seasonal variation is very high for the maximum values as well as for the 95-percentile. The roughness influence on traffic safety is thus much higher during the winter months, also considering that the friction is lower and daylight period is shorter. The 2010 seasonal variation is listed in Table 3.

Regarding the rut depth and IRI as well TS 6 exhibits the best performance. The roughest section is TS 2 while TS 7 shows the largest difference between directions. TS 1, 2, 4, and 7 were subjected to maintenance actions between 1996 and 2010.

Table 3: Rut mm and IRI mm/m, section 1,2,3,4,6 and 7 spring and summer.

Test Section	Quality	Rut mm				IRI mm/m			
		Northbound		Southbound		Northbound		Southbound	
		April 2010	June 2010	April 2010	June 2010	April 2010	June 2010	April 2010	June 2010
1	Mean	12,8	12,2	11,4	10,8	1,9	1,5	1,7	1,5
	95 %	16,9	15,0	15,7	14,3	3,3	2,3	2,5	2,1
2	Mean	8,8	9,5	10,3	11,6	3,3	2,4	4,1	3,2
	95 %	12,8	14,3	14,2	17,8	5,8	4,2	7,0	4,7
3	Mean	9,4	9,7	9,6	9,2	1,4	1,4	2,0	2,2
	95 %	11,7	12,6	13,1	13,3	2,2	2,2	3,8	3,7
4	Mean	6,3	6,7	5,5	6,7	3,4	2,3	3,7	2,8
	95 %	9,2	9,3	7,3	9,1	6,2	4,2	7,2	5,0
6	Mean	5,9	5,5	6,3	5,8	0,9	0,9	1,2	1,2
	95 %	7,3	7,0	7,4	7,2	1,3	1,2	2,0	2,2
7	Mean	5,6	6,1	9,1	10,6	1,8	1,4	2,4	2,0
	95 %	7,0	7,8	15,2	18,4	3,1	2,3	5,6	4,3

3.2 FWD testing

The FWD testing was synchronized length-wise with the L-RST data. FWD tests were made in the middle of each 20-meter long sampling range cell in the northbound direction. The southbound tests occurred just across the road, i.e. the same sections. Thus, southbound tests were not staggered, which is otherwise a common practice. L-RST cells did not match the southbound tests perfectly as the shift between cells depended on the location of the starting point. The shifts coincide only if the length of the link in question is a multiple of 20m. Figure 4 shows the average center deflection D0 for each TS in the spring of 1995, the fall of 1996, the Spring of 2009 and 2011. Figure 5 shows the deflection at 120 cm from the loading plate D120 reflecting the deformation in the subgrade.

3.3 Backcalculation of E-moduli.

The linear elastic backcalculation program CLEVERCALC versions 3.8, 3.9 and 4.0 were used for determining the E-moduli. The version used depended on the year of testing. Usually, three load levels were used and the drops were repeated twice. Hence, stress sensitivity prevailing in the materials involved could be assessed.

The backcalculated E-moduli for the asphalt concrete does not vary much over time, except for the effect of temperature. Most test basins were near 6000 MPa for a temperature-adjusted modulus to 15 degrees Celsius. The base and subbase moduli vary with the season. This variation is far larger than any long-time effects. For TS 2 the base exhibits the lowest values 60 MPa at spring thaw and the highest values are around 200 MPa in the fall. The subgrade also varied within the section being 50 MPa for most of the section 2, the far end actually was stiffer. Typical values for TS 3 were 200 MPa for the subgrade and 300 to 400 MPa for the base layers with less variation over the seasons, but also within the sections.

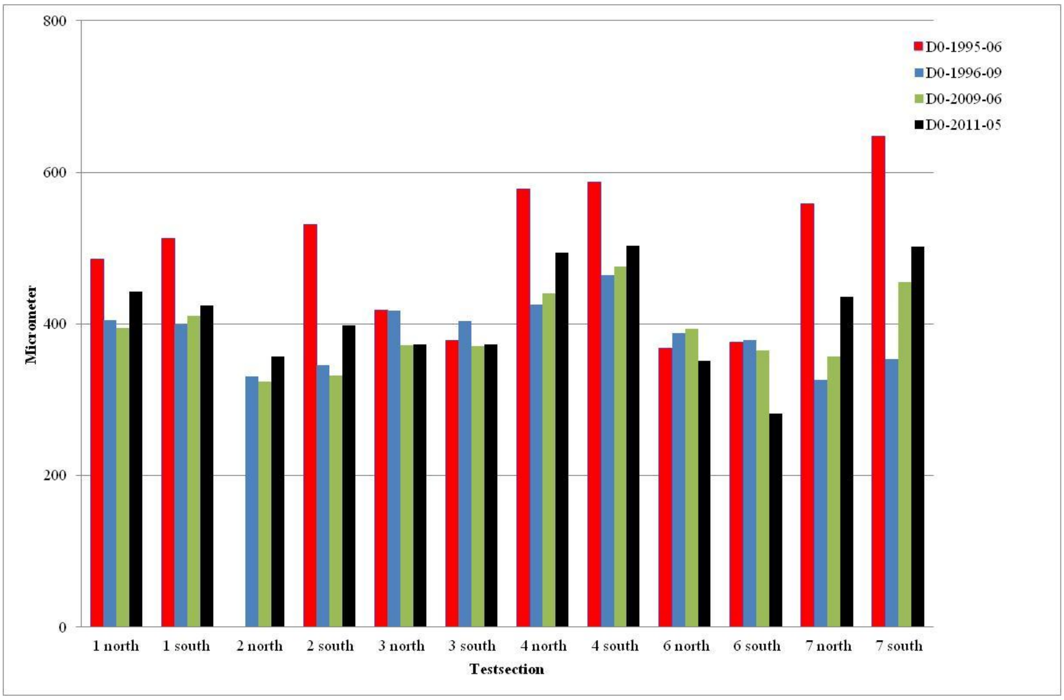

Figure 4: Deflections (D0) road W850 North- and southbound 1995, 1996, 2009 and 2011

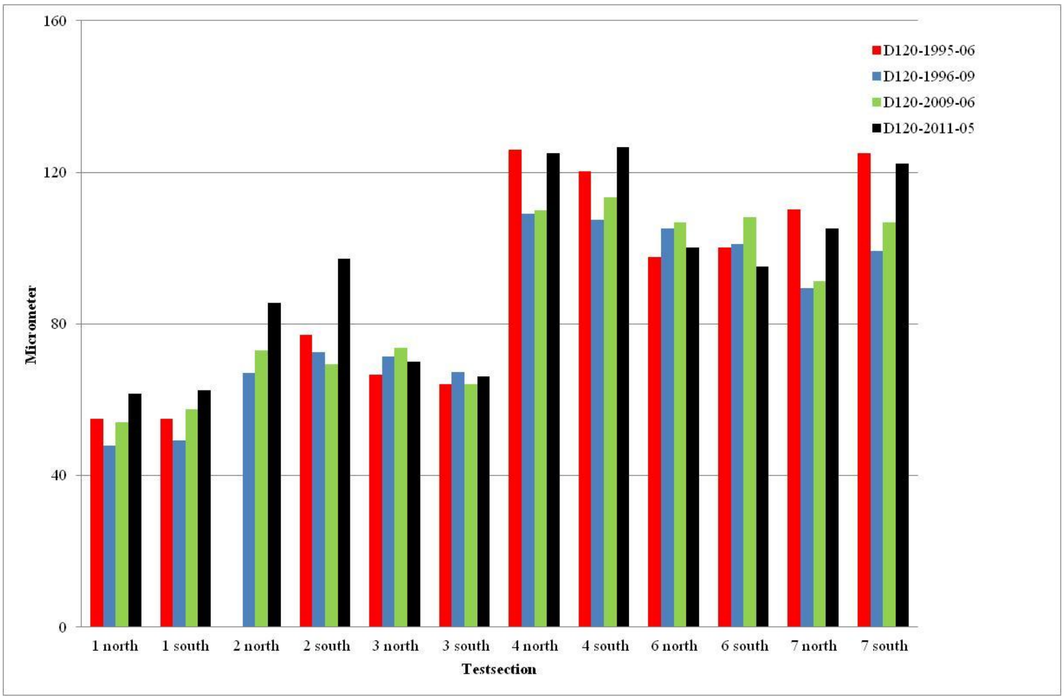

Figure5: Deflections (D120) road W850 North- and southbound 1995, 1996, 2009 and 2011.

Table 5 lists comments regarding the results for the test sections.

Table 5: Comments for the various Test Sections

TS	Comment
1	Maintenance actions did not affect the deflections much.
2	This section exhibits great variability along the section and between testing occasions. The soils here are susceptible to moisture content. The relatively large deflections in 2011are due to water entering the subgrade through a new (faulty) drainage system.
3	The largest deflections are found where the excavation and replacement took place in 1995. This action and the frost insulation for section 1680 – 1740 (northbound) were obviously not necessary and rather counterproductive.
4	Poor subgrade support and a high flow of water, which is not drained, are the reasons for high deflections. The bearing capacity variability did not decrease in spite of the chosen maintenance actions.
6	The deflection has not changed much over the years. The subgrade consists of coarse esker material, providing excellent bearing capacity. A short stretch consists of silt, which indeed cause high deflections even the average is only affected marginally.
7	The deflections during spring thaw 2011 are much higher than 2009. The timing was such that the first fully thawed day was measured in 2011, resulting in extremely high deflections that day.

3.4 Rolling Resistance

Damping in the pavement layers was assessed by using FWD time histories and plotting a load-deflection hysteresis loop, see Figure 6. From a rolling resistance test with different pavement materials and fuel consumption measurements the work from the loop could be estimated, (Lenngren 2009). In this reference the method is described and the FWD used in 2009 and 2011 was dynamically calibrated for this purpose in mind.

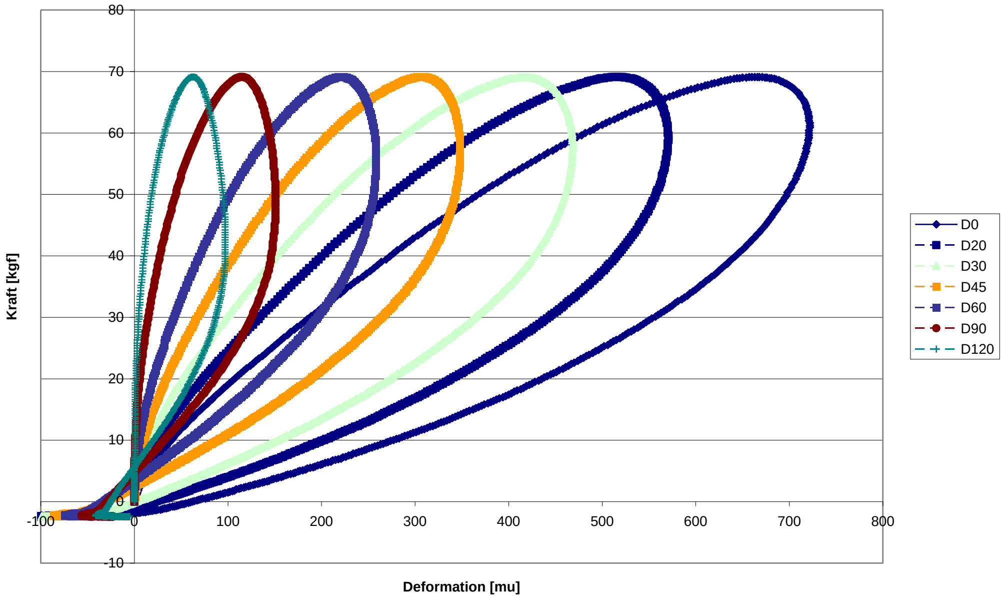

Figure 6: Load deflection diagram hysteresis curves from TS6.

One may surmise the rolling resistance is decreasing as time goes by as more of the unbound materials settle. Further, the hardening of the asphalt concrete is generally reducing the loop, in effect making the response more elastic. However, as bound layers start to fatigue and crack, the stress on the unbound layers becomes higher and more energy is needed to compact these layers before they settle. A “new” initial rutting phase occurs and it could also be detected by the fact that the stress sensitivity regression is poor, (Hansson and Lenngren, 2006). When comparing the tests from 2009 with 2011, most of the latter tests displays a lower rolling resistance, except those sections were cracks appeared and those with a higher water content mentioned above.

4 FINDINGS FROM THE STUDIES

Over the years, several reports have been made, many as university diploma works. Many employ regression techniques, e.g. between stress and rutting. The vast amount of data collected during almost two decades cannot be presented in full in the present context, but some results are presented below.

Overall the predominant reasons for distress types like rut depth variation, rut shape, roughness, pavement cracks, hysteresis curves are fine subgrade soils in relation to water. The water is present either as high ground water level, or entering from the side coupled with poor drainage.

4.1 General Findings

High volume roads are intrinsically thicker and have a wider margin for variability. Thus, the deterioration correlates well with traffic. Low volume roads however tend to deteriorate due to the environment, i.e. the subgrade type and the climate. In the former case, structural rutting is a good indicator of the asset value. In the latter case roughness is the predominant distress parameter. It can be related to a number of distress types. The present road is neither a high volume nor a low volume road per se, but the traffic of about 300 ESAL:s per day is large enough to correlate with rutting, but other distress types are dominating. So subgrade type and climate factors like freeze/thaw cycles are the prime reasons for the deterioration. Lack of proper maintenance and faulty maintenance accelerates the deterioration.

4.2 Climate Related Findings

The deterioration rate is higher on frost susceptible subgrade classes in spite of measures called in the design code. The freezing index, spring thaw, rapid temperature change, subgrade type, and drainage all contribute more to the deterioration than traffic. The freezing index, spring thaw, rapid temperature change, subgrade type, and drainage all initiate and accelerate the deterioration.

4.3 Findings about Rutting

A thicker asphalt concrete layer decreases the deterioration caused by traffic. Excavation and replacement of new material will often render a rather high initial rut, or first year rutting. The initial rutting varies a lot from section to section, but the rutting rate seems to be about 0.8 mm per year after the first year. Frost heave may affect the rut depth measurements to temporarily lower values. This effect is gone by the end of the summer.

4.4 Findings Related to Rolling Resistance

Prevailing water in the road and subgrade contributes to the rolling resistance. There is no linear relationship by the load and rolling resistance. The higher loads affect much more subgrade soil, which are prone to damping when moist. Road contribution to rolling resistance for passenger cars is a mere function of texture and pavement-tire interaction. The rolling resistance tends to decrease somewhat with time. For bitumen bound materials the visco-elastic property decreases with time. For unbound materials the initial deformation settles. Both these circumstances will reduce the hysteresis loop. If cracks have propagated through the bound layers the higher stress level in the unbound materials initiate more deformation and consequently a higher rolling resistance. Higher water contents in unbound materials will also increase the rolling resistance; this may be caused by the higher deformation rate.

5 CONCLUSIONS AND RECOMMENDATIONS

The long term pavement performance monitoring has shown that the design of maintenance projects is much more important than previously believed and practiced. Some of the solutions chosen have either accelerated the deterioration and/or lead to higher operations costs. These could have been avoided if proper bearing capacity tests and analyses had been made. For mid to low volume roads the environment and climate and unbound materials as well become much more important in the design process. There are more parameters to deal with than for high volume roads, but the complexity is manageable. Also, by using the dynamic response data, much of the detrimental effects of water can be identified and dealt with. Finally, the long term follow up contributes to the understanding of the deterioration process, the processes of the load dynamics and what is finally showing up on the surface.

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