Modelling of Pavement Response From a Field Test

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ABSTRACT: Modelling of pavement response is central in mechanistic pavement analysis and the validity of response models is hence an important prerequisite for reliable evaluation of structural pavement condition. With the objective of verifying predicted pavement response, an instrumented field test pavement was established on top of a sandy material sufficiently thick to be considered a halfspace. Later a layered flexible pavement system was constructed on top of the halfspace. Falling Weight Deflectometer (FWD) loads were applied at each layer of the pavement to provide input to built-in instruments, which registered stress and strain in three orthogonal directions in the sandy subgrade. At each pavement layer additional FWD measurements were carried out at several locations to determine elastic layer moduli. Based on layer moduli, pavement response was predicted using response methods ranging from a simple Boussinesq model to finite element methods. Comparison between the predicted and observed pavement response showed very good agreement for strain response while the agreement was more questionable for stress. The paper shows that although it was difficult to establish a completely homogeneous test field, several response models reliably predicted strain response in a halfspace, while a three-dimensional finite element program considering the dynamic load situation performed best in the prediction of response in a multilayer pavement system.

KEY WORDS: Pavement, testing, modelling, response, FWD.

1 BACKGROUND

Structural pavement condition is mainly evaluated on the basis of nondestructive tests, which provide pavement surface deflections caused by a known load, which is comparable to normal traffic loads. Several types of (backcalculation) models are capable of relating the surface deflections to Young's modulus, E, for each of the different pavement layers and the natural subgrade. With known stiffnesses and thicknesses of all layers in the pavement structure the remaining life time of a pavement section or the thickness of a new overlay can be determined using information regarding the expected traffic loading. Most state-of-the-practice pavement design procedures apply a mechanistic-empirical approach, where the mechanistic part determines stress and strain in characteristic locations in the pavement, while the empirical part determines the critical stresses and strains. Based on the current pavement condition, the future pavement condition can be predicted for different maintenance strategies or budgets.

Over the years pavement tests have indicated a lack of agreement between in situ measured pavement response from traffic loading (or similar loading) and response calculated using commonly accepted theoretical response models (e.g., Frölich, 1934 and Ullidtz, et al., 1996). This may constitute a problem for the pavement designer who aims at optimising actual pavement design response parameters within limits provided by allowable design response parameters. The empirical component of most current pavement design methods calibrates an analytical prediction of pavement life to what is experienced for in-service pavements. If this practical calibration is conducted, the practical consequence of an incorrect estimation of pavement response may be negligible. If, however, pavement response predictions are used without proper, local calibration, the resulting pavement design may be flawed with economical consequences for either the road authority or the road user. Incorrect pavement response models may also be problematic for the planning of laboratory tests, which require control of the applied stress or strain regimes.

The main purpose of the research project reported here was to evaluate current methods for determination of pavement response to see if the methods can correctly predict the response (Hildebrand, 2002). Finding a solution to this issue is seen as a major step towards improving the quality of pavement analysis and design both for the researcher and the practitioner.

2 CONSTRUCTION OF TEST SITE

A field test was chosen for the study to overcome a potential problem of earlier tests which indicated problems with pavement response models: those tests were carried out in indoor facilities with possible problems related to borders (concrete floors or walls). The first aim was then to test the simplest case: a halfspace, and next a multilayered pavement system. To aid the installation of sub-surface response instruments it was desirable to have a thick layer of a fine-grained material which should constitute a halfspace and the test pavement should moreover be homogeneous in the plane.

2.1 Test site layout

The test site was established at a quarry near Roskilde, Denmark. The test pavement had a size of 6 metres by 10 metres and was divided into 1 metre by 1 metre squares (Figure 1).
The influence from climate on the condition of the (especially unbound) materials at the test site was limited through the use of a tent, which covered the entire test area during the period where the unbound materials were exposed.

2.2 Pavement structure

The pavement was designed to have a significant magnitude of subgrade material constructed on top of the natural subgrade to constitute a halfspace. Later a granular base layer and an asphalt surface layer were added (Figure 2).

2.3 Quality of test pavement construction

The construction quality control showed a practically homogeneous pavement with regard to layer thickness, density, water content, and compaction. Hence, it was concluded that the test site had fairly homogeneous layers and constituted an approximate halfspace.

The asphalt surface layer was a densely graded material typically used in Denmark for high-stability asphalt base layers. The material consisted of both crushed and un-crushed quarry material (maximum grain size 16 mm), sand and fillers (7 percent weight) mixed with a penetration 51 binder. A binder content of 5.2 percent was used. Laboratory tests on asphalt cores showed a relative compaction of 96 percent of the Marshall density and air voids of 7.8 percent.

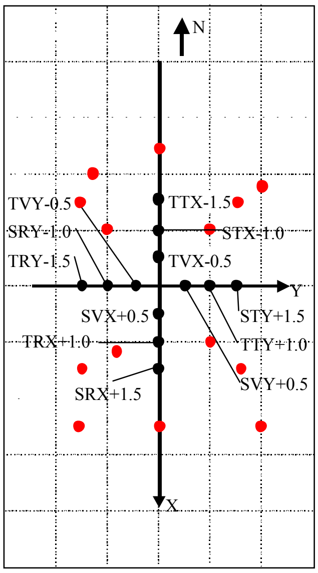

Figure 1: Test site layout. Letter codes represent instruments, red dots show FWD test locations.

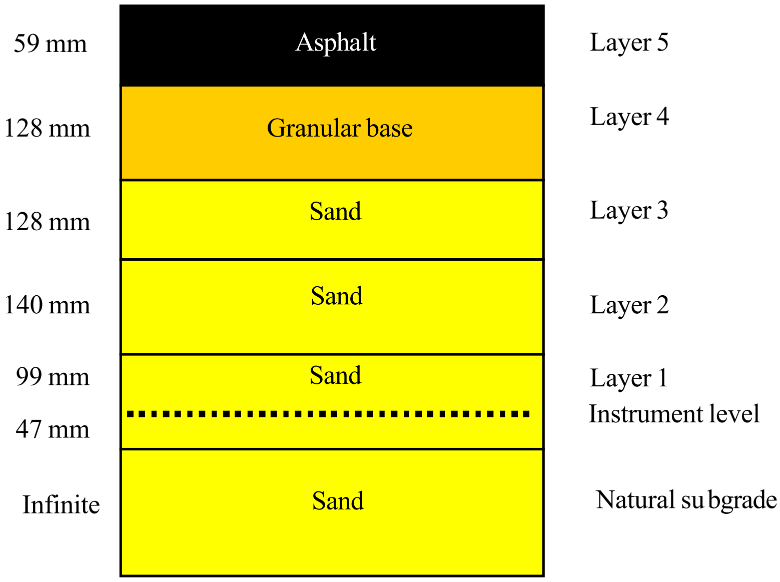

Figure 2: Pavement structure at test site.

Table 1: Pavement material properties for unbound materials.

Layer	In situ dry density (kg/m^3)	In situ water content (%)	Degree of Compaction (%)	Particle size distribution
Base	2000	4.5	93	Material > 32 mm: 0%; material > 16 mm: 42%; material < 75 µm: 3.6%;
Sand layer 3	1700	11	96	Material < 75 µm: 15%.
Sand layer 2	1650	11	98	Material < 75 µm: 7%.
Sand layer 1	1665	11	94	Material < 75 µm: 25%.
Subgrade	1700	12	100	Material < 75 µm: 8%.

Two geotechnical borings carried out after the experiment period showed the existence of an approximately 3 metres deep fine-grained sand layer below the pavement, such that the halfspace had a thickness of 3.3 metres. Below the sand material, clay and silt were found for 2.5 metres and below these 4 metres of moraine clay was discovered. Below the moraine clay, a dense granular layer with hard and dry stones existed for at least 5 metres. All unbound materials were characterized as non-plastic.

3 DETERMINATION OF PAVEMENT PARAMETERS

Parameters regarding material stiffness and behaviour (Young's modulus, E, Poisson's ratio, υ, stress dependencies, etc.) in the laboratory and at the test site were necessary to evaluate the pavement condition and to provide a basis for the subsequent pavement response modelling. The data also provided important input to an evaluation of homogeneity of the test site. The two main types of tests were repeated-load triaxial tests and falling weight deflectometer tests.

3.1 Repeated-load triaxial tests

Repeated-load triaxial tests were carried out on a blend of the natural and constructed subgrade material. Tests were planned to simulate the in situ conditions with regard to compaction, density, water content, loading magnitude, and loading pulse. Five samples were tested according to a test pattern aiming at covering the stress regimes experienced at the test site. Results were given in terms of E, and modelling was done with different constitutive models for pavement materials. It was found that the sand material had a Young's modulus in the range from 62 MPa to 128 MPa for the conditions tested.

The material was found to be stress-hardening, and E could be described as a function of bulk stress and octahedral shear stress. A very fine fit was found between experimental data and resilient moduli determined with a model, which considered bulk stress, octahedral shear stress, compaction and saturation.

3.2 Falling weight deflectometer tests

Standard FWD load tests at 10-14 points were carried out on all layers except layer 1 (cf., Figure 1). FWD load magnitudes were planned to avoid severe damage to the pavement, and no tests were made on top of sub-surface instruments. E-moduli for the pavement layers were determined using the backcalculation programs MODCOMP5 (Irwin, 1994) and FeBack (Ullidtz et al., 1999), which are both capable of performing linear as well as nonlinear, i.e., stress dependent, analyses. One important difference between the two programs is that MODCOMP5 considers the overburden stress in its nonlinear constitutive material models, while FeBack does not.

3.3 Overall, linear pavement layer moduli

The study of pavement parameters showed good agreement between E from laboratory and FWD. Table 2 summarizes the overall moduli determined from repeated-load triaxial tests and from FWD tests. A thorough statistical analysis of E moduli found the test site to be homogeneous. The analysis also found E of the constructed sand layers (layers 1-3) lower than E for the natural subgrade (i.e., a modulus gradient existed in the halfspace). The results in Table 2 provided input to the response modelling detailed in Chapter 5.

Table 2: Pavement layer parameters based on FWD and triaxial tests.

Natural subgrade	Sand layers 1-3	Base layer 4	Asphalt layer 5
100 (55-140)	55 (30-140)	270 (130-450)	5700 (3000-7700)
All moduli are reported in MPa. Asphalt moduli are adjusted to 15 °C. ( ) represents a range of calculated moduli.			

4 PAVEMENT RESPONSE OBSERVATIONS

Pavement response was registered at the test site in part to demonstrate that it is possible to collect good quality in situ pavement response information and in part to collect in situ pavement response and compare with theoretically predicted response.

4.1 Instrumentation

Two types of pavement response were observed: strain and stress. The sub-surface instruments applied were developed at The Technical University of Denmark and have shown good performance through several experiments in the Danish Road Testing Machine. The pressure cell is a hydraulic cell with a double membrane between which an incompressible liquid is found. The soil deformation transducer is based on a standard LVDT, which has been modified to withstand the often harsh environment in a pavement. The instruments were calibrated before installation and again after they had been recovered 18 months later - all survived with un-changed calibrations. Data collection took place in the field with a laptop computer as the core of the system from which data acquisition was initiated and to which data were stored.

Six soil pressure cells and six soil deformation transducers were installed on top of the natural subgrade to become integral parts of layer 1. Each type of instrument was installed double and in three orthogonal directions to allow registration of two horizontal and one vertical component of strain as well as stress. The instruments were spaced 500 mm apart to avoid disturbance from one instrument to the next. The layout of the instrumentation can be seen in Figure 1.

All gauges were named with three letters, and a signed number. The first letter is S (SPC) or T (SDT), the second letter (R: horizontal radial, T: horizontal tangential, or V: vertical) describes the orientation of the instrument's sensing element relative to the instrument line, while the third letter (X or Y) describes the instrument line at the test site. The signed number provides the gauge's horizontal distance from the origin of the test site co-ordinate system.

4.2 FWD response testing

FWD response tests were carried out as 75-mm stepwise tests at each layer surface (except layer 1) on top of the two instrumented lines of the test site. A FWD response test was carried out as one seating drop followed by one drop at a load level planned to provide optimum response signals. One load level was selected for each layer surface. During the FWD response testing a hydraulic pad was mounted below the FWD load plate to secure uniform load distribution to the pavement. For each FWD response test location, the peak of the dynamic response of each instrument in layer 1 was determined and stored.

4.3 Observed response data

The response data were plotted as shown in Figure 3. The graph shows peak responses illustrated as influence lines, which look approximately like if a wheel had been driving along the measurement line. Figure 3 shows that the maximum vertical response for stress is found when the load is applied directly on top of the instrument. This is also true for horizontal, tangential stress, while the maximum horizontal, radial stress is found approximately 500 mm to each side of the instrument location.

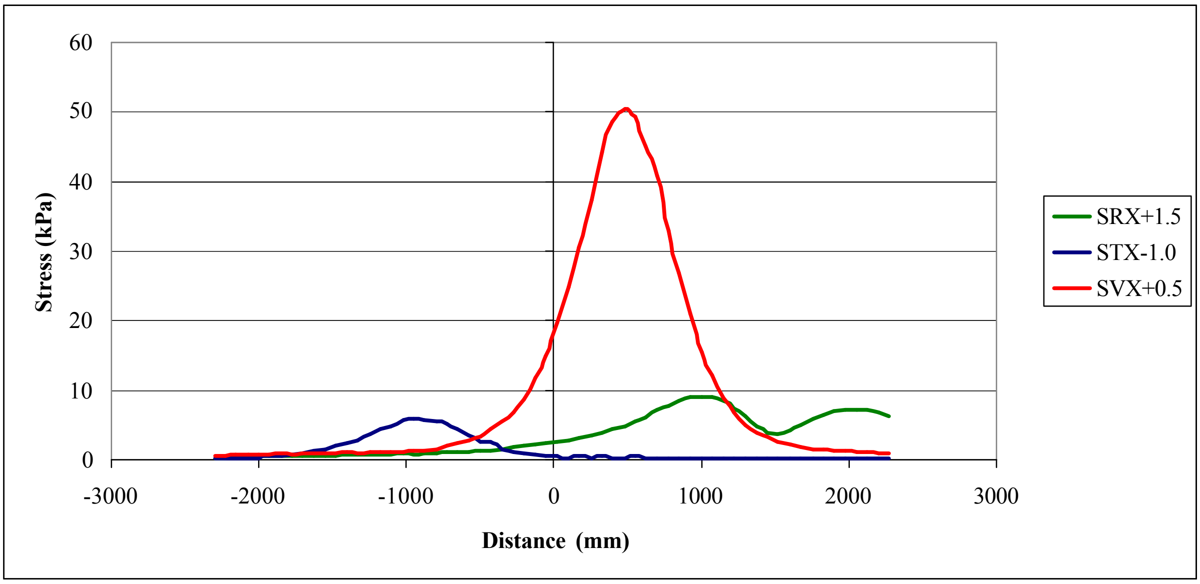

Figure 3: Observed stress in layer 1 as a result of FWD response testing in X direction on layer 5.

Figure 3 displays very clear response curves for stress (similarly is found for strain). It is interesting to note that the two horizontal stress components are very small compared to the vertical stress. There is not complete agreement (which should be expected) between the peak values of the two horizontal stress components. The same applies for the horizontal strains as well.

4.4 Quality control of stress observations

Using the vertical pressure distribution in Figure 3 and assuming axial symmetry, the pressure distribution should integrate to the applied FWD force. The validity of this statement was tested for response tests on all layer surfaces. Large differences between applied and integrated forces were found, however, with the difference increasing with the addition of more layers. The possible reasons for this is not known but it may be related to installation, calibration and sensitivity of the pressure cells or it may be related to lack of appropriateness of the integration method. As the reason for the observed lack of agreement was unknown, peak vertical stresses were corrected with the appropriate factors.

Figure 5 illustrates very low responses from the horizontal pressure cells. The low response at the very low end of the pressure cell measuring range may lead to problems with reduced accuracy of horizontal stress measurements, which means that the use of the measurements of horizontal stresses was limited.

4.5 Homogeneity and isotropy of layer 1

Based on the registered response it was found that the test site was homogeneous. From Hooke's law and response registrations in three orthogonal directions on four different layers, E and ν were determined using linear regression analysis. Hooke's law:

$$E \cdot \varepsilon_x = \sigma_x - \nu \cdot (\sigma_y + \sigma_z) \tag{1}$$

can be re-arranged to:

$$\sigma_x = E \cdot \varepsilon_x + \nu \cdot (\sigma_y + \sigma_z) \tag{2}$$

and similarly for σ_y and σ_z. From these expressions E and ν were determined: With the corrected vertical stresses E was found as 44 MPa, while E was found as 65 MPa with the original vertical stresses. In both cases, ν was 0.50.

5 MODELLING OF PAVEMENT RESPONSE

Calculated and measured pavement response were compared for response tests carried out on pavement layers 3 (sand) and 5 (asphalt). The data allow an analysis of response in the halfspace (consisting of the natural and constructed subgrade) as well as in the multilayered pavement system (consisting of the entire pavement structure). At this place, only the multilayer system will be evaluated. Backcalculated moduli from the FWD testing on the asphalt layer form the basis for the predictions of pavement response, which was carried using the following response models:

- BISAR
- Method of equivalent thicknesses
- NELAPAV 4
- FeBack (FE, finite element)
- CAPA-3D

CAPA-3D was only applied for the multilayer system, while the remaining models were used for both situations.

5.1 Response models

BISAR
BISAR (De Jong et al., 1973) was evaluated as an example of a simple linear elastic, multilayer method.

Method of Equivalent Thicknesses
The MET routine of FeBack was evaluated as an example of a simple model with a single nonlinear layer (the natural subgrade).

NELAPAV 4
NELAPAV 4 (Irwin, 2000) was used to predict pavement response using the constitutive model (deviator stress) and corresponding parameters determined by nonlinear backcalculation in MODCOMP5.

FeBack(FE)
The finite element (FE) part of FeBack was applied to predict pavement response based on the backcalculation performed with the FE routine.

CAPA-3D
CAPA-3D (Scarpas et al., 1997) was used as an example of an advanced finite element program, which can include dynamic loading and nonlinear material models. CAPA-3D applies a visco-plastic constitutive material model. For the specific purpose a four-layer

pavement system (asphalt, base, layers 1-3, and natural subgrade) was created in CAPA-3D using a mesh consisting of 4480 twenty-node cubic block elements. The mesh extended six metres in both horizontal planes and seven metres in the vertical direction; due to symmetry only one quarter of the problem was analyzed.

The linear moduli found from the FWD backcalculation (Table 2) were input together with the FWD load history, the latter providing an opportunity to perform a dynamic analysis.

5.2 Results of pavement response modelling

An example of the results of the pavement response modelling is shown in Figure 4, which shows the results for vertical strain in layer 1 as a consequence of FWD loading on layer 5. Similar results were found for stress for testing on layer 5 and for both stress and strain for testing on layer 3. The response curves in Figure 4 are plotted with offset distance zero at the location of the sub-surface instrument.

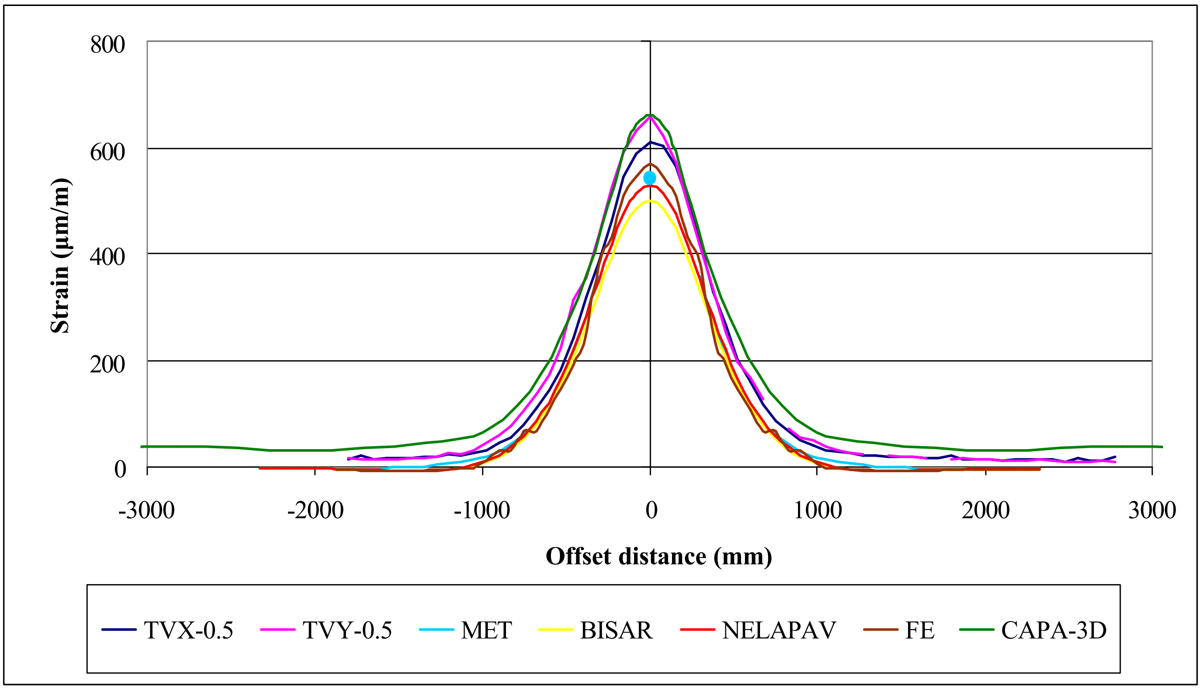

Figure 4: Modelling of vertical strain for FWD testing on layer 5.

Figure 4 shows that CAPA-3D models one of the observations of vertical strain almost perfectly around the location of the peak, while the agreement between CAPA-3D and the other observation is very good and much better than for the other models. Table 3 summarises the results of the response modelling for loading on layer 5.

Table 3: Summary of estimated peak responses in layer 1 based on FWD loading on layer 5.

Method	σ_r	σ_t	σ_z	ε_r	ε_t	ε_z
BISAR	9.4	9.2	34.3	-187.1	-185.6	506.0
MET	11.7	11.8	32.6	-167.4	-166.7	545.2
NELAPAV 4	6.5	6.4	31.4	-206.5	-205.2	534.3
FeBack(FE)	16.2	16.1	41.6	-162.2	-166.3	571.8
CAPA-3D	11.0	10.9	41.0	-254.9	-255.7	661.7
Measurement	3.6	5.2	50.5	-283.1	-282.8	633.3
Corrected measurement	2.4	3.5	34.1	-283.1	-282.8	633.3
Note: Stress is reported in kPa and strain is reported in μm/m.						

6 EVALUATION OF PAVEMENT RESPONSE MODELS

Modelling of stress and strain for the multilayer system shows that CAPA-3D yields the best prediction of pavement response. CAPA-3D, which performed best when used with a dynamic, linear elastic response model, predicts all three strain components very well. Several of the models predict stress reasonably, but the questionable validity of the stress measurements make it difficult to rate the models based on stress.

Strain was found much easier to predict correctly than stress. The overall deviation between observed and predicted peak strain for all models for both pavement systems was approximately 20 percent. The overall deviation between observed and predicted vertical stress is approximately 35 percent. The reason for this difference may be that stress is difficult to measure or it may be because the stress measurements at the test site, unfortunately, seem to be flawed. The correction to the vertical stresses improves the agreement between observation and prediction somewhat, but it has still been found difficult to obtain reasonable agreement for vertical stress. For horizontal stress it was found impossible to obtain agreement. The problems related to the stress measurements are not fully known, but it may well be related to instrument calibration or installation. An alternative reason for the problem could, however, also be that the applied response models do not describe granular soil behaviour properly.

The clear, overall conclusion of the analysis is that the most advanced program, CAPA-3D, which takes into account the dynamics of the actual FWD load pulse produces the best agreement between in situ observed and calculated pavement response. For the remaining programs, it is difficult to select a program, which is better than any other.

7 CONCLUSIONS

Based on the reported study it can be concluded that although field verification is difficult it is possible to construct an approximately homogeneous and isotropic test site suitable for pavement response verification. Based on the research carried out at the test site in Denmark it was found that it is difficult to predict stress while strain is predicted well by several models. The overall conclusion is that a dynamic, linear finite element model provided the best agreement between measured and predicted pavement response in a multilayer system.

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