How to Assess Elastic Properties of Heterogeneous Pavement Layers

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ABSTRACT: Backcalculation of elastic moduli is a common mechanistic approach for pavement overlay design. The method contains a model for assessing stresses and strains in various parts of the structure predicting the bearing capacity and future expected life. The pre-computer era model was based on a single surface deflection only, which could be elaborated to various states of the loading, temperature and materials. Today, most pavement engineers analyze the shape of a deflection basin, allowing for a 3-dimensional view of the structure. Thus, we assess different elastic properties for a number of layers in the structure. By changing the nature of the load by magnitude or duration we can also derive non-linear properties often by using finite element modeling. However, if too many properties are being solved at the same time, there are two drawbacks. One is the long computational time being required, the other is that solutions may not be robust. So many pavement engineers use e.g. a standard load, a standard temperature et cetera. Unfortunately, in such cases it is difficult to use heterogeneous layers in the model, like coating and penetration binders. It has been suggested that such layers are treated as two separate ones, but one of their key functional roles of eliminating sliding interfaces is then being forfeited. The present paper suggests a guide to assess appropriate elastic moduli, as well as layer thickness and layer interface friction that could be used for linear elastic analyses.

KEY WORDS: Backcalculation, Falling Weight Deflectometer, Heterogeneous Layers

1 BACKGROUND

Road design specifications were not too long ago a set of tables for layer thickness for various road materials. The actual design was based on the expected traffic load as a number of standard axles, the climate, and local conditions as drainage, type of soil and whether cut or fill persisted. Originally, the design was based on experience backed up with some tests from the industry, e.g. for cement concrete road design in Pennsylvania in the 1920:ies. Hveem and others concluded that bearing capacity could be attributed to surface deflection for certain design types. However, the relationships could not be extrapolated to a general formula.

The theory of elasticity was used early on to connect the relationships to some engineering measure. By the work of Odemark the theory was extended to include several layers with different elastic properties, which still is the approach used to day for flexible pavements (Odemark 1949). Westergaard did pioneering work for concrete slabs resting on soils with a slightly different approach controlling the stresses in the slab, but also relying on the theory of elasticity (Westergaard 1947).

As roads do not rupture or collapse but rather gradually decay it was needed to introduce a pavement performance concept. It will tell if the road fulfills its expected task, conveying traffic safely at a reasonable comfort, rate and speed, including environmental aspects. Physical measurable parameters can be correlated to the performance. The roughness of the road is a good single indicator as such and could be used as a present serviceability index (PSI), which in turn could be converted into monetary terms.

To predict future pavement performance the PSI is not an adequate measure though. There are many examples of new roads, perfectly fine when they opened, but with an exceptionally large deterioration rate. These roads' performance may fail after a few years, where equally fine roads at the opening day may last well beyond thirty years before failure. Here, obviously a relationship between structural and functional status is needed. Large road tests, like the AASHO Road Test 45 years ago were done in an attempt to establish these relationships, (AASHO 1962). In later years Mn/Road (VanDeusen et al. 1994), Smart Road (Al-Qadi et al. 2000) and other sites where numerous instruments are used to measure the actual deterioration is a great help in refining the equations describing what is going on for more types of road materials and under different climate conditions.

So, there are some mechanistic tools for behavior and then a model for performance, which will give the design life for a certain amount of traffic. Such models are called empirical-mechanistic as they still are based on experience of a standard design; the mechanistic part is given within some range of a property that may not deviate too much from the original design.

For the experienced road engineer that lived to see the past fifty years of evolution, this is nothing new really. However, a new cadre of engineers is inevitably replacing the people who now are retiring from the business. Therefore, it is necessary to reiterate that the relationships being used are only good for the circumstances that they were derived from. Thus, if improved structural models dealing with non-linearity, visco-elasticity or dynamics effects are being used, the original criteria must be adjusted accordingly. In some cases the criterion may be altogether replaced entirely.

In the traditional road design two different criteria are being used. One is material specific and relates to the fatigue strain. It is rather straight-forward. Find what strain the material is exposed to and derive the number of repetitions allowed. The other criterion is more indirect as it relates to the deformation (rutting) on the surface. Engineers found early that the strain on top of the subgrade did indeed correlate fair with the rutting. This does not mean that this criterion is particularly good; it does mean that it is one of the best single parameters you can find in a one variable regression. However, it is likely to be replaced in the near future with deformation in individual layers being calculated instead.

Another culprit is the rigid base concept. In some situations there is bedrock present near the surface. Treating the subgrade as one layer may be violating homogeneity a little bit too far, if for instance soft clay is resting on hard rock. The top of the subgrade strain will not be calculated correctly. Well, the problem can be solved more accurately if it is understood that at some point in the subgrade there is no movement at all induced by the given load at the top. There are actually some clever ways of finding that point by looking at the shape of a deflection basin such as the one from a Falling Weight Deflectometer (FWD) load. If that is done it should be possible to adjust the criterion accordingly. In addition, testing at many load levels will further help in finding the depth to a rigid base.

Anyway, it is important to understand the concepts of these models. Some people do rightfully believe that the current method should be scrapped immediately and be replaced by an accurate behavioral model accounting for the aforementioned non-linear, visco-elastic, dynamic models and also include plasticity. However, there are some difficulties with these models as well. First, you must derive the various parameters which may be quite expensive to do. Secondly, the variability of these models may be large, so errors may be introduced just in the spatial selection of testing. So in conclusion, there is still room for improving existing models in a feasible way.

2 HETEROGENEOUS LAYERS

When deriving in-situ layer moduli by backcalculation methods, each layer is considered homogenous. In a crude way, this will work for overlay design purposes as long as the strains used for the respective criteria are near that of the actual (non-homogenous) ones. If indeed such conditions prevail the backcalculated basin will fit the measured one perfectly well. The common practice of using a FWD for the input data will usually render a goodness of fit as a root mean square (RMS) average value per sensor. If this value is 1.5 % or lower one can assume that the model being used is adequate for the overlay design. However, if most values are higher than that, one should reconsider the model. The skilled pavement engineer would perhaps try altering the layer thickness being used, or if there is reason to believe that there are gradients in the layers, try to subdivide the layers into more layers. Most commonly this would occur when there is a temperature gradient in thick asphalt pavements, or when high stress sensitivity occurs in unbound layers or the subgrade.

Assume that the upper half of a 130 mm thick asphalt concrete layer is 3000 Mpa and the lower half is 6000 MPa. There is a 150 mm unbound base layer and a 400 mm thick subbase of 300 and 100 MPa stiffness respectively. The structure is resting on an 80 MPa stiff subgrade. Using a four-layer system combining the asphalt concrete layers with the linear elastic backcalculation program CLEVERCALC 3.9 a layer modulus of 4279 MPa was derived for the combined layer. The corresponding horizontal strain at the bottom of the layer in the model however was 253 microstrain, considerably higher than the actual 217 microstrain induced by a 50 kN load. Hence, it is worthwhile to check for gradients otherwise the overlay may be over designed.

If there is a colder and thus stiffer upper half the overlay will be underdesigned instead. Using the same structure as above, but with the stiffer layer on top, the design strain for the combined four-layer model is now 248 microstrains, much lower than the actual 276 microstrain.

The computer program CLEVCHECK (Clevercalc Laboratory 1.0) can be used to produce any number of given combinations and variations of the input parameter used for backcalculation. Thus, it is a test bench if you like for backcalculation programs like CLEVERCALC. It has previously been used for testing the influence of Poisson’s ratio and sliding interfaces, and faulty layer thickness data. It was now used to check when small to intermediate differences in moduli start to affect the result. It was found that if the difference of the layers is reduced to 6000 and 5400 MPa, the difference in strain is a mere 2 % between using a combined or two different layers. So as a rule of thumb one can consider a layer as homogenous if the stiffness does not vary more than percent, which is also a number that can be used for thickness variation. When in doubt, a simple test like the one described above can always be tried.

Yet, there are still layers that are constructed in such a way that they are intrinsically heterogeneous. Typically, various types of coating procedures and penetration binders on coarse materials would be suspected to have a varying stiffness by depth by more than 10 percent. Is it possible to assess the “equivalent modulus” property appropriately, or should they be divided into finer layers?

3 DISCUSSION

Previously when backcalculating coated layers it has been suggested that the layer should be considered bound down to a depth depending on the amount of binder and the actual penetration in turn depending on the gradation of the mix and the binder viscosity. As a rule of thumb, two thirds of the actual thickness is often considered bound and the rest is attributed to an unbound material. When backcalculating these structures a somewhat higher RMS per sensor is achieved than for the more traditional asphalt-gravel structure. Usually one derives values around 1.5 percent rather than the usual one percent when the actual thicknesses are known for homogeneous layers. This is somewhat disturbing, but not upsetting in any way. (One can easily check this by comparing two calculations with different settings on the allowed tolerance).

As mentioned earlier, any "top of the subgrade criterion" should not be affected by backcalculating the layer moduli this way. The question is whether this point in the structure is adequate for the fatigue calculations? Certainly, the tension at this point in depth (2/3) is lower than at the bottom of the layer and that will affect the fatigue equation. If the binder penetrated deeper, tensions will be higher. If it did not go that far, backcalculated tensions will be lower, so in some respect this is self-regulating. Also, if there is binder lacking, the derived modulus will be lower, tensions higher, so from an engineering standpoint, the tensions work for design and/or overlay design purposes. It remains to be proven though if a standard asphalt fatigue equation could be used or if it has to be modified. Only long term pavement performance could tell really, but data gathered so far does not indicate large discrepancies for this type of layer as compared to the traditional ones.

The current Swedish Design Guide recommendations seem to be very conservative when calculating these layers. The bound part of the layer is only considered to be 20 mm thick and is assigned an elastic modulus of only 25% of that for a standard bound base. The rest of the layer is considered to be 450 MPa, which actually is a little bit high. As the bond with the layer beneath is quite important, this modulus is a function on that layer. Anyway these values are far from what one usually derives from backcalculating a combined layer.

4 FIELD TEST

In a long term pavement performance project in central Sweden, described at earlier BCRA conferences (Lenngren & Fredriksson 1998 and 2002), it was found that some sections with coated macadam from the 1960:ies performed exceptionally well in comparison to traditional asphalt concrete structures. One explanation is that the friction between the bound and unbound layers serves as to stabilize the structure considerably. A slipping interface could in effect reduce the bearing capacity as much as loss of stiffness in an unbound base from 300 to 150 MPa, (Lenngren 2002a).

The Swedish Road Administration Construction Company decided to test this type of layer in a number of new structures, but it was modified somewhat and dubbed "Runbase". It consists of grouted macadam coated with an open-graded asphalt concrete mix. It comes normally in three different thicknesses, 5, 7.5 and 10 centimeters.

About ten field sites with Runbase have been carefully monitored since the mid 1990: ies. They have been subjected to FWD testing and surface characteristics evaluation as well. In the present paper one of the sites is discussed as an example of what can be expected from such a comparison.

A section of a two-lane national road near Uppsala in the lake district of Sweden was reconstructed with Runbase. There were also traditional asphalt bound base and cement treated base (CTB) sections as well for comparison. An FWD was taken to the site shortly after it opened in 1996, one year after the fact and finally in August of 2000 after four years of traffic. The latter measurement is being presented in this context. Surface characteristics measurements were done on a yearly basis.

Table 1: Layer Thickness used in the model [mm]

Construction Type	Asphalt Concrete Base	Runbase	Cement Treated Base
Asphalt Bound Layers (including wearing coarse)	170	175	90
Cement Treated Base	0	0	240
Unbound Layers	1000	1000	1000

Figure 1: Field Site, a rural two-lane Highway 55. Photo courtesy of Rune Fredriksson.

A backanalysis was made using the software CLEVERCALC 3.9 as to derive the modulus of the layers in the structure. As is a common procedure layers were grouped into their respective material groups, asphalt bound, cement bound, unbound and subgrade. The Runbase layer was treated as an asphalt bound layer. Full friction was assumed between layers. If less than full friction prevails with this assumption the underlying layer would be backcalculated soft, but no such conditions were found. The day of the year (August 10) and a measured pavement temperature of 20°C did support that full friction really existed. For some sections a very high subgrade modulus was backcalculated, most likely due to a rigid base, i.e. bedrock near the surface. As it is important to compare the different sections on an equal basis, all sections with a subgrade modulus higher than 200 MPa were excluded in the comparison.

A built-in statistical package in CLEVERCALC 3.9 was used to derive the following characteristics for the three sections, shown in Tables 2-4. The average, standard deviation (Std.Dev), coefficient of variation (CoV), minimum (Min) and maximum (Max) values are shown for a number of variables. Namely, Asphalt Bound Layers E(1), Cement Bound Layer E(2), Unbound Layers E(3), and subgrade E(4). Next comes the pavement temperature, and the critical (highest) strains for the respective layers, (Strain-I). RMS-error is the fit of the backcalculated basin in percent per sensor. D0 is the center deflection, the Impulse Stiffness Modulus (ISM) and Curvature based on D0 and D30 (cm) are also given in the tables.

Table 2: Derived E-moduli, strains and other data for the asphalt bound base. N=204

Variable	Average	Std.Dev	CoV	Min	Max
E(1)	2963	903	0.305	1346	5451
E(3)	351.6	74	0.21	145	601
E(4)	125.09	28	0.225	74	188
T°C	19.41	1.75	0.09	16	22
Strain-1	149.43	35	0.232	83	250
Strain-3	96.127	26	0.272	50	227
Strain-4	72.245	18	0.252	43	123
RMS-error	1.5529	0.625	0.402	0.5	3.2
D0	302.49	74	0.244	188	547
ISM	310.68	42	0.135	236	449
Curvature	264.66	78	0.296	121	521

Table 3: Derived E-moduli, strains and other data for Runbase. (N=82)

Variable	Average	Std.Dev	CoV	Min	Max
E(1)	2768	765	.276	1732	5426
E(3)	332	47	.143	230	420
E(4)	115	24	.209	81	177
T°C	19.3	1.55	.08	18	21
Strain-1	155	34	.217	92	226
Strain-3	98	22	.221	68	152
Strain-4	76	19	.246	47	122
RMS-error	1.372	.496	.362	.4	2.5
D0	314	74	.235	209	501
ISM	324	36	.110	258	419
Curvature	257	71	.278	143	250

The backcalculated stiffness of Runbase seems to be marginally lower than the for the asphalt concrete base, which is consistent with experience. In fact all parameters are within the statistical margin and there is no difference between the two sections. The CTB sections showed considerably lower deflections due to the high modulus of the cement-treated base itself. Bear in mind though that much less strains are allowed than for asphalt materials and that they are prone to crack, after which a higher deterioration rate will occur.

Replacing the Runbase modulus with those recommended in the Swedish Design Criteria and calculating the strain at 95 mm down in the asphalt layer there is a seven-fold increase in strain. The subgrade strain criterion is less affected, but only 78% of the traffic would be allowed using those numbers rather than the backcalculated ones.

Table 4: Derived E-moduli, strains and other data for CTB. (N=26)

Variable	Average	Std.Dev	CoV	Min	Max
E(1)	3784	3046	0.805	1866	15508
E(2)	21329	12724	0.597	5932	43073
E(3)	491.19	211	0.43	184	1016
E(4)	151.73	27	0.181	100	199
T°C	21.196	0.201	0.009	21	22
Strain-1	6.5	3.68	0.566	1	19
Strain-2	19.154	13	0.667	6	53
Strain-3	26.115	7.54	0.289	14	45
Strain-4	28.462	7.31	0.257	19	46
RMS-error	1.0885	0.531	0.488	0.5	2.5
D0	125.5	31	0.248	85	195
ISM	130.35	13	0.099	111	152
Curvature	1147	338	0.295	541	1922

The pavement performance for the three sections was monitored with a surface characteristics vehicle on a yearly basis. As far as the roughness is concerned the CTB performs better than the other two types as can be seen in Figure 2. In this climate zone, a thirteen meter wide two-lane road can be expected to have an initial IRI of around 1 m/km. Thus, both Runbase and the CTB perform better than that. The traditional asphalt bound base shows a normal development. Note that the rate is different with the traditional Asphalt Base and CTB having a three times higher rate than Runbase for the three-year period. See also Table 5.

Table 5: Annual change of IRI [m/km] and expected value after 10 and 20 years in service.

Material	Rate per year	10 Yr Linear Prediction	20 Yr Linear Prediction
Asphalt Bound Base	.050	1.64	2.14
Runbase	.016	1.11	1.27
CTB	.056	1.26	1.82

The rutting data seem to support that all sections are better than what is normally the case for this type of road. See Figure 3. Low initial rutting rate may be due to the high subgrade modulus, which in turn lends itself to better compaction, as little compaction energy is being wasted on damping in the soil. The CTB has a higher average value after four years of service which is somewhat surprising. The 95 percentile is also rather high near 8.5 mm. This could indicate that some sections are beginning to crack, in turn resulting in a higher stress state in the unbound subbase layer.

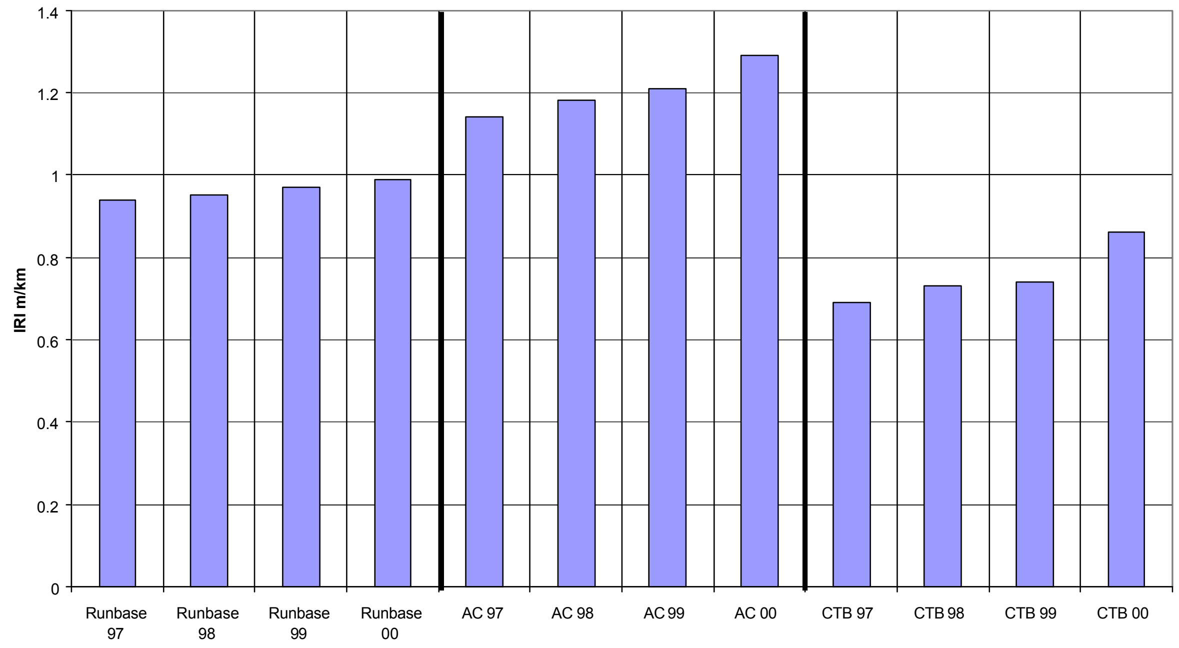

Figure 2: Roughness development for the first four years of service.

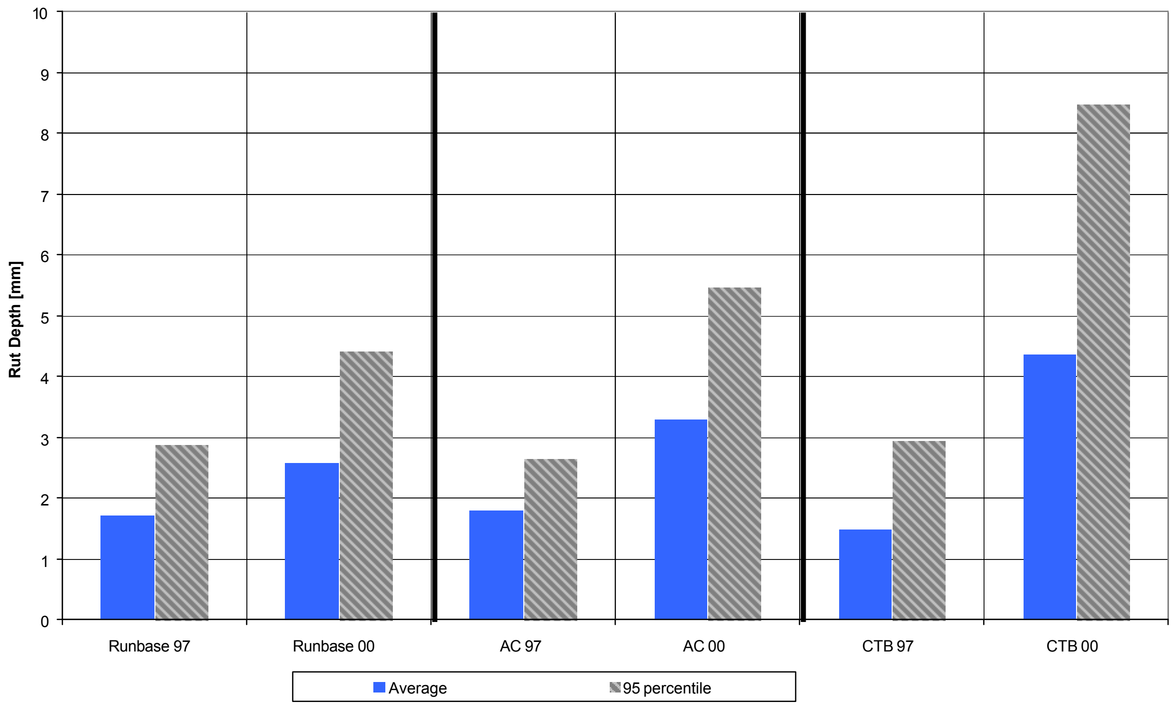

Figure 3: Rutting development for the second and fourth years of service.

5 CONCLUSIONS AND RECOMMENDATIONS

As any pavement engineer knows, sometimes unusual materials are encountered in road structures. The question then arises if it is possible to, for instance, backcalculate a layer modulus of a material that is composed of two or several different materials.

The first recommendation here is to give it a try. If the achieved rms-error of the backcalculated basin is within three percent then it is very likely that the model as such could be considered robust for e.g. overlay design purposes. Make sure that the spatial variation is not higher than that for the subgrade. (This phenomenon would occur for large mesh steel reinforcement near the surface).

Heterogeneous layers may be used in linear layer mechanistic design. The deformation criterion (top of the subgrade strain) needs not to be modified, only because some layers are heterogeneous. Fatigue criteria for other layers than the heterogeneous one is also to be treated as usual. Any criterion for fatigue or deformation for the heterogeneous layer must be adjusted to the pavement performance derived for it.

The grouted macadam coated with an open-graded asphalt concrete mix described in the present paper serves as an example of how a heterogeneous layer could be used in the backcalculation procedure. If the Swedish Design Criteria for moduli are used and calculating the strain at 95 mm down in the asphalt layer there is a seven-fold increase in strain that does not make sense for this type of design modeling. The subgrade strain criterion is less affected, but only 78% of the traffic would be allowed using those numbers rather than the backcalculated ones. Apparently, the former fatigue criterion cannot be used, it must be adjusted entirely. The latter criterion might need some tweaking also. It is the present author’s opinion though that the backcalculated values should be used instead. The performance shown was actually on par or better than the reference asphalt bound base.

For people trying to invent new products it is best to check if the layer will fit the model. If it does by all means find out if pavement performance can be related to any of the existing structural parameters. If it cannot, some other data points may have to be taken into consideration. The old top of the subgrade criterion is soon to be replaced by layer deformation parameters anyway.

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