Implications of Changing the Maximum Legal Truck Load for The Pavement Service Life

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Implications of changing the maximum legal truck load for the pavement service life

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ABSTRACT: Some major roads in Sweden are being designed to carry a higher vehicle load of 74 metric tons. This is almost 25% higher than the previous maximum legal load. There is no change of the maximum 10 ton axle load, and the 11.5 ton driving axle. Thus, in the elastic world, the change would not incur any changes in the pavement design, as the number of equivalent axle loads will remain the same, provided the freight levels are the same. However, the experience from a previous change of the legal loads showed clearly that sub base layers tended to deform at a faster rate. Hence, the specifications for sub base layer materials were changed with tougher criteria for the gradation curve. The present paper compiles experience from a failed mining truck road to address issues that may affect specifications for unbound materials.

1 INTRODUCTION

The legal limits e.g. truck axle loads is governed by the vehicle design, which is in turn decided on a common standard. Thus, bridges and roads can be optimized to such norms.

Every so often, there is a demand from the industry to raise the standard to a higher level. The reason is often from an economic aspect, or it may be driven simply by bigger trucks, being available on foreign markets. One major concern is the bridge design, which is optimized for a certain maximum load limit that cannot be changed without major investments. For pavement design, there is a general rule of the number of equivalent axle loads, which is widely used. Thus, an increase of loads could be calculated as accelerated wear and tear per vehicle.

In the 1980:ies, with an effort to harmonize European transportation standards, there was a change of the truck axle loads and wheel configurations. In Sweden, the gross weight was not affected by this. However, there were other changes like allowing 11.5 tons on the driving axle, and using wide tires, and three axle combinations with higher loads than previously. Within a few years, all bridges were either replaced or rebuilt on all the national roads. There was an overview of the design classes for pavements. In essence, there was a shift to a higher class for each traffic category, and some of the lower volume classes were abandoned altogether.

For a while, there were also strengthening projects going on, particularly on the smaller roads. More importantly, the rather generous maximum gross weight was increased from about 51 tons to 60 tons for truck and trailer combinations. The maximum axle load remained at ten tons for non-driving axles. The change went rather smoothly, and the trucking industry quickly phased out older three-axle trailers to four axle ones, which was one of the tax incentives.

However, some newer roads suffered from excessive rutting. It turned out the most of this rutting occurred rather deep in the sub base layer, and that those layers consisted of sand. Thus, the pavement specifications were changed to favor crushed materials, which also became mandatory in the higher traffic classes. Even if scarcity of materials, which increased costs, the stricter specifications solved the deterioration problem.

Now, about thirty years later there is a proposal to increase the maximum gross weight to 74 tons, without increasing the axle load. The rational being more payload in less number of vehicles. Trucks would be longer, but highways have improved, so the need for passenger cars to pass in a lane with opposing traffic is much less frequent today. A check with proposed axle spacing on bridges has been approved. A typical 74-ton combination would consist of nine axles. 8 + 11.5T + 3x18T perhaps. A four-axle trailer 2x18T could be replaced by a five-axle ditto carrying 18 tons on a tandem and 22 tons on a triple combination.

In a linear elastic model, there is no difference in pavement wear from the proposal, as the number of axle passes would remain constant. In reality, the rest period between loads is shorter, which would likely increase fatigue damage, and rutting as well. In the Swedish Pavement Design Code (2011), there is uncertainty risk factor, ranging from 1.0 to 1.15 depending on the road category. With the proposed new gross weight, a factor of 1.25 should be used instead, to account for the extra wear and tear. However, with the previous experience in mind it may not be so easy as to factor in a number. Rather, there is an obvious risk for exceeding the strength of some unbound materials.

2 INITIATIVES FROM OTHER COUNTRIES

According to McKinnon (2005) and based on UK data, there are advantages of rising the maximum truck weight on traffic levels, road haulage costs and emissions. The study took account of three key factors: the migration of loads to heavier vehicles, a traffic generation effect and the diversion of freight from the rail network. The net reduction in trucking kilo meters by 2003 was at the upper end of the forecast range. A comparison with the issue in the United States was also provided.

In 2013 Finland allowed the maximum weight limit of trucks increased up to 76 tons, although Finland, like Sweden, already differed from most EU countries by using EU's exemption to allow longer and heavier trucks to operate in national roads. The Finnish government hopes to achieve both economic and environmental benefits in the road freight sector by raising the total load, but at the same time, Finnish road infrastructure has been reported to be in poor condition. As a result, the Finnish haulers adopted higher maximum payloads mostly over a period of one year. In 2015, the cost savings of HCT trucks were about 58 million euro and CO_2 savings about 0.07 Mt or 3.7% of total truck CO_2 emissions in Finland. These savings are significant, but if the savings will not increase in the future, they will remain significantly lower than was estimated prior to the change (Liimatainen & Nykänen 2014).

Increasing the maximum truck-trailer load has also implications for bridges. A formula for limiting truck and combination vehicle weights is proposed by James and others (James *et al*, 1986). The proposed formula gives the maximum allowable gross vehicle weight as a function of extreme axle spacing. It is intended to replace the existing formula, which depends on the number of axles in the string as well as the extreme axle spacing. According to the authors, there are several benefits of the proposed formula for the bridges protection.

A hint may be to examine a mining truck road that was exposed to exempted heavy loads, before construction was completed. It failed only a few weeks after it was first used, and the data from FWD tests and sampling are used to find a solution to the failure.

3 OBJECTIVE

The objective with the present paper is to find out if there are other concerns of pavement design criteria than the traditional top of the subgrade strain and the maximum strain in the bound layers. In this context, the concern is the unbound base and sub base layers only. Typically, yield criteria for soil can be used as Mohr-Coulomb, shake down limits *et cetera*.

4 HIGH LOAD PAVEMENT DAMAGE

The change of the maximum gross weight is being introduced without any thorough full scale testing. It is assumed that the elastic model is appropriate for this purpose, and as an extra precaution, a fudge factor is introduced in the design. However, one can get experience from similar situations with high loads on industrial pavements, harbors and similar facilities.

One striking example of load related pavement damage is a 20 km long section of Highway 99 in Northern Sweden. It was being reconstructed in 2015 to accommodate mining truck trailer combinations weighing as much as 90 metric tons. In this remote area, traffic is sparse, so the new design called for a substantial increase in bearing capacity. It is in a cold region and the frost index amounts to 1500 degree-days Celsius, according to 1960–1990 climate data.

The design called for six years of 20-ton axle loads, corresponding to six million 10-ton standard axle loads. The design bound layer thickness design was 150 mm, but traffic was permitted with only 50 mm of asphalt concrete in place. Soon enough, the deterioration became evident, and an independent investigation was initiated. It was divided into two parts being 8.4 and 11 km long respectively.

5 FIELD TESTING

5.1 *Testing*

During 2015 several tests were done:

- Ground Penetrating Radar (GPR)
- Visual Survey with Frost Damage Inspection

- Falling Weight Deflectometer Testing (FWD)
- 300 mm Bore Coring
- Two Full Width Forensic Cuts

5.1.1 *Ground Penetrating Radar*

The GPR tests were used to confirm the asphalt layer thicknesses. At the time of measurement, only one 50 mm thick layer was placed. It was found to vary from 30 to 80 mm. About 10% of the entire length was recorded as less than 50 mm.

5.1.2 *Visual survey with frost damage inspection*

The visual survey noted four potholes, and some 10–30 mm wide cracks. There were five incidences of fatigue cracking and one settlement. Severe rutting was found at three locations. This inspection did not find any damage from frost actions.

5.1.3 *FWD testing*

A standard test method using the nominal load of 50 kN was used. In addition, time histories were saved to disc. The testing took place in June of 2015 during two days, and the pavement temperature was around 15 degrees Celsius. A total of 845 sections were tested.

5.1.4 *FWD back calculation and analysis*

A back calculation analysis was done with four layers. A thin 50 mm pavement resting on a 100 mm unbound base course and a 600 mm thick sub base. The GPR indicated some variability of the pavement layers. In order to compare strains at certain depths, the layer thicknesses were kept constant nonetheless.

The asphalt layer modulus was found to vary. From experience, this is quite common as a small variation of thickness affects the stress situation in the material. The focus here is on the unbound layers and the median moduli are shown in Table 1. As mentioned previously, the test is divided into two parts. E(2) refers to modulus of layer two counted from the surface, i.e. the unbound base. E(3) is the modulus of the sub base layer. E(4) is the sub grade modulus.

Table 1. Unbound Layer Moduli.

	Section	
Modulus	1	2
E(2) median	502	464
E(3) median	199	190
E(4) median	209	185
E(2) 10%-ile	170	256
E(3) 10%-ile	190	149
E(4) 10%-ile	185	144

Table 2. Sections ranked after low base stiffness (part 1).

Section	Modulus	Section	Modulus
7452	51	3652	86
7024	71	6974	87
3375	78	7375	94
3528	80	3652	94
3428	83	3326	97
7515	84	1075	99
7425	84	802	99

Table 3. Sections ranked after low base stiffness (part 2).

Section	Modulus	Section	Modulus
11103	53	13672	76
14896	56	7615	81
17923	64	13623	85
12572	71	17222	86
12672	72	14847	87
14896	73	17572	97
17522	76	13173	97

Table 4. Sections ranked after low subbase stiffness (part 1).

Section	Modulus	Section	Modulus
7616	84	7565	105
7641	90	7591	105
7606	98	6325	106
1550	100	8094	109
7743	100	7765	112
1600	102	6600	113
6325	103	6248	114
7692	103	8170	116

As can be seen the median values provide a rather high bearing capacity. However, there are quite a few exceptions. Table 2 and 3 show the base and Table 4 and 5 show the sub base layer modulus sections sorted from lowest ones.

Evidently, the median stiffness-values are quite good, likely a result of compaction from the heavy trucks. Nevertheless, a large portion of the length of the road lacks bearing capacity. It seems the materials lost their stability somehow. It could be that the materials failed once a certain level of stress is obtained.

Figure 1 shows the vertical strains in the middle of the base layer sorted from worst to best. Note that some stations are in tension. The corresponding sub base layer strain at the same testing points is shown as Strain(3). There is no good correlation between the two. The coefficient of determination R^2 is about 0.2. It is interesting to see that the

Table 5. Sections ranked after low subbase stiffness (part 2).

Section	Modulus	Section	Modulus
14972	83	16597	109
16822	92	14822	109
16796	93	16847	110
16796	95	16572	110
14772	98	20033	111
14872	100	10981	112
14920	105	17396	114
10981	105	15372	118

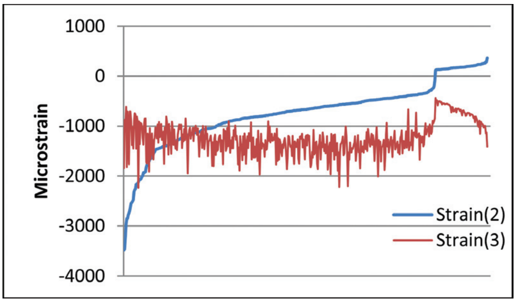

Figure 1. Strains in base (2) and subbase (3) layers, sorted after (2).

distribution is not linear. The shift into tension occurs where the pavement failed and the horizontal stress is larger than the vertical stress. More interesting is that the strains higher than 1000 micro strain in compression seem to have a wider spread, maybe because the materials start to deform.

Figure 2 shows the same plot, but now sorted after the sub base vertical strain. The confining pressure is much higher than for the base, so the "bend" at the left side of the curve occurs at a higher strain of about 1700 micro strain. At the right side the lower strains occur when the strain is higher in the base layer. This could mean that there is more dissipation higher up; energy that is used for post compaction.

From load-deflection charts, one may estimate the internal losses from the FWD and corresponding truckload. On the highway, it is usually in the range of 3–5 Nm on asphalt concrete roads. In this case, it is about 16 Nm at section 14972, where the sub base stiffness was only 83 MPa. The graphs from three sensors at this section is shown in Figure 3. Note that the area from the D_{30} sensor is almost as large as the D_0 sensor, which indicates that most dissipation occurs in the sub base layer here. The highest dissipation found was a whopping 44 Nm, where all layers deformed for a total surface displacement of almost three millimeters.

5.2 *Destructive testing*

Over 100 hundred bore core samples were made and the material was brought to the laboratory for sieving. As it turned out most of the samples had too much fines in them and the gradation curves were outside the permitted area. This did not compare with the constructer's self-quality control tests, which were on target. Assuming, same source material for both tests, the grains must have been subjected to mechanical wear. Micro-Deval and Los Angeles Drum Tests (LA) were taken from materials in the Base and Subbase, and there was quite some variability as shown on Table 6.

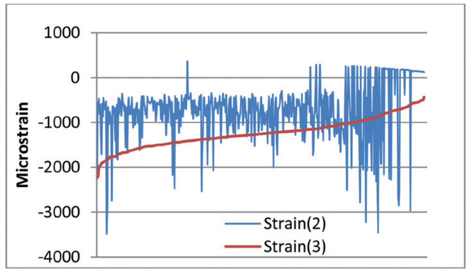

Figure 2. Strains in base (2) and subbase (3) layers, sorted after (3).

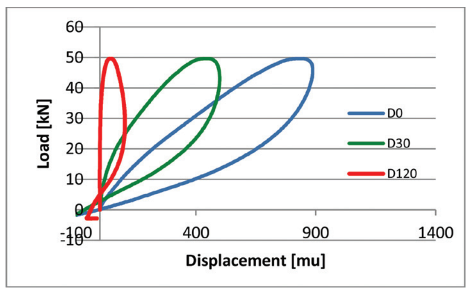

Figure 3. Load-deflection diagram indicates dissipation in the sub base.

Table 6. LA and micro-deval samples.

	Base layer			Subbase	
Section (m)	Micro-Deval %	LA %	Modulus (MPa)	LA %	Modulus (MPa)
2129	7	33	193		
3296	–	–	–	6	251
5762	15	23	178		
5826	–	–		6	188
6733	6	–	168		
8279	–	–	–	6	251
11066	10	18	298	15	170
13830	13	21	250	16	175
14210	11	20	214	14	255

The Micro-Deval values are required to be less than 20% and the LA values less than 40%. Most of these samples were taken on sites that had better than average values for the E-modulus. As the materials evidently had been worn down to finer gradation, one should reconsider the LA and Micro Deval limits for super heavy loads.

6 FINITE ELEMENT ANALYSIS

From the FWD testing we saw a great variability of the stresses in the unbound layers. We have to deal with on-going processes. To get a better picture of the normal stress, shear-stresses and displacements a Finite Element Modelling (FEM) was done for four cases, by considering two different structures, with the averages and second worst modulus, and two different loads, for the standard truck axle and a typical mining truck axle. The characteristics of such structures are shown in the Table 7 and the loads, in the Table 8. The two simulated axles have a separation of 3 meters and the configuration of the four loads (tires) can be seen in Figure 4 and Figure 5. Figure 5 also shows the rotation of grains under the load, something that could have contributed to the wear of the individual particles.

The Figure 6 through 17 are showing the results of the FEM for stress, shear stress and Y displacement. It is possible to see that there is no relevant overlap among the stress and shear-stress for all cases of structures and loads for the considered axles separation (3.00 m), however, there is some overlap for Y displacement, i.e., in the horizontal direction perpendicular to the traffic lane. Displacement in other directions were found as irrelevant for the purposes of this study.

Table 7. Pavement structure characteristics.

Layer	Thickness (mm)	Modulus (MPa) Average	Second worst
AC	50	10000	10000
Base (granular)	100	464	56
Sub-base (granular)	600	190	92
Subgrade	–	185	88

Table 8. Axle load characteristics.

Characteristics	Axle type Standard	Exempt
Number of axles	2	2
Axles separation (m)	3.00	3.00
Tires separation (mm)	350	350
Load per axle (kN)	100	180
Load per tire (kN)	25	45
Tire type	Regular	Exempt (non-off-road)
Tire pressure (kPa)	552	862
Tire pressure (PSI)	80	125
Tire radius (mm)	120	129

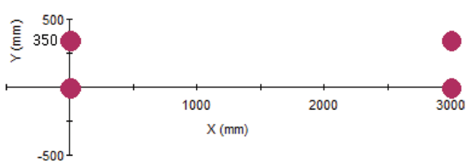

Figure 4. Tires position (load configuration).

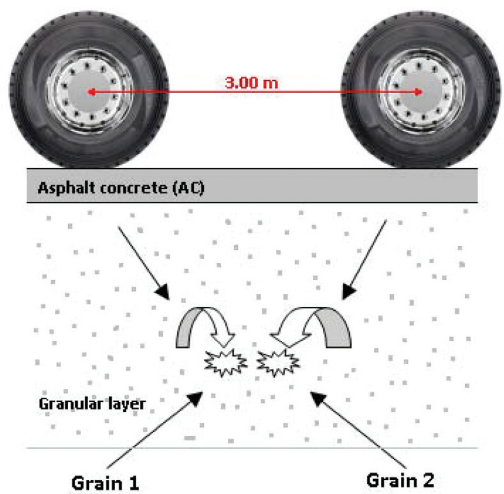

Figure 5. Grains rotation under the load.

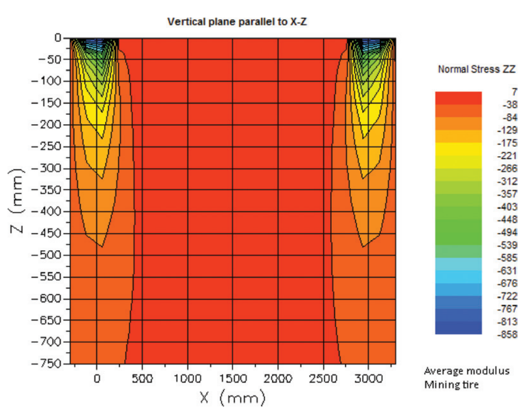

Figure 6. Normal stresses for structure with average modulus and mining tire load.

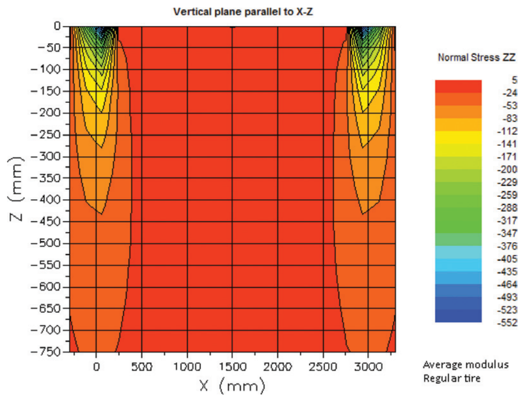

Figure 7. Normal stresses for structure with average modulus and regular tire load.

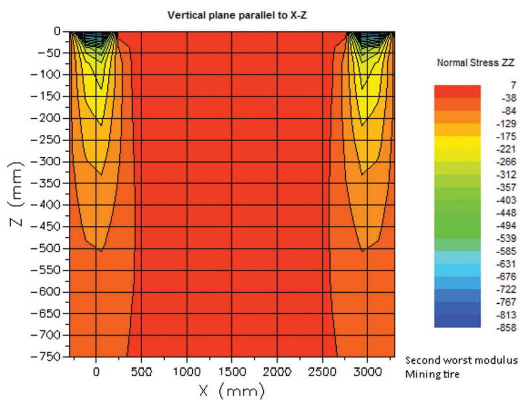

Figure 8. Normal stresses for structure with second worst modulus and mining tire load.

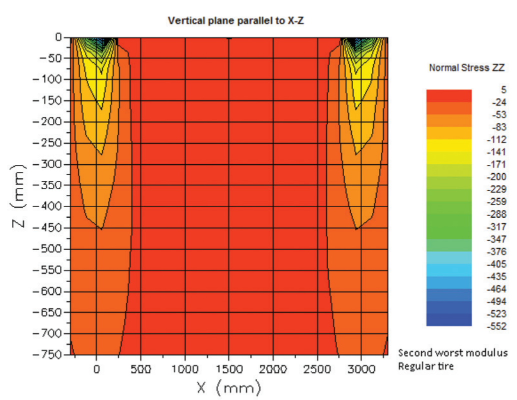

Figure 9. Normal stresses for structure with second worst modulus and regular tire load.

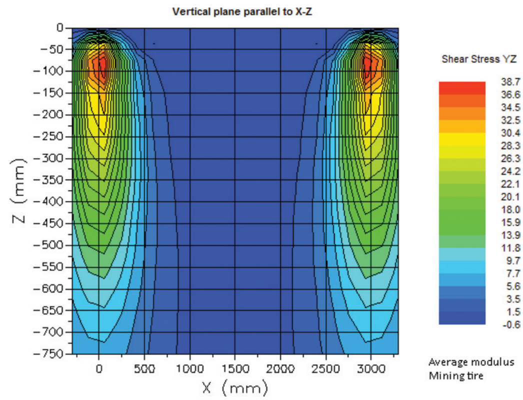

Figure 10. Shear-stress for average modulus structure and mining tire load.

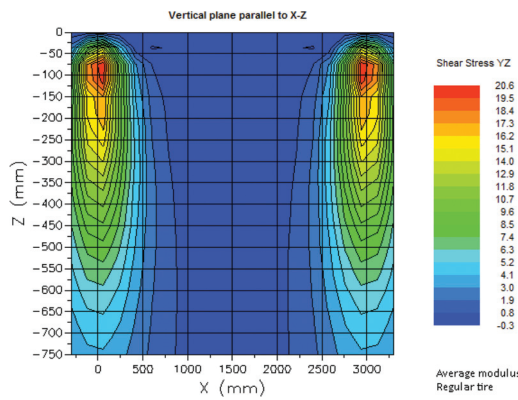

Figure 11. Shear-stress for average modulus structure and regular tire load.

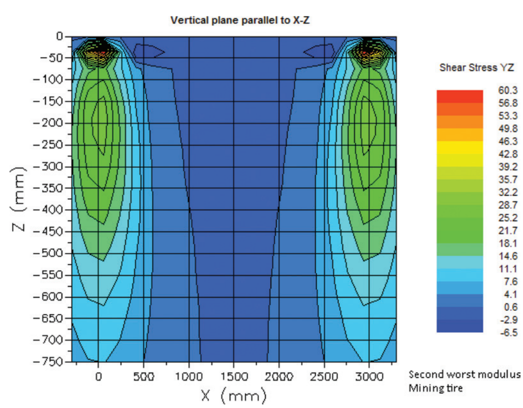

Figure 12. Shear-stress for second worst modulus structure and mining tire load.

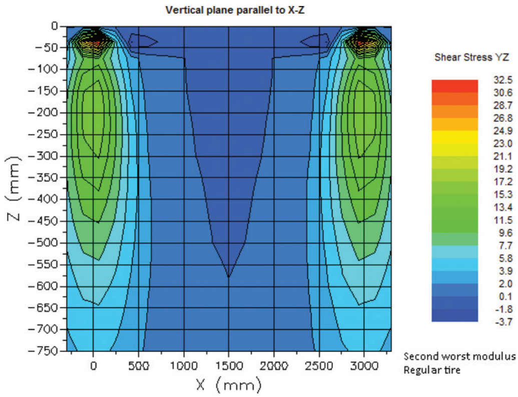

Figure 13. Shear-stress for second worst modulus structure and regular tire load.

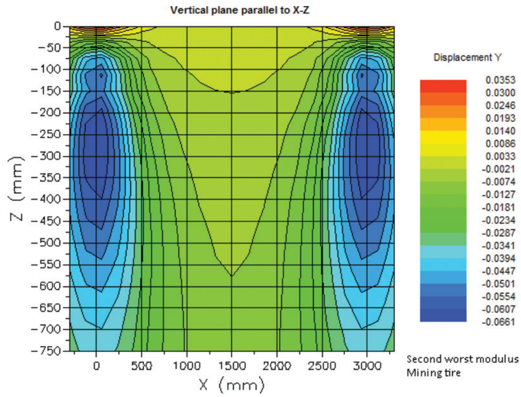

Figure 16. Y displacement for second worst modulus structure and mining tire load.

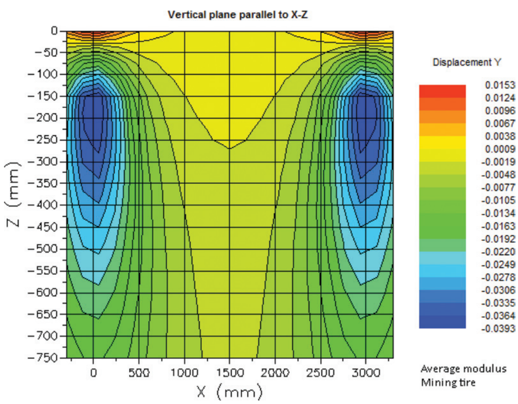

Figure 14. Y displacement for average modulus structure and mining tire load.

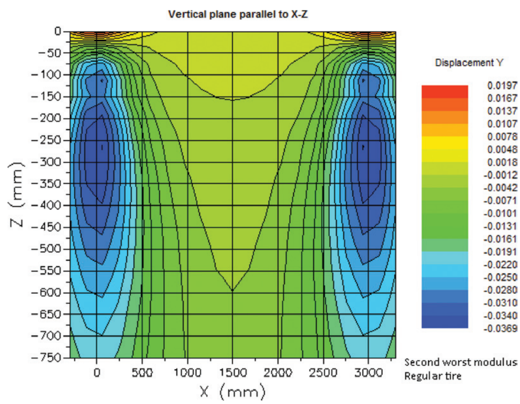

Figure 17. Y displacement for second worst modulus structure and regular tire load.

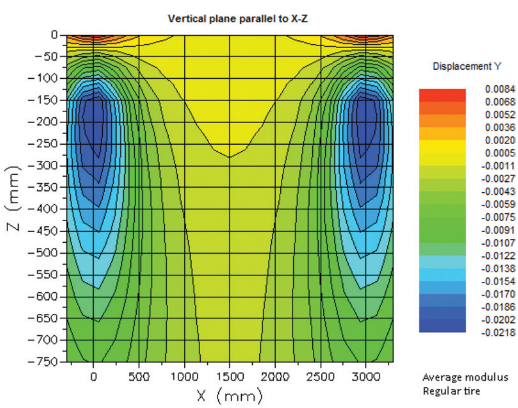

Figure 15. Y displacement for average modulus structure and regular tire load.

7 DISCUSSION

The previous Figures show that the shear stresses are quite high for the exempt loads on the weaker structure. However, the suggestion to increase the total truckload does not seem to be challenging to road deterioration as long as the legal maximum axle load is not changed. As can be seen in the same Figures, axles spaced by three meters do not interact much in the base and sub base material, but the left and right-hand wheels do. Some materials, such as hot asphalt and saturated unbounds, will have a shorter time to recover between loads though. This will likely increase rutting rates.

If axle loads increase, it is a very different story though. From the field we gathered data from a site that had been subjected to exempted heavy

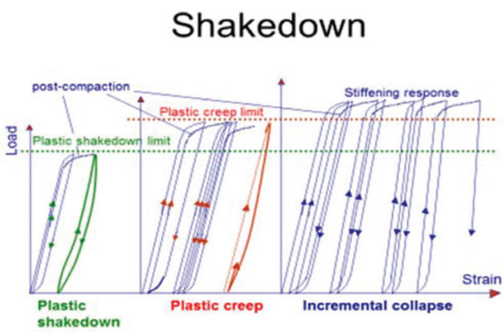

Figure 18. Unbound Layer behaviour (Adapted from Werkmeister *et alia*, 2001).

loads. In addition this occurred before all asphalt layers were in place. All layers, including the subgrade were subjected to much higher loads than in a normal design of a road.

We did observe that the stresses were high, but many elastic responses were normal nonetheless.

At the weaker points, one can see the following stages occur:

1. Post-compaction of upper unbound layers, energy is being dissipated.
2. As stiffening occurs, less energy is dissipated; layers further down are being compacted.
3. Plastic creep limit is exceeded; the process starts all over again, as shown on Figure 18.

The explanation to the process by Werkmeister et al (2001) and the influence of the octahedral shear stress as presented by Bonaquist (1997) are important pieces of the puzzle of understanding the deterioration of unbound materials. Add the seasonal variations and the dissipation of energy; it is clear that we are dealing with a very complicated process.

8 CONCLUSIONS

Allowing higher loads on fewer trucks is a good idea, from a sustainability standpoint, as the amount of greenhouse gases produced per weight unit moved will be reduced. The maximum axle load is not to be increased.

Using a linear elastic model, the fatigue and rutting will not increase for a constant amount of goods being hauled. However, as rest periods between loads become shorter a factor increasing the number of loads by 1.25 is suggested for pavement design. For higher roads categories, in effect this factor is about 1.09.

Material strength specifications may have to be changed, as the higher gross weights combines the loads to higher stress levels, and possible more grinding. We do know this from previous experience and the mining road example as well. This may mitigate the sustainability of allowing higher gross weight, and further lead to shortage of available materials.

At this point, we do not exactly know what the stress and respective strain limits should be in real numbers for any given unbound material. The interaction between layers, the surrounding environment and the loads comprise as complex situation, which is difficult to model. For performance-based construction in particular, there is a need for simple guidelines to control the deterioration and keep risks manageable. With field tests from the FWD, one can effortlessly derive vertical strain in any layer. As we can see from the present example unbound base material having a vertical strain less than 1000 micro strain did fairly well. Sub base layers having a higher confining pressure could probably endure somewhat higher strain. Weaker materials being close to the LA drum test percentage currently used should not be used. Expect stricter limits on materials, when higher loads are permitted.

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