

# Comparing Design Strain to Actual Strain on New Roads

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**ABSTRACT:** In an active design project it was found that the target design strain was fulfilled for about 90 % of the several hundred tests made with a Falling Weight Deflectometer (FWD). The tests were done after one year of traffic. This outcome is in line with current design principles employing statistical methods based on reliability. More disturbing is the fact that about half of the sections would have an expected life that exceeds twice the design life. This was in spite the active design efforts to reduce bearing capacity variability. From a structural point of view, this may not be alarming; but from a sustainability standpoint one could see resources being wasted on sections that do not need such a well-built design. Thus, it would be of interest to see what the bearing capacity is on new road sections and compare it with the design code. In 2010 the Swedish Road Administration funded a project where a number of new and widened roads were tested for bearing capacity with a FWD. The roads comprised different categories and types. The tests showed that generally the roads are oversized as compared to the design code. The present paper discusses whether this is purely an effect of calibration in the mechanistic-empirical process or if there are any inherent errors in the code handling the natural variability. If the latter is the case, investment costs could be lowered for the benefit of sustainability. Further, more efforts could be placed on the sections exceeding the design strains.

## 1. INTRODUCTION

Road authorities in Sweden are steering road construction contracts towards performance criteria with a long-term warranty commitment. The intention is to get more roads for less money as there is an incentive for the builder to come up with costs savings and smarter ways to build and maintain as well. Usually, there is a reduction of materials as well that suits sustainable solutions. In future contracts sustainable solutions is likely to increase as they will become a part of the overall economics when choosing the most competitive bid. By the time this text is published, several five and eight year contracts have been finished and some long-term ones; up to sixteen years have been initiated. The performance criteria used are cross slope, friction, rutting, and roughness. The cross slope usually does not change much over time so that is more or less regulated at the time of construction. The friction is normally handled in the

maintenance contracts, but the rutting and roughness are parameters that change over time and their respective deterioration rates are predicted in the design.

On high volume roads the rutting rate has proven to be the most significant parameter for the contracts so far. Small, unforeseen mistakes by the contractor have been very costly in some cases. These include not having control of the bitumen supplied to poor knowledge of the mechanisms behind the rutting rate. For the administration on the other hand there is some concern about relying too much on road user parameters for asset valuation. This becomes crucial when a 20 year contract is ending as rutting as measured can be mitigated with several different methods. Even though the present serviceability is fine, the rutting rate is much more important for the asset value which is needed for finalizing the contract. Some structural evaluation is indeed necessary and the present authors are confident that backcalculating E-moduli and determining strains in critical points is one of the better ways of achieving this. Alternately one could use instrumented sections to directly measure the strains in question. However, even though there is quite a lot of data on reconstructed projects designed by FWD testing and backcalculation, there is very little knowledge on the evolution of bearing capacity from new to design life in Sweden. The interest for measuring new projects has been limited. Early tests usually showed indifferent results for unbound layers as these are gaining strength when being compacted by traffic. In turn this yields some initial rutting in the one to five mm range. In later years it has been shown that sections prone to initial rutting can be identified. Therefore the rutting rate in future years could be assessed together with the initial year, [Englund 2011]. It was suggested to test a number of new roads and assess strain in the field and compare data to the design strain.

## 2. OBJECTIVE

The objective with the research is to find out how well the design criteria meet actual strains on new roads. The results are then used to improve on reasonable criteria for long-term warranty performance contracts.

## 3. SCOPE

In the present paper, a part of the investigated material is discussed due to space limits. The conclusions are based on all tested sites. There is no evaluation of the current design guide per se.

## 4. BACKGROUND

Like many countries, Swedish road design criteria are based on controlling strains at the bottom of the asphalt layer and at the top of the subgrade, [Vägverket, 2008]. In 2007 a project employing active design was completed. The traditional mechanistic-empirical design parameters were used as subbase, base, and binder layers were constructed. The design was altered after evaluating each layer. After the wearing course was placed, a final FWD was performed as a check of the outcome. The distribution of bearing capacity as additional asphalt needed was determined. A negative value would indicate that there is actually more bearing capacity than needed. The need was then sorted to form a distribution and the result is shown in Figure 1 below. As is shown in the Figure

the zero-need is found at the 90-percentile, which is right on target for the active design. Ten percent is thus likely to fail before the design period has ended based on reliability. A weakest point design would have rendered an extra 50 mm layer of asphalt concrete or approximately 1000 ton asphalt concrete per 1 km of 9m wide two-lane road. Employing good engineering practice, it is more cost-effective to set aside some extra maintenance money rather than use a design after the very weakest point. It is also interesting to see that the median as built needs 25 mm less AC than the design thickness. In this particular case it corresponds to allowing for twice the equivalent number of standard axles. The graph hints towards active design over shorter segments and huge potential savings.

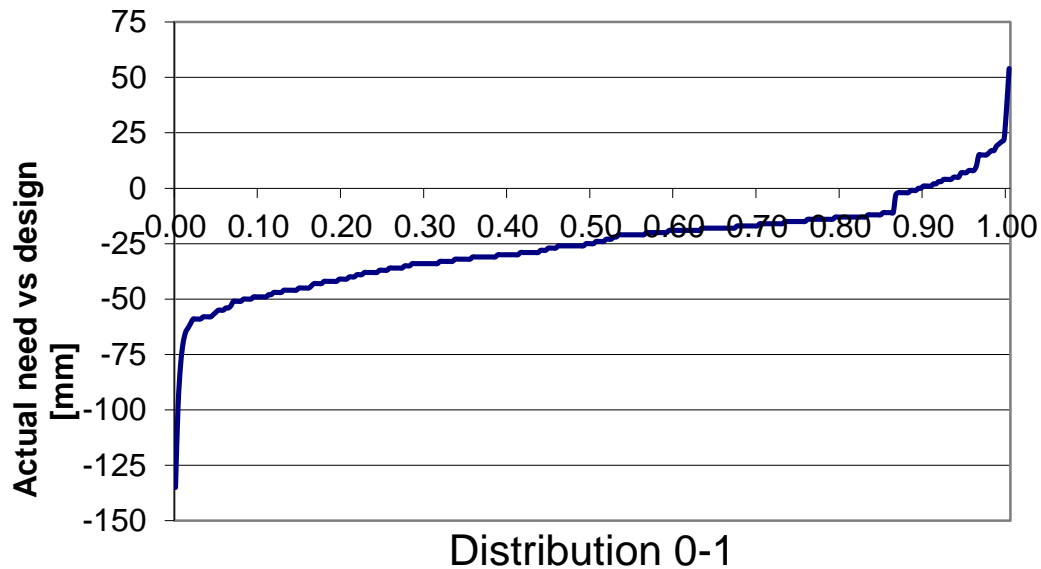


Figure 1. Frequency Distribution of needed asphalt thickness to match the design life for a new road.

## 5. FIELD COLLECTION

Five different designs were chosen in different design categories from rural four-lane divided carriageway highway to a local trunk road, see Table 1. In the present paper the new projects are presented.

Table 1: Sites chosen for the new road bearing capacity evaluation

| Site                 | Road Category           | Design EASL | Comment          |
|----------------------|-------------------------|-------------|------------------|
| E18 Oslo-Stockholm   | 4 lane dual carriageway | 20 000 000  | New alignment    |
| Rv68 at Hästbo       | 2 lane bi-directional   | 3 000 000   | New alignment    |
| X512 Hästbo          | 2 lane county road      | 300 000     | New extension    |
| Rv68 Örebro-Gävle    | 3 lane divided          | 6 000 000   | Widening project |
| Rv70 Enköping-Norway | 3 lane dual carriageway | 10 000 000  | Widening project |

#### 4.1 Motorway E18 at Sagån

Sagån is located about 100 km due West of Stockholm at the Uppsala-Västmanland county border. This section represents a rural freeway with the passing lane designed with the same thickness as the slow lane. In reality it means that the passing lane is heavily overdesigned as most heavy trucks use the slow lane. Thus, FWD-testing was done in both lanes.



Figure 2: E18 Sagån site at the time of testing prior to being opened to traffic

The testing took place in mid-September 2010, when conditions were near the annual average for the unbound layers. The asphalt mean temperature was near 15 degrees C, which is the annual mean temperature. The road was not yet opened to traffic, and some construction work was going on like pavement markers and guard rails et cetera. A Ground Penetrating Radar (GPR) unit was mounted on the FWD so that the pavement thickness could be checked. Ten multiple drops were used at three different load levels; 40, 50 and 70 kN repeated twice with a seat drop of 50 kN.

##### 4.1.1 *Stress sensitivity in the unbound layers.*

By comparing the backcalculated moduli for the different load levels and thus the bulk stress, a regression can be made between the two.

$$E = k_1 * (\sigma_1 + \sigma_2 + \sigma_3)^{k_2} \quad \text{Eq. 1}$$

If the correlation is good and if the  $k_2$  is in the range of .25 to .50, it usually means that the material is in a well-compacted state such as in a conditioned tri-axial test cylinder. As it turned out the regression here was poor, a true indicator that the surface was only sparsely trafficked and that some initial rutting is to be expected during the first year.

##### 4.1.2 *E-moduli*

The backcalculated moduli were considered to be normal and the subgrade values were rather high, see Table 2 below. The asphalt modulus was adjusted to 15 degrees Celsius, but from a narrow range of a few degrees. The slow lane was assessed somewhat higher

values for the unbound materials and somewhat lower for the bound ones. However the difference is very small and must be considered to be within the expected variation and no further evaluation was considered necessary.

Table 2: E-moduli at Sagån, Mean and Standard Deviation

| Lane    | Asphalt Layer Mean [MPa] | Asphalt Layer Std.Dev. [MPa] | Unbound layers Mean [MPa] | Unbound layers Std.Dev. [MPa] | Subgrade Mean [MPa] | Subgrade Std.Dev. [MPa] |
|---------|--------------------------|------------------------------|---------------------------|-------------------------------|---------------------|-------------------------|
| Slow    | 9500                     | 949                          | 215                       | 44                            | 187                 | 45                      |
| Passing | 10800                    | 962                          | 194                       | 25                            | 178                 | 39                      |

#### 4.1.3 Time Histories and Load-deflection diagrams

All drops were recorded in the time domain. The FWD used was calibrated to achieve good data well beyond the peak of the deflections. This serves two purposes. One is a quality check to see if there are saturated soils with moving water present. The other is an estimate of the hysteresis in the structure, [Lenngren 2009]. The former check did not reveal any water moving within the influence of the deflection basin, see Figure 3. As for the hysteresis it varied in the 7 to 13 Nm range for the highest load. This variation is somewhat higher than considered normal, but no adjustment was made for temperature.

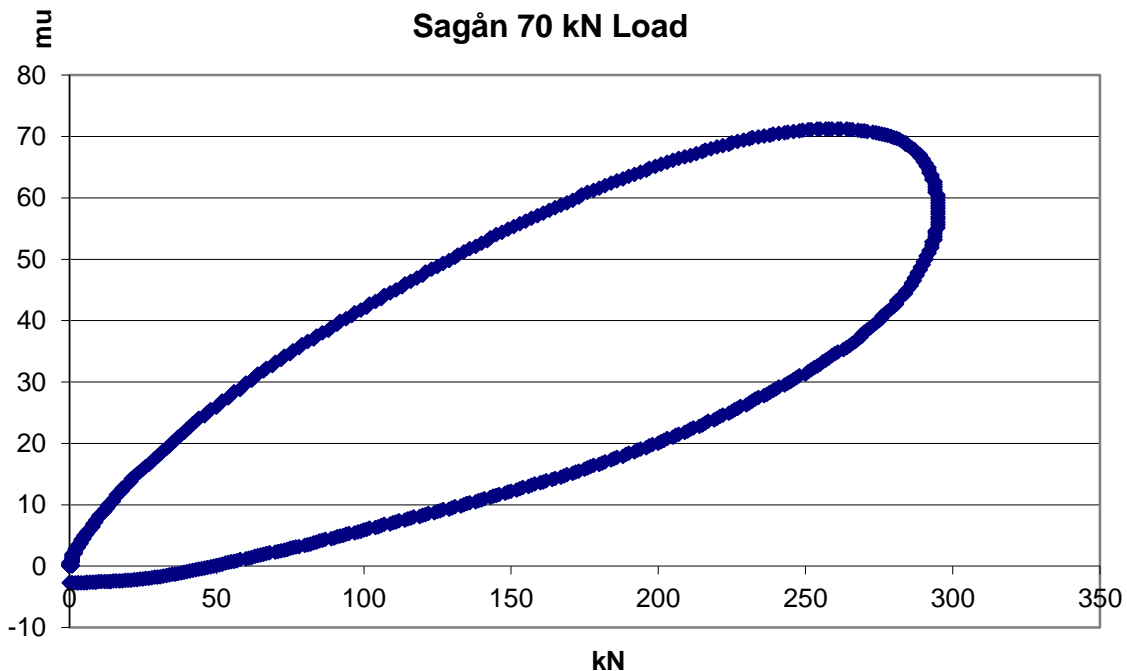


Figure 3: Load-Deflection diagram for a 70kN load. A typically shaped curve for a 200mm asphalt concrete pavement at around 15 degrees C. Time progression is clockwise.

#### 4.1.4 Design strains versus actual strain

A program for overlay design was used to calculate the need for an extra layer of asphalt on the measured pavements. On new pavements one would not expect more than

a few percent of the tests points to be under-designed, if any. This proved to be the case at Sagån where all tests were well within the required criterion. However, this may not be the case for every season and load situation, but the late summer to early fall situation is close to the weighted average for the difference in season. Usually, the winter time produces less damage and the spring thaw the most. The latter is usually shorter though and whatever accumulated damage there is, is indeed balanced by the next to nothing damage during the winter. This can be verified by any mechanistic-empirical design software. Recall that the asphalt modulus must be adjusted to the reference temperature for all tests. However, extreme overloads during spring thaw or during the hottest weather in the summer are very hard to predict and cannot be fully compensated for, using any method.

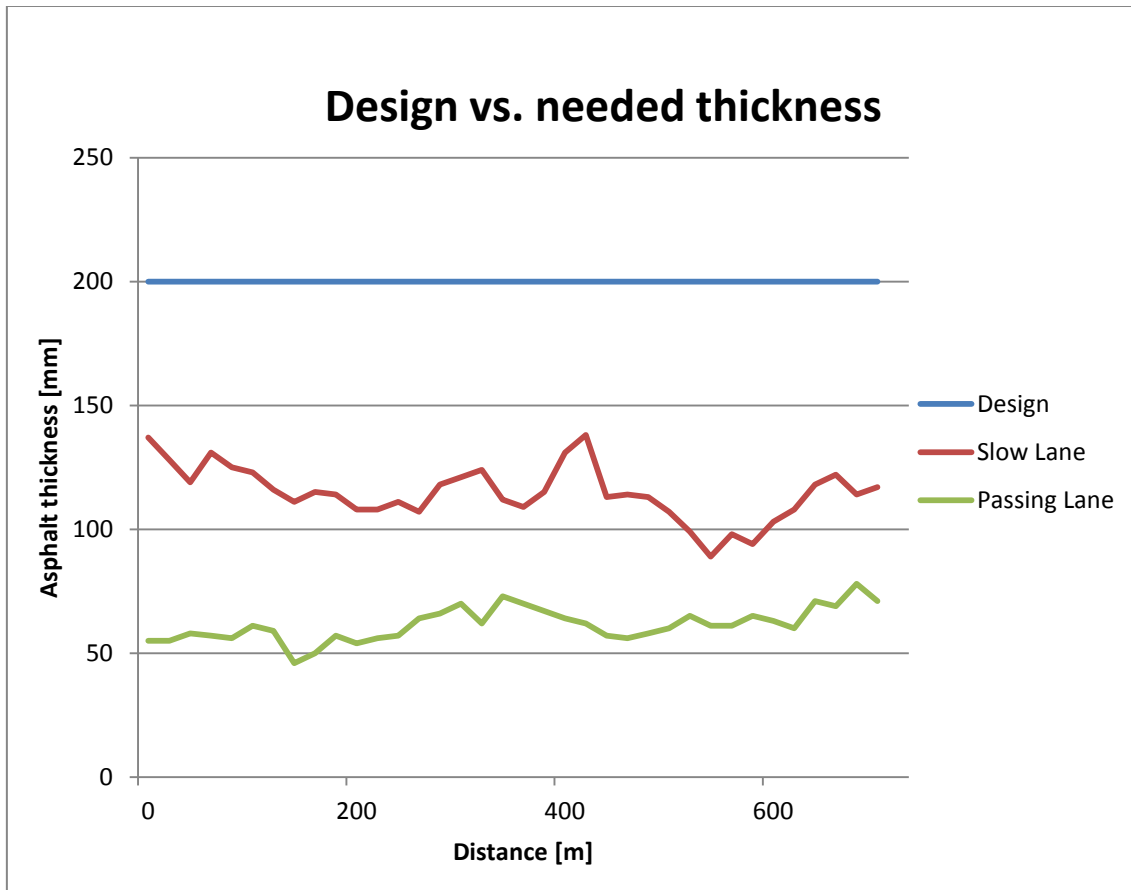


Figure 4: Sagån. Bearing Capacity “when new” expressed as need for AC thickness.

It is obvious that the relatively stiff subgrade does not require an asphalt thickness of more than 140 mm in the slow lane and only about 70 mm in the passing lane, based on lane distribution for the number of standard axles.

#### 4.2 Highway 68 at Hästbo New Two-Lane Construction

This project is located on a highway in a steel producing area, so truck traffic tends to be heavy. It is located about 50 km southwest of the Baltic seaport Gävle. The final wearing course was placed after a year in traffic. Hence, traffic compaction of the unbound layers is expected to have occurred. The FWD testing was done shortly after the final layer was placed and one day after the Sagån test was done. The subgrade consists of coarse moraine.

#### 4.2.1 *Stress sensitivity in the unbound layers.*

In contrast to the site above the regressed stress sensitivity for the unbound layers was much better with a coefficient of determination ( $r^2$ ) of up to .89. (Most were above .50). The  $k_2$  was indeed in the range of .2 to .4 indicating a stress situation found in tri-axial tests. This is a behavior likely to occur when the grains have settled by the compaction from traffic. The subgrade did not exhibit such a behavior here. It is further from the load and needs more time to settle, and besides in might be disturbed by frost actions, which are moderate to high in this area.



Figure 5: Testing at highway 68 near Hästbo.

#### 4.2.2 *E-moduli*

This test section comprised 5000 meters and is much longer than the previous site, so there is a higher variability of the subgrade. The asphalt layer is also varying more, maybe due to the fresh status of the wearing course. The unbound base yields about the same modulus as the other case, but the subgrade is much stiffer, likely without any cohesive content.

Table 3: E-moduli at Hästbo, Mean and Standard Deviation

| Lane | Asphalt Layer Mean [MPa] | Asphalt Layer Std.Dev. [MPa] | Unbound layers Mean [MPa] | Unbound layers Std.Dev. [MPa] | Subgrade Mean [MPa] | Subgrade Std.Dev. [MPa] |
|------|--------------------------|------------------------------|---------------------------|-------------------------------|---------------------|-------------------------|
| Both | 4900                     | 1850                         | 200                       | 54                            | 421                 | 188                     |

#### 4.2.3 *Time Histories and Load-deflection diagrams*

The load deflection diagrams at this site show close to elastic behavior. Note the almost linear response for the outermost sensor in Figure 6. This sensor is mostly affected by the subgrade, which was surmised not to contain any finer material. The overall hysteresis is generally less than 4 Nm, due to the thinner asphalt.

#### 4.2.4 Design strains versus actual strain

As in the previous case a program for overlay design was used to calculate the need for an extra layer of asphalt on the measured pavements. Again, the thick design proved to be more than enough for what was actually needed as can be seen in Figure 7. If 40 mm is deducted for the wearing course, the first year in traffic was exceeding the normal design rutting rate of .8 mm per year for about half the length, so with the initial rutting rate the first year rutting could have been as much as 5 mm though. A follow-up for this site of the rutting would be highly recommended. At the present time there are only two more years of data and it falls outside the scope of the paper.

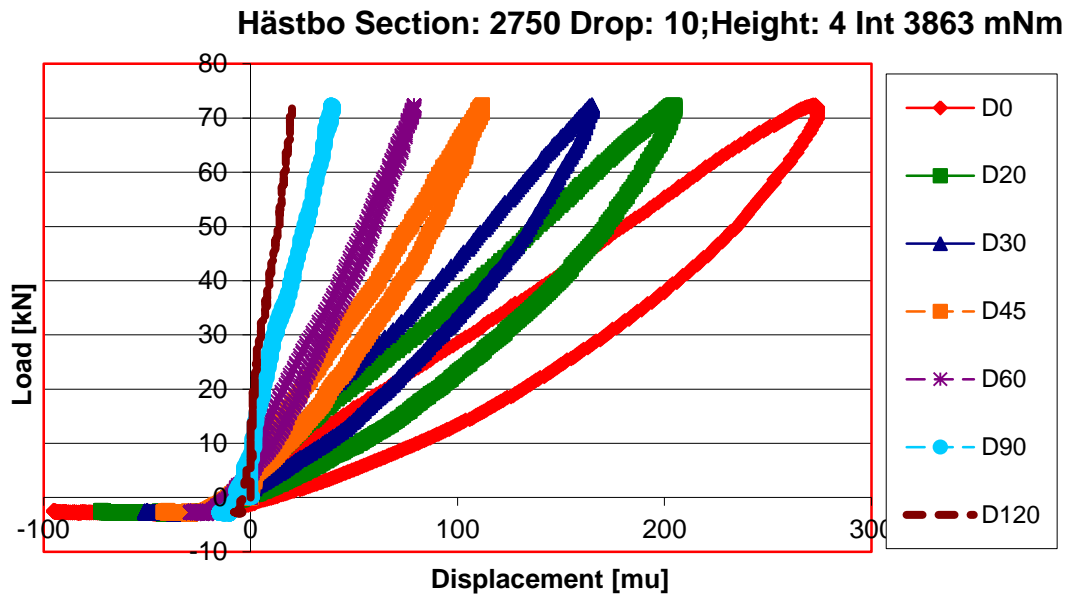


Figure 6: Load Deflection for all sensors for a 70 kN load at the Hästbo site.

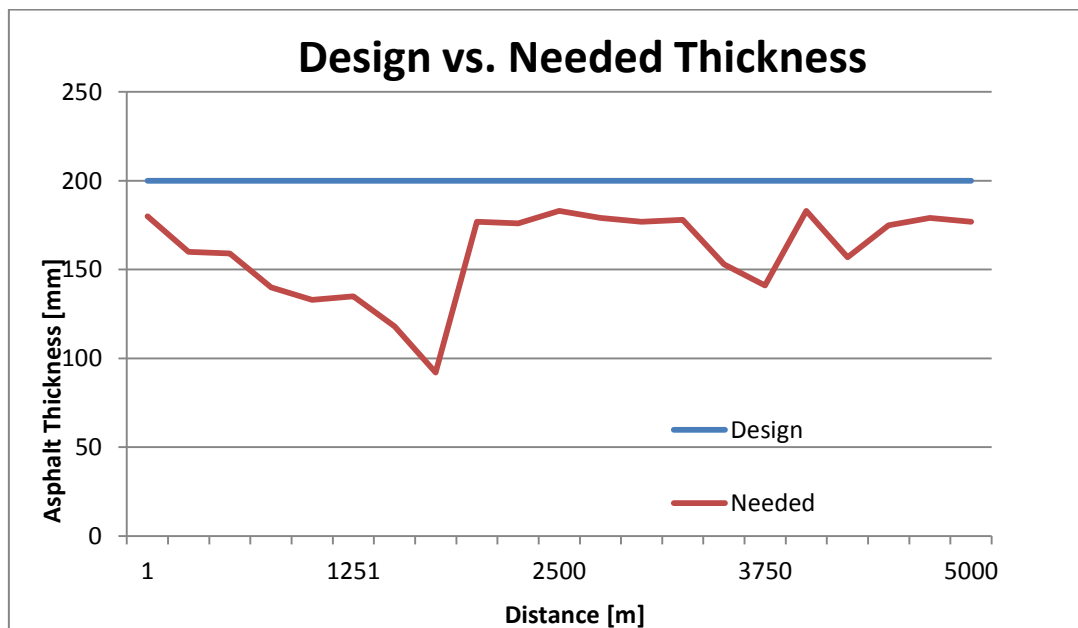


Figure 7: Design and needed AC thickness for the Hästbo site.



### 4.3 X512 Low Volume Trunk Road

As the new alignment at Hästbo was further to the west than the old road, a secondary county road had to be extended. It was suitable to include it in the study representing a much thinner design. A 495 meter long section was tested every 25 meters in both directions on the 6 meter wide road.

#### 4.3.1 Stress sensitivity in the unbound layers.

The stress sensitivity indicated well compacted unbound layers except for two of fifteen stations tested. The road had been exposed to traffic for about one year at the time of testing.

#### 4.3.2 E-moduli

The asphalt layer is just a thin surfacing layer, so the E-modulus is calculated in a compressed state. The subgrade is stiff here as well.

Table 4: E-moduli at X512 Trunk Road, Mean and Standard Deviation

| Lane      | Asphalt Layer (compressed) Mean [MPa] | Asphalt Layer Std.Dev. [MPa] | Unbound layers Mean [MPa] | Unbound layers Std.Dev. [MPa] | Subgrade Mean [MPa] | Subgrade Std.Dev. [MPa] |
|-----------|---------------------------------------|------------------------------|---------------------------|-------------------------------|---------------------|-------------------------|
| Both ways | 15500                                 | 7900                         | 190                       | 54                            | 323                 | 113                     |

#### 4.3.3 Time Histories and Load-deflection diagrams

In comparison with National Highway 68, the thinner road here is subjected to much larger deformation and the outermost sensor is indicating quite some damping in the subgrade, which will contribute to the rolling resistance. There is also some heaving at the very end of the cycle, which could be some reflection wave. The overall hysteresis is over 9 Nm.

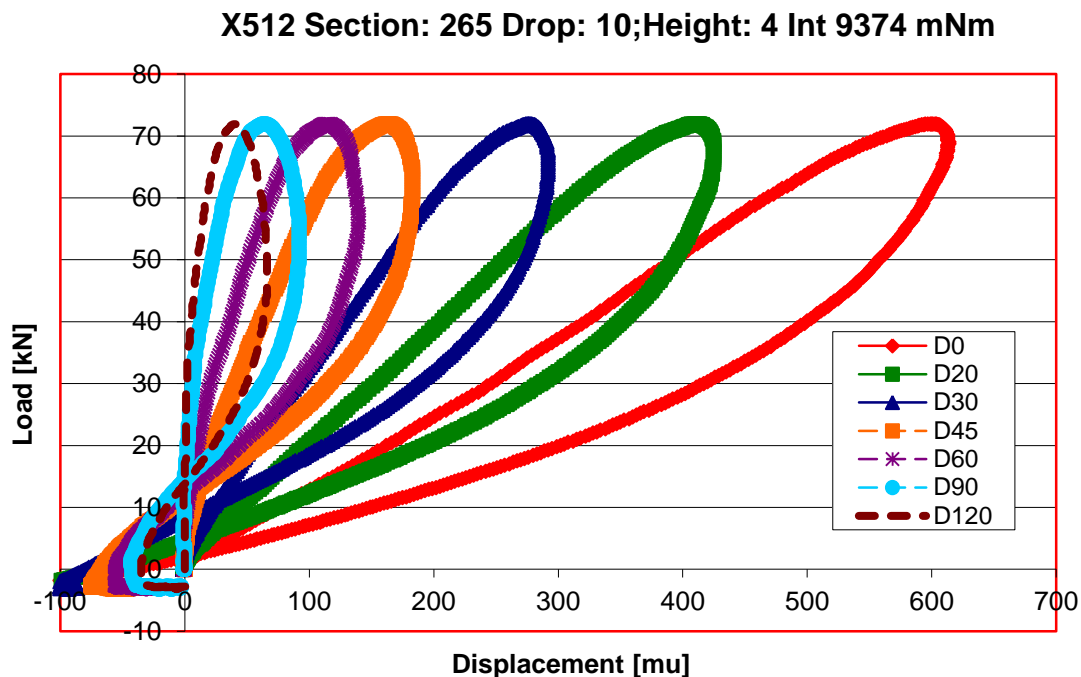


Figure 8: Load-Deflection diagram for a low volume road, X512.

#### 4.3.4 Design strains versus actual strain

In this case about half of the sections actually need more asphalt than the design. This is actually not far from what is desired on a minor road, i.e. the median need is often the design for this road category. In comparison with the higher volume roads this is closer to the intentions in the mechanistic-empirical design guide.

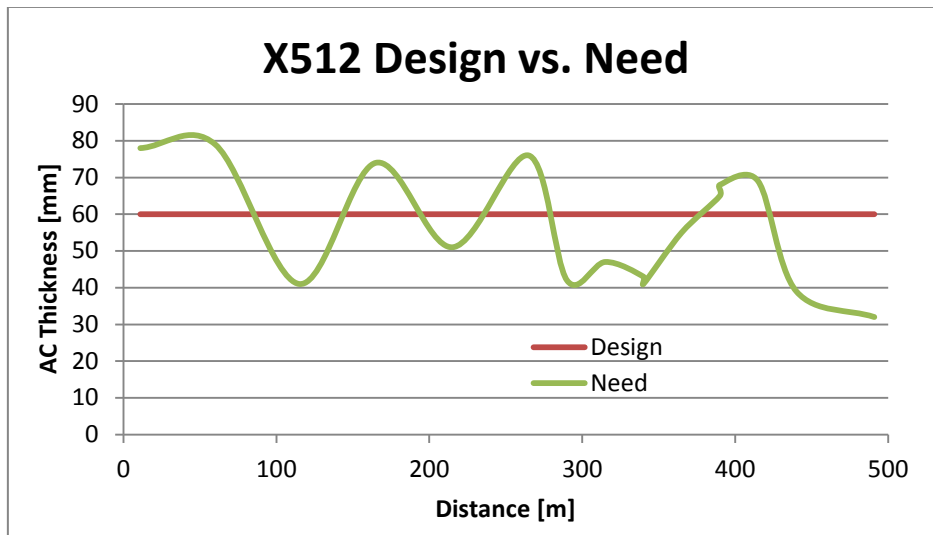


Figure 9: Design and needed AC thickness for the Hästbo trunk road site.

## 5 CONCLUSIONS

In comparing the mechanistic-empirical design with the actual need for newly constructed roads, it seems that the high and intermediate design traffic examples are often overdesigned. The low volume road however varied more, and half of the sections needed a thicker asphalt layer than the design. Most of the sites investigated exhibited relatively stiff subgrade conditions, a state that the design code does not fully appreciate, which could lead to the overdesign. The field testing and analytical design concept opens up for a more sustainable approach as the favorable conditions could be accounted for. Further, the performance contracts are often regulated by bonuses and penalties depending on the outcome. These could be adjusted or revised as the conditions indeed vary depending on the site.

## REFERENCES

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