1. Introduction

Arraigada et al. (2009) have been studied the use of accelerometers to measure pavement deflections due to traffic loads. The finite element models (FEMs) revealed the inability of the accelerometers to measure very slow or quasi-static motion. Chea and Martinez (2008) were carried out three-dimensional finite element (3D FE) simulations of the deflection under a standard axle load in order to detect interface flaws between bituminous and hydraulic layers of composite pavements.

Most backcalculation programs used to evaluate the pavement layer properties assume static deflections even though dynamic deflections are generated from the Falling Weight Deflectometers (FWD). Losa et al. (2008) proposed a statistical model for the straight evaluation of critical strains in pavements by using the deflections measured by the FWD and the layer thicknesses without backcalculating layer moduli. The model was calibrated on the basis of experimental data and it is useful to evaluate statistical parameters of the homogeneous sub-sections with the aim to evaluate the residual pavement life taking into account the reliability concepts. A pseudo-static backcalculation procedure Dynamic BALMAT (DYN-BAL) was developed by Seo et al. (2009) to calculate the layer moduli after converting dynamic deflections into static deflections. From the test results, it was found that DYN-BAL gives the most reliable results when compared with several other computer codes in use. The results of Bayrak and Ceylan (2008) study demonstrated that the ANN-based models, which were trained to predict the layer moduli by using the FWD deflection basin data and the thickness of the concrete pavement structure, are capable of successfully predicting the rigid pavement layer moduli with high accuracy.

A dynamic analysis based on the spectral element method Grenier et al. (2009) described for the interpretation of FWD tests on flexible pavements. While the deflection basin currently used in static methods gives some details of the pavement response under transient loading, the simulations of FWD tests using the dynamic model suggest that the time histories should be included as well for the interpretation of FWD deflection measurements. In fact, important dynamic phenomena due to inertial effects and viscous effects are only revealed by deflection histories. Grenier and Konrad (2009) presented a robust backcalculation methodology that uses the Levenberg-Marquardt iterative minimization technique to identify the value of unknown layer parameters from FWD tests using a dynamic approach based on the spectral element.
method, too. The efficiency of the proposed methodology is demonstrated by interpreting FWD tests on three flexible pavements that cover a variety of structures, soil, and bedrock conditions. Results indicate that the dynamic approach is capable of simulating quite well the measured deflection histories using effective backcalculated moduli. In addition, comparison of critical strains between static and dynamic interpretation of FWD tests indicates that both approaches predict similar traction strains at the bottom of the asphalt concrete layer. However, the prediction of the compression strain in the subgrade with the static approach is erratic compared with the dynamic method.

Donovan and Tutumluer (2009) presented a methodology based on analyzing FWD test data between trafficked and non-trafficked lanes to determine the degradation and rutting potential of flexible pavement unbound aggregate layers in comparison to the subgrade damage.

According Dawson et al. (2009) several procedures can be used for the determination of the resilient modulus: laboratory testing, backcalculation with Non-Destructive Testing (NDT) data, and correlations to other soil parameters (California bearing ratio, density, and water content). Backcalculation with NDT data procedure is relatively inexpensive and fast and can be designed to cover representative soils under the pavement network. NDT devices are used to determine pavement structural capacity and for pavement condition assessment. FWD are mostly used NDT devices all over the world because of the testing accuracy, repetitiveness and similarity to the real loading magnitude and duration.

Since using the NDT devices, many different parameters have developed describing their deflection basins. The main purpose of the parameters is to evaluate whole pavement or single layer condition. Widely used FWD deflection basin parameters (DBPs) are presented in Table 1. Different researches (Kim et al. 2000; Park 2001; Tiehallinnon 2006) have shown their utility possibilities as calculating pavement layers modulus of elasticity ($E$ modulus) or assessing pavement structural condition. Current research focuses on the three basic DBPs ($SCI$, $BDI$, $BCI$) and is trying to find relationship between FWD deflections and pavement condition:

- **Surface Curvature Index ($SCI$)** – difference of deflections measured with load cells in the center of the loading plate ($d_0$) and 300 mm from the center ($d_{300}$): ($d_0 - d_{300}$), which is characterizing condition of the pavement layers;
- **Base Damage Index ($BDI$)** – difference of deflections measured with load cells in the distance 300 mm ($d_{300}$) and 600 mm ($d_{600}$): ($d_{300} - d_{600}$), which is characterizing condition of the base layers;
- **Base Curvature Index ($BCI$)** – difference of deflections measured with load cells in the distance 600 mm ($d_{600}$) and 900 mm ($d_{900}$); ($d_{600} - d_{900}$), which is characterizing condition of the subbase or subgrade.

### Table 1. Widely used FWD deflection basin parameters (Kim et al. 2000; Talvik 2007)

| Deflection basin parameter                  | Equation                                                                 | Unit       | Parameter’s objective                                  |
|---------------------------------------------|--------------------------------------------------------------------------|------------|--------------------------------------------------------|
| Surface Curvature Index                     | $SCI = d_0 - d_{300}$, $SCI = d_0 - r$, (used also $r \in [450, 600]$) | μm, mm     | Characterizing condition of bound layers               |
| Base Damage Index                           | $BDI = d_{300} - d_{600}$                                                | μm, mm     | Characterizing condition of base layers                |
| Base Curvature Index                        | $BCI = d_{900} - d_{600}$ (used in USA)                                  | μm, mm     | Characterizing condition of subbase or subgrade       |
|                                             | $BCI = d_{1200} - d_{1500}$ (used in Estonia)                            |            |                                                        |
| Area                                        | $AREA = \frac{6(D_0 + 2D_1 + 2D_2 + D_3)}{D_0}$                        | mm         | Characterizing shape of the deflection basin close to the load by the normalized area on the top of the deflection basin |
|                                             | $AREA = \frac{150(d_0 + 2d_{300} + 2d_{600} + d_{900})}{d_0}$           |            |                                                        |
| Area under pavement profile                 | $AUPP = \frac{5d_0 + 2d_{300} + 2d_{600} + d_{900}}{d_0}$              | mm         | Characterizing condition of the pavement upper layers |
| Shape factors                               | $F_1 = \frac{(d_0 - d_{600})}{d_{300}}$, $F_2 = \frac{(d_{900} - d_{600})}{d_{600}}$ | -          | Determination of condition of the layer at the equivalent depth |
| Deflection ratio                            | $DR = \frac{d_{600}}{d_0}$                                             | -          | Determination of condition of the layer at the equivalent depth |

Note: $d_0, d_{300}, d_{600}, d_{900}, d_{1200}, d_{1500}$ – measured deformations at the distance of 0, 300, 600, 900, 1200, 1500 mm from the center of the loading plate; $D_0, D_1, D_2, D_3$ – measured deformations at the distance of 0 ft, 1 ft (305 mm), 2 ft (610 mm), 3 ft (914 mm) from the center of the loading plate.
2. Initial database of the research and analysis of data

The aim of the current research was to study employing of FWD deflection basin parameters for pavement condition assessment in Estonia. As the analysis had to rely on larger data than 26 control FWD measurement points (measured 1999–2006 every year) used in the researches until now, it was decided to construct database based on state road network data. The data of defects (29790 100 m sections), rut depths (24333 100 m sections) and FWD measurements (37936 points) was derived from the Estonian Road Data Bank to the initial database. Additionally, different pavement types and traffic loadings were taken into account.

2.1. Analyze groups of the research and analysis of data

As pavement design depends on forecasted traffic loading in the end of service life, it was purposeful to divide analyze groups according to traffic loadings. It has to be mentioned that most Estonian roads have reached the end of their service life, but the traffic loading for the current analysis was determined according to the actual traffic volumes based on the counting data of 2006. Estonian Standard for Road Design (Metsvahi et al. 2005) is determining required min equivalent $E$ modulus ($E_{\text{eq}}$) and related forecasted traffic loadings and those were used for dividing data into analyze groups (Table 2).

| Analyze group | Traffic loading, standard axle load (100 kN), vpd | Required $E_{\text{eq}}$, MPa |
|---------------|----------------------------------|-----------------------------|
| 1             | < 30                             | 140                         |
| 2             | 30–59                            | 160                         |
| 3             | 60–114                           | 180                         |
| 4             | 115–224                          | 200                         |
| 5             | 225–439                          | 220                         |
| 6             | 440–869                          | 240                         |
| 7             | > 870                            | 260                         |

2.2. Transformation of measured deflections to the standard load level and standard temperature

During the standard FWD measurement the dropped weight and dropping height are always the same, but in reality, the load applied to the pavement depends on site conditions. Applied load ($p_{\text{measured}}$) is affected by the pavement stiffness, its surface profile and the properties of the FWD device.

To have comparable deflection values, they have to be normalized to the standard load by multiplying by the factor ($p_{\text{target}}/p_{\text{measured}}$). In our case the target load is 50 kN as standard axle load used in Estonia for pavement design is 100 kN. The contact pressure equivalent on a 300 mm diameter plate for 50 kN load is 707 kPa according COST 336:1999 Falling Weight Deflectometer.

$E$ modulus of the bituminous-bounded layers is dependent on the temperature. Therefore, measured deflections of the same structure at different temperatures are different and depending on the stiffness of the bituminous-bounded layers. During the FWD measurements the temperature of the bituminous pavement can vary in the range $+5 \ldots +35 \, ^\circ C$. As result of this the measured deflection values have to be corrected to the standard temperature. According to the Estonian guidelines for flexible pavement design Procedure 2001-52, in the case of calculation of the pavement structure to the elastic deformation, standard temperature is $+10 \, ^\circ C$. For correction of FWD measured deflection values to the standard temperature ($+10 \, ^\circ C$) can be used temperature correction factors ($K_t$), calculated using Eqs in Table 3 (Aavik 2003), depending on the bituminous pavement type and the average temperature of the bituminous layer during the FWD measurement ($T$).

**Table 3.** Bituminous pavement layer temperature correction factors $K_t$ (Aavik 2003)

| Pavement layer type     | Temperature correction factor $K_t$ |
|-------------------------|-----------------------------------|
| Asphalt concrete        | $K_t = 0.000203 T^2 - 0.014841 T + 1.127603$ |
| Cold bituminous mix     | $K_t = 0.000205 T^2 - 0.015198 T + 1.135192$ |

FWD measured deflections are transformed to the standard load level (50 kN) and standard temperature ($+10 \, ^\circ C$) using following Eq:

$$d_{50kNT} = d_r \times \frac{p_{\text{target}}}{p_{\text{measured}}} \times K_t,$$

(1)

where: $d_{50kNT}$ – deformation at the load 50 kN and temperature $+10 \, ^\circ C$ at the distance $r$ (mm) from the center of the loading plate, $\mu$m; $d_r$ – FWD measured deflection at contact pressure $p_{\text{measured}}$ (kPa) at the distance $r$ (mm) from the center of the loading plate, $\mu$m; $p_{\text{target}}$ – contact pressure, corresponding to the 50 kN load ($p_{\text{target}} = 707$ kPa); $K_t$ – temperature correction factor (Table 3).

2.3. Relationship between Deflection Basin Parameters (DBPs) and pavement defects

In the current research only longitudinal cracking and alligator cracking (fatigue cracking) were examined. Those types of defects are forming if whole pavement or single layers have insufficient structural capacity and therefore bituminous layers submit fatiguing easier.

As data of these surface defects has gathered separately, the Partial Defect Sum (PDS) parameter was taken into use to give better comparison. PDS is describing extent of cracks in % of road surface area on the section of
100 m (Eq 2). The Eq 2 is found on the basis of Defect Sum Eq (used in Estonian Pavement Management System), where are presented all types of defects with their weight coefficients.

\[ PDS = \left( \frac{0.5 \times LCRACK + 1.0 \times ALLIG}{RWIDTH \times 100} \right) \times 100 \% \]  \( \text{Eq} \ 2 \)

where: \( LCRACK \) – length of longitudinal cracks, m; \( ALLIG \) – extent of alligator cracking, \( \text{m}^2 \); \( RWIDTH \) – width of road, m.

Analysis of data showed that there is no definite relationship between DBPs and investigated road surface defects. It was clearly stated that presenting any of DBPs and PDS on the graph, the dispersion of data is extensive. Values of determination coefficients (\( R^2 \)) were less than 0.1 (\( R^2 < 0.1 \)), which is indicating the absence of relationship. For example the \( SCI \) vs \( PDS \) of dense asphalt concrete surface of analyze Group 2 is presented in the Figure 1.

Fig. 1. SCI vs PDS of asphalt concrete surface of analyze group 2

Main reason for absence of the relationship can be the difference between data collection principles: FWD measurements are performed only once per every 100 m and the measurement represents only the condition of the pavement at this exact point, but other condition indicators are collected from all length of the 100 m sections. In addition to that the determinations of defects and FWD measurements have been done in different times.

2.4. Relationship between DBPs and rut depth

Measurements of rutting in Estonian roads are performed two times a year: in the spring and in the autumn. The depth of rut in the spring is usually smaller than in the autumn, because of different driving trajectory with studded tires in the winter. Reducing the affect of studded tires to the data of rut depths and to survey better permanent deformations, rut depths measured in the autumn were only taken into account.

As the FWD measurements are carried out on the spot of the right wheel lane, the rut depth is taken also from the right wheel lane.

It is clearly perceivable in the Fig. 2 that similar rut depths have appeared in the case of different values of \( SCI \). The same situation appeared with other parameters and groups. It is complicated to determine relationships because the rut depths are presented as mean values of the sections, but DBPs represent the pavement condition at the exact point.

2.5. Relationship between Deflection Basin Parameters (DBPs) and pavement equivalent \( E \) modulus (\( E_{eq} \))

The Eq for back-calculation of pavement equivalent \( E \) modulus (\( E_{eq} \)), which is expressed in the BCH 46-83 “Инструкция по проектированию дорожных одежд нежесткого типа” [Guidelines for Flexible Pavement Design VSN 46-83] (the previous Soviet Union flexible pavement design procedure), which derivation the Procedure 2001-52 is as follows:

\[ E_{eq} = \frac{0.25 \pi \times \left( 1 - \nu^2 \right) \times F \times S}{d_0}, \]  \( \text{Eq} \ 3 \)

where: \( E_{eq} \) – pavement equivalent \( E \) modulus at the center of the loading plate, MPa; \( \nu \) – Poisson's ratio (in Procedure 2001-52 \( \nu = 0.3 \)); \( F \) – contact pressure under the loading plate, kPa; \( S \) – diameter of the loading plate, mm; \( d_0 \) – deflection at the center of the loading plate, \( \mu \)m.

The Eq for the calculation of the \( E_{eq} \) comparable with the Procedure 2001-52, taking into account possible different known influencing variables, can be written in the form (Aavik 2003):

\[ E_{eq2001-52} = C \times E_{eq}^C \times T^T \times R^R \times M_i \times H_j, \]  \( \text{Eq} \ 4 \)

where \( E_{eq} \) – pavement equivalent \( E \) modulus at the center of the loading plate, MPa, calculated using Eq (3); \( T \) – mean temperature of the bituminous pavement surface at the moment of FWD measurement, °C; \( R \) – summarized amount of rainfall in 30 days before FWD measurement, mm; \( M_i \) – factor taking into account the month when FWD measurement is performed (\( i = 4, \ldots, 10 \), April–October); \( H_j \) – factor taking into account the height of embankment at the
FWD measurement site \((j = < 0.5 \text{ m}; 0.5–1.0 \text{ m}; > 1.0 \text{ m})\);

As during the FWD measurements carried out in the network level the height of the embankment or amount of the rainfall is not known at every measurement site, the Eq (4) can be transformed as follows:

\[
E_{eq2001-52} = C \times E_{eq}^p \times T^t \times M_i, \tag{5}
\]

where \(e = 0.793; t = 0.098; C = 2.039\) (Aavik 2003) and factors taking into account the month \(M_i\) according to the Table 4.

| \(M_4\) | \(M_5\) | \(M_6\) | \(M_7\) | \(M_8\) | \(M_9\) | \(M_{10}\) |
|-------|-------|-------|-------|-------|-------|--------|
| 1.000 | 0.911 | 0.830 | 0.816 | 0.831 | 0.825 | 0.817 |

In the analysis onwards used \(E_{eq}\) are calculated using Eq (5).

Relationships between DBPs (SCI, BDI, BCI) and back-calculated \(E_{eq}\) (Eq (5)) were analyzed. There were examined separately the pavements with and without surface defects, to determine, if there are differences between distressed pavements and undamaged \(E_{eq}\) and DBPs. In all analyze groups different pavement types were studied separately.

Getting visual survey from characters relationships, they were presented in the dispersion graphs and regression curves were added. It can to be seen from the Figs 3 and 4, that most suitable regression line is power function in the form: \(y = a_0 x^{a_1}\).

It was found, that relationship between DBPs and \(E_{eq2001-52}\) are quite strong. Relations SCI–\(E_{eq2001-52}\) and BDI–\(E_{eq2001-52}\) are described by mathematical functions, which give values of \(R^2\) between 0.5–0.9. In the case of relations BCI–\(E_{eq}\) values of \(R^2\) were smaller than 0.5. Also was found, that among the roads, forming the database, appeared to be different values of BCI (characterizing condition of subgrade) in the case of similar \(E\) modulus. In the case of different values of SCI and BDI (characterizing upper layers) the divergence of \(E_{eq}\) was smaller. This confirms that pavement structural capacity in Estonian roads is assured if there are strong upper layers on the weak subgrades. While the upper layers are also weak, then the whole bearing capacity of the road structure is insufficient. It is illustrated on the Figs 3 and 4, how the value of the DBP was determined according to the min required \(E\) modulus (\(E_{req2001-52}\)).

During the analyses was found, that there exists no tendency, like pavements with surface defects have bigger values of DBPs. Nevertheless, it was recognized, that values of DBPs are decreasing if the traffic loading (or \(E_{req}\)) is increasing. This confirms that pavements with higher structural capacity have smaller deflections under the loading.

As usually the \(R^2\) of mathematical models describing BCI–\(E_{eq2001-52}\) were lower than 0.5 and mostly lower than 0.3, then pavements that are stabilized with complex binders (bitumen + cement) were found to have higher \(R^2\) values (0.55–0.88). This shows that subbases with good structural capacity affect strongly on BCI–\(E_{eq2001-52}\) relationship.

\subsection*{2.6. Determination of limit values for Deflection Basin Parameters (DBPs)}

On the assumptions of preliminary analysis, based on the min \(E_{req}\) of particular pavement, the Eqs were developed for calculation the max limit values of deflection basin parameters for different types of pavements. Graphs, where \(E_{req}\) and DBPs are presented, were composed to as many pavement types as possible. Used mathematical models are power functions, because of the former research which showed non-linearity between parameters and \(E\) modulus:

\[
y = a_0 x^{a_1}, \tag{6}
\]

where: \(x = \text{min } E_{req} \text{ MPa}; y = \text{deflection basin parameter (SCI, BDI, BCI); } a_0, a_1 = \text{constants according to Table 5.}\)
For example, the relationship between \( \text{SCI} - E_{\text{req}} \) of dense asphalt concrete on top of existing pavement (Fig. 5) can be described as follows:

\[
\text{SCI} = 1795660 E_{\text{req}}^{-1.70}, \quad R^2 = 0.83. \tag{7}
\]

![Fig. 5. SCI–E_{\text{req}} AC pavement on top of existing pavement](image)

Based on similar Eqs, it is possible to calculate max limit values for \( \min E_{\text{req}} \) (Table 6).

The Eq for calculation \( \min E_{\text{req}} \) according Procedure 2001-52 is following:

\[
E_{\text{req}} = (a \log(Q) + b)K_{tt} \tag{8}
\]

where \( Q \) – (forecasted) traffic load, standard axle load vpd \((E_{\text{req}} \geq 2)\); \( a, b \) – constants (Table 7); \( K_{tt} \) – pavement strength factor (Table 8).
3. Conclusions

Even though it was not succeeded to identify relationships between deflection basin parameters (SCI, BDI, BCI) and pavement surface deflections, the strong relationship with back-calculated \( E_{eq} \) proved the practical utility possibilities of DBPs. Stronger relationships were found between upper layers indicators (SCI and BDI) and \( E_{eq} \) \( (E_{eq2001-52}) \), as relationship between subgrade indicator BCI and \( E_{eq2001-52} \) found in the research, was not very strong.

Analyses confirm that poor condition of Estonian road pavements is due to weak subbases and subgrades. Pavements that are stabilized with mixed binders (bitumen + cement) were found to be with higher \( R^2 \) of mathematical models representing the relationship between BCI and \( E_{eq2001-52} \).

As the statistical analyses of such extensive database have been done for the first time in Estonia, the determined limit values have to be evaluated in practical use and if needed corrected. Initially, deflection basin parameters limit values, developed in this research, can be used for pavement condition assessment in network level. It is possible to determine road sections with insufficient pavement structural capacity using FWD measurements and proposed method for determine max limit values of deflection basin parameters.

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