THE BALTIC JOURNAL
OF ROAD AND BRIDGE ENGINEERING

2009
4(4): 196–202

USE OF FWD DEFLECTION BASIN PARAMETERS (SCI, BDI, BCI) FOR PAVEMENT CONDITION ASSESSMENT

Ott Talvik[1], Andrus Aavik[2]

Dept of Transportation, Tallinn University of Technology,
Ehitajate tee 5, 19086 Tallinn, Estonia
E-mails: [1]ott.talvik@ttu.ee; [2]andrus.aavik@ttu.ee

Abstract. Current research studies relationship between Falling Weight Deflectometer (FWD) deflection basin parameters and road pavement structural condition indicators, such as fatigue cracking and permanent deformations induced by low bearing capacity of pavement, and pavement equivalent *E* modulus (E_{eq}). The primary aim of the research was to develop limit values for deflection basin parameters: surface curvature index (*SCI*), base damage index (*BDI*) and base curvature index (*BCI*). Analyses of data derived from the Estonian Road Databank disproved the hypothesis of relationship between deflection basin parameters and pavement defects or rutting. Deflection basin parameters and back-calculated E_{eq} were found to be in good correlation. Strong relationships were found between upper layers indicators (*SCI* and *BDI*) and E_{eq}. Relationship between subgrade indicator *BCI* and E_{eq}, found in the research, was not very strong. Based on the aforementioned relationships, and the required min equivalent modulus of particular pavement, the equations to determine the limit values of deflection basin parameters for different types of pavements were developed. 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 practice and, if needed, corrected.

Keywords: FWD, deflection basin, surface curvature index (*SCI*), base damage index (*BDI*), base curvature index (*BCI*), limit value.

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

ISSN 1822-427X print / ISSN 1822-4288 online DOI: 10.3846/1822-427X.2009.4.196-202
http://www.bjrbe.vgtu.lt

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;

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	$SCI = d_0 - d_{300}$, $SCI = d_0 - d_r$ (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	$BCI = d_{600} - d_{900}$ (used in USA) $BCI = d_{900} - d_{1200}$ (used in Finland) $BCI = d_{1200} - d_{1500}$ (used in Estonia)	μm, mm	Characterizing condition of subbase or subgrade
Area	$AREA = \frac{6(D_0 + 2D_1 + 2D_2 + D_3)}{D_0}$ $AREA = \frac{150(d_0 + 2d_{300} + 2d_{600} + d_{900})}{d_0}$	mm	Characterizing shape of the deflection basin close to the load by the normalized area on the top of the deflection basin
Area 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	$F_1 = \frac{(d_0 - d_{600})}{d_{300}}$, $F_2 = \frac{(d_{300} - d_{900})}{d_{600}}$	–	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.

– Base Curvature Index (*BCI*) – difference of deflections measured with load cells in the distance 1200 mm (d_{1200}) and 1500 mm (d_{1500}): (d_{1200} – d_{1500}), which is characterizing condition of the subgrade.

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 paved 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 ans 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_{req}) and related forecasted traffic loadings and those were used for dividing data into analyze groups (Table 2).

Table 2. Analyze groups, traffic loadings and required $E_{eq\min}$ (Metsvahi *et al.* 2005)

Analyze group	Traffic loading, standard axle load (100 kN), vpd	Required $E_{eq\min}$, 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 ($p_{measured}$) 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 ($p_{target}/p_{measured}$). 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 dependant 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 … +35 °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 °C. For correction of FWD measured deflection values to the standard temperature (+10 °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	$K_t = 0.000203T^2 - 0.014841T + 1.127603$
Cold bituminous mix	$K_t = 0.000205T^2 - 0.015198T + 1.135192$

FWD measured deflections are transformed to the standard load level (50 kN) and standard temperature (+10 °C) using following Eq:

$$d_{r50kNT} = d_r \times \left(\frac{p_{target}}{p_{measured}} \right) \times K_t, \quad (1)$$

where: d_{r50kNT} – deformation at the load 50 kN and temperature +10 °C at the distance *r* (mm) from the center of the loading plate, μm; d_r – FWD measured deflection at contact pressure $p_{measured}$ (kPa) at the distance *r* (mm) from the center of the loading plate, μm; p_{target} – contact pressure, corresponding to the 50 kN load (p_{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 deflections 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.

$$PDS = \frac{(0.5 \times LCRACK + 1.0 \times ALLIG) \times 100}{RWIDTH \times 100}, \%, \quad (2)$$

where: *LCRACK* – length of longitudinal cracks, m; *ALLIG* – extent of alligator cracking, 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 absences of relationship. For example the *SCI* vs *PDS* of dense asphalt concrete surface of analyze Group 2 is presented in the Figure 1.

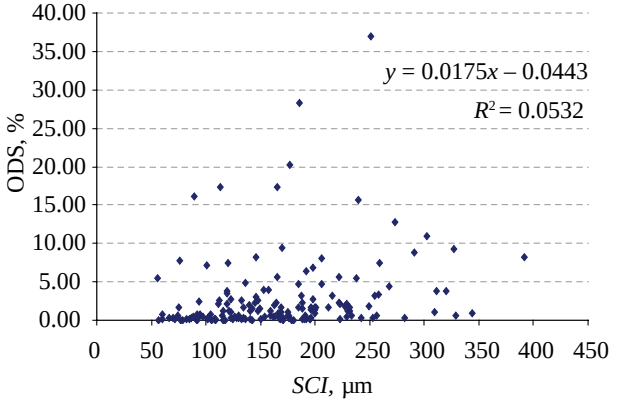

Fig. 1. *SCI* vs *PDS* of asphalt concrete surface of analyze group 2

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.

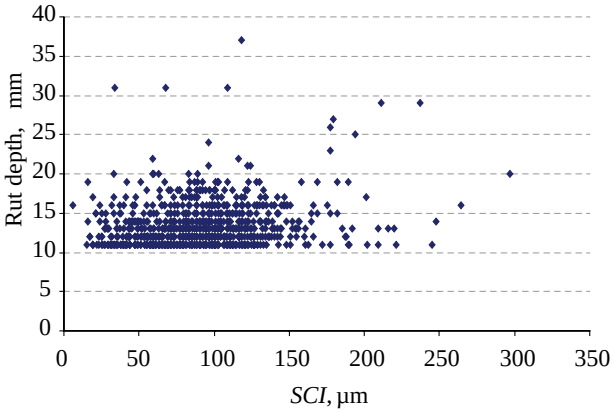

Fig. 2. Values of *SCI* when rut depth > 10 mm, analyze group 7

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:

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

where: E_{eq} – pavement equivalent *E* modulus at the center of the loading plate, MPa; ν – Poisson's ratio (in *Procedure 2001-52* ν = 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, µm.

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):

$$E_{eq2001-52} = C \times E_{eq}^{e} \times T^{t} \times R^{r} \times M_i \times H_j, \quad (4)$$

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, …, 10, April–October); H_j – factor taking into account the height of embankment at the

FWD measurement site (j = < 0.5 m; 0.5–1.0 m; > 1.0 m); C, e, t, r – empirical constants.

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:

$$E_{eq2001\text{–}52} = C \times E_{eq}^{e} \times T^{t} \times M_i, \quad (5)$$

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.

Table 4. Values of factor M_i, taking into account the month when measurement is performed (Aavik 2003)

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 $E_{eq2001\text{-}52}$ are quite strong. Relations SCI–$E_{eq2001\text{-}52}$ and BDI–$E_{eq2001\text{-}52}$ 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

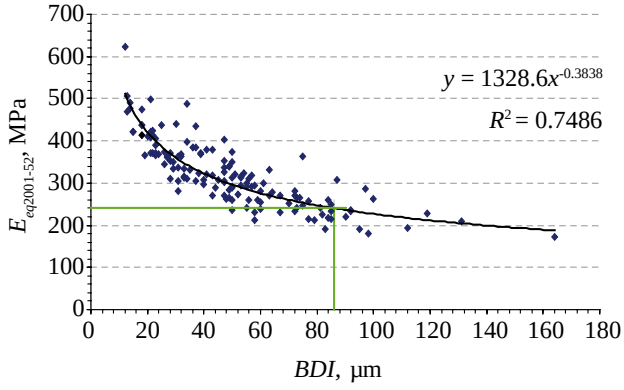

Fig. 3. $E_{eq2001\text{-}52}$ – *BDI, AC*, analyze group 6, without surface defects

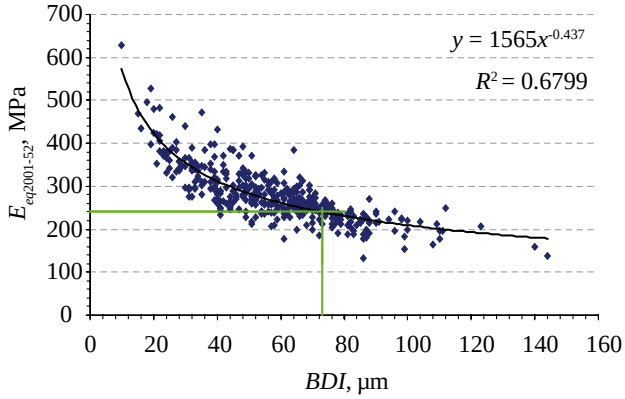

Fig. 4. $E_{eq2001\text{-}52}$ – *BDI, AC*, analyze group 6, with surface defects

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 ($E_{req2001\text{-}52}$).

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–$E_{eq2001\text{-}52}$ 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–$E_{eq2001\text{-}52}$ relationship.

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:

$$y = a_0 x^{a_1}, \quad (6)$$

where: x – min E_{req}, MPa; y – deflection basin parameter (*SCI, BDI, BCI*); a_0, a_1 – constants according to Table 5.

Table 5. Values of constants a_0 and a_1 in Eq (6) for calculation of the max allowable deflection basin parameter (*SCI, BDI, BCI*) values (Talvik 2007)

Type of pavement (based on Estonian Road Data Bank)	Deflection basin parameter y	Value of constants		R^2
		a_0	a_1	
AC pavement on top of existing pavement	*SCI*	1 795 660	–1.70	0.83
	BDI	1 265 966	–1.74	0.78
	BCI	51 220	–1.36	0.68
AC pavement on top of leveling milling	*SCI*	655 780 050	–2.76	0.87
	BDI	15 319 713 999	–3.47	0.93
	BCI	11 182	–1.13	0.09
AC pavement on top of leveling layer	*SCI*	169 150 407	–2.54	0.92
	BDI	104 111	–1.27	0.38
	BCI	–	–	–
AC pavement on bitumen-stabilized base	*SCI*	88 410	–1.113	0.54
	BDI	62 337	–1.161	0.51
	BCI	985 977	–1.909	0.35
AC pavement on complex-stabilized base	*SCI*	1 225 980	–1.63	0.53
	BDI	137 949	–1.307	0.85
	BCI	497 43	–0.492	0.14
AC on crushed stone base	*SCI*	498 577	–1.45	0.87
	BDI	10 645	–0.84	0.21
	BCI	51 984	–1.31	0.61
Cold bituminous mix	*SCI*	834 463	–1.55	0.97
	BDI	2 055 457	–1.84	0.94
	BCI	983 446	–1.99	0.84
Oil shale ash stabilized pavement	*SCI*	2 491	–0.44	0.14
	BDI	1 325 498	–1.80	0.75
	BCI	12 473 680	–2.43	0.44
Surface-dressed gravel pavement	*SCI*	13 705	–0.72	0.66
	BDI	258 341 445	–2.83	0.96
	BCI	255 760	–1.87	0.95

For example relationship between SCI–E_{req} of dense asphalt concrete on top of existing pavement (Fig. 5) can be described as follows:

$$SCI = 1795660E_{req}^{-1.70},\ R^2 = 0.83. \quad (7)$$

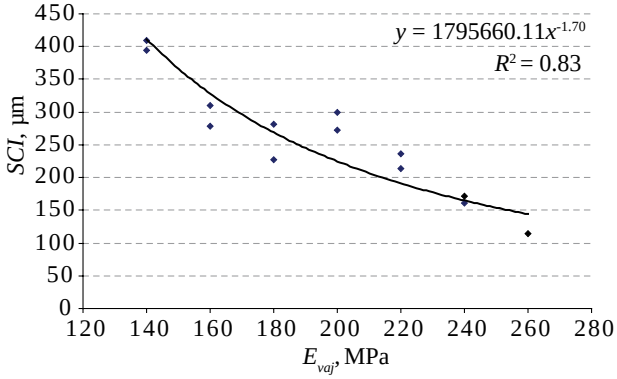

Fig. 5. SCI–E_{req}, AC pavement on top of existing pavement

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

The Eq for calculation min E_{req} according *Procedure 2001-52* is following:

$$E_{req} = (a\log(Q) + b)K_{tt}, \quad (8)$$

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

Table 6. DBPs limit values for E_{req}, AC pavement on top of existing pavement

$E_{req,}$ MPa	*SCI*	*BDI*	*BCI*
140	403	233	62
160	322	185	51
180	263	151	44
200	220	125	38
220	187	106	33
240	161	91	30
260	141	80	27

Table 7. Values of constants a and b according *Procedure 2001-52*

	Load group	a	b
Lorry	A	67.60	61.3
	B	73.37	-7.7
Bus	A	77.00	62.0
	B	84.70	0

Table 8. Pavement strength factors according *Procedure 2001-52*

Road class	Pavement	K_{tt}
Motorway, I, II	permanent pavement	1.00
III	permanent pavement	0.94
III, IV, V	light pavement	0.90
IV, V	transient pavement	0.63
V, out of class roads	primitive pavement	0.63

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} ($E_{eq2001\text{-}52}$), as relationship between subgrade indicator *BCI* and $E_{eq2001\text{-}52}$, 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 $E_{eq2001\text{-}52}$.

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.

References

Aavik, A.; Paabo, P.; Kaal, T. 2006. Assessment of Pavement Structural Strength by the Falling Weight Deflectometer, *The Baltic Journal of Road and Bridge Engineering* 1(4): 193–199.

Aavik, A. 2003. *Methodical Basis for The Evaluation of Pavement Structural Strength in Estonian Pavement Management System (EPMS).* PhD thesis. Tallinn: Tallinn University of Technology. 152 p. ISSN 1406-4766.

Arraigada, M.; Parti, M. N.; Angelone, S. M.; Martinez, F. 2009. Evaluation of Accelerometers to Determine Pavement Deflections under Traffic Loads, *Materials and Structures* 42(6): 779–790. DOI: 10.1617/s11527-008-9424-4

Bayrak, M. B.; Ceylan, H. 2008. Neural Network-Based Approach for Analysis of Rigid Pavement Systems Using Deflection Data, *Transportation Research Record* 2068: 61–70. DOI: 10.3141/2068-07

Chea, S.; Martinez, J. 2008. Using Surface Deflection for Detection of Interface Damage Between Pavement Layers, *Road Materials and Pavement Design* 9(SI): 359–372. DOI: 10.3166/rmpd.9hs.359-372

Dawson, T. A.; Baladi, G. Y.; Sessions, C. P.; Haider, S. W. 2009. Backcalculated and Laboratory-Measured Resilient Modulus Values, *Transportation Research Record* 2094: 71–78. DOI: 10.3141/2094-08

Donovan, P.; Tutumluer, E. 2009. Failing Weight Deflectometer Testing to Determine Relative Damage in Asphalt Pavement Unbound Aggregate Layers, *Transportation Research Record* 2104: 12–23. DOI: 10.3141/2104-02

Grenier, S.; Konrad, J. M. 2009. Dynamic Interpretation of Failing Weight Deflectometer Tests on Flexible Pavements Using the Spectral Element Method: Backcalculation, *Canadian Journal of Civil Engineering* 36(6): 957–968. DOI: 10.1139/L09-010

Grenier, S.; Konrad, J. M.; LeBaeuf, D. 2009. Dynamic Simulation of Falling Weight Deflectometer Tests on Flexible Pavements Using the Spectral Element Method: Forward Calculations, *Canadian Journal of Civil Engineering* 36(6): 944–956. DOI: 10.1139/L08-118

Kim, Y. R.; Lee, Y. C.; Ranjithan, S. R. 2000. Flexible Pavement Condition Evaluation Using Deflection Basin Parameters and Dynamic Finite Element Analysis Implemented by Artificial Neural Networks, in *ASTM STP 1375 Non-Destructive Testing of Pavements and Backcalculation of Moduli,* vol 3. Ed. by Tayabji, S. D.; Lukanen, E. O. American Society for Testing and Materials (ASTM International), West Conshohocken: PA, 514–530. ISBN 0803128584

Losa, M.; Bacci, R.; Leandri, P. 2008. A Statistical Model for Prediction of Critical Strains in Pavements from Deflection Measurements, *Road Materials and Pavement Design* 9(SI): 373–396. DOI: 10.3166/rmpd.9hs.373-396

Metsvahi, T.; Koppel, M.; Pihlak, I. jr. 2005. *Maanteede projekteerimisnormid. ptk. 4 – Katendid* [Standard for Road Design. Chapter 4 – Pavements]. TTÜ, Teedeinstituut [Tallinn University of Technology, Dept of Transportation].

Park, H. M. 2001. *Use of Falling Weight Deflectometer Multi-Load Level Data for Pavement Strength Estimation*. PhD thesis. Raleigh, North Carolina.

Seo, J. W.; Kim, S. I.; Choi, J. S.; Park, D. W. 2009. Evaluation of Layer Properties of Flexible Pavement Using a Pseudo-Static Analysis Procedure of Falling Weight Deflectometer, *Construction and Building Materials* 23(10): 3206–3213. DOI: 10.1016/j.conbuildmat.2009.06.009

Talvik, Ott. 2007. *FWD mõõtmistulemuste alusel arvutatud parameetrite SCI, BDI ja BCI kasutamine teekatendi seisukorra hindamisel* [Use of FWD Deflection Basin Parameters (*SCI, BDI, BCI*) for Pavement Condition Assessment]. Magistritöö [Master thesis]. Tallinn: TTÜ Teedeinstituut [Tallinn University of Technology, Dept of Transportation], 163 p.

Received 22 August 2008; accepted 11 November 2009