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THE CONTROL OF SOIL COMPACTION DEGREE BY MEANS OF LFWD

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Abstract. Compaction is a method of *in situ* soil modification to improve its engineering properties. The paper aim is to present the light falling weight deflectometer applied in a laboratory and *in situ* to the compaction control for non-cohesive soils. It also includes a short description of the method development history and ways of results interpretation. The study resulted in the correlations between test results obtained by means of the light falling weight deflectometer and degree of compaction for 4 groups of non-cohesive soil types.

Keywords: subgrade, soil compaction control, *in situ* test, Light Falling Weight Deflectometer (LFWD).

1. Introduction

Compaction quality of soil layers built-in various earth structures is very important for their durability, compressibility, and bearing capacity. A gradual increase in the number and quality and cubic content of earthworks can be observed, particularly in road-building industry. Requirements referring to the quality of embankments still arise. Therefore, quick and efficient methods for controlling the compaction of soils built-in the embankments are searched for.

Classic methods of compaction control are based on determining the degree of compaction or deformation modulus tested by means of Plate Loading Test. However, the tests are time-consuming and make breaks during the process of a building erection, hence methods and devices for quick testing of another measures of soil compaction are developed, which are subsequently calibrated in comparison to results from traditional and well-recognized methods. The most modern methods that are currently developed are: Briaud Compaction Device (Briaud *et al.* 2006), Soil Stiffness Gauge (or Humboldt Geogauge) (Edil, Sawangsuriya 2005), and Continuous Compaction Control (Brandl *et al.* 2005). Different Light Falling Weight Deflectometers (LFWD) were also brought in the engineering practice (Brandl *et al.* 2003; Rahman *et al.* 2008; Tompai 2009).

Present paper deals with the method for the determination of embankment compaction using LFWD and contains a short description of the method development history, as well as correlations between results achieved from non-cohesive soils studies using LFWD and degree of compaction.

2. Light Falling Weight Deflectometer

Developing the method for soil tests using LFWD has begun in the 50's of the 20th century. LFWD constructed in a similar way as modern one was tested in Japan in 1957. Stages of development and improvement of this method, as well as constructional modifications of LFWD were described in details in earlier publications (Bohn 1968; Brandl *et al.* 2003). Theoretical principles of the method were worked out by Weingart (1978), although they were also developed in subsequent research (Brandl *et al.* 2003).

In Germany, works upon the construction of LFWD have been carried out since the mid of 20th century, and mainly focused on the vibration generator improvement that determines the power impulse and subsoil load duration. The method was implemented into the GDR standard in 1980, into Czechoslovak standard in 1981, and then into German road-engineering directives in 1991.

Different types of LFWD have become common in many countries recently: in Germany as Leichtes Fallgewichtsgerät, while in other EU countries as light drop-weight tester (LDWT), German dynamic plate bearing test (GDPBT), or light falling weight device (LFWD); in USA they are called as follows: light drop weight (LDW), light falling weight deflectometer (LFWD), portable FWD (PFWD), dynamic load plate (DLP).

The principle of LFWD work is to simulate the conditions of subgrade soil loading due to a moving vehicle

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wheels. Tests using LFWD relies on invoking a short-term force impulse by hitting the weight that falls down the guide rod onto a damper from a given height h. The elastic modulus of the subgrade soil was calculated from the surface deflection using the following Boussinesq equation:

$$E_0 = \frac{k(1-\nu^2)\sigma_0 r}{s_0}, \quad (1)$$

where $E_0 = E_{LFWD}$, MPa; k – coefficient ($k = \pi/2$ for flexible plates and 2 for rigid plates, respectively); ν – Poisson's ratio, depended on tested subgrade type $\nu = 0.25 - 0.50$ (Fujyu *et al.* 2004); σ_0 – contact pressure, kPa; r – plate radius, mm; s_0 – deflection at centre, mm.

There are some types of LFWDs that differ from one another with constructional details and technical parameters. Producers make their modifications consisting mainly in that the LFWDs are equipped with kits for regulation and measuring the impact force values, besides sensor for measuring the plate deformation due to the fallen weight. Some types of LFWDs may have variable diameter of the plate. Further studies upon vibration amortizing device that can be built of steel disc springs, plastic elements, or rubber disc springs are still carried out (Brandl *et al.* 2003).

Particular countries conduct their own studies upon applying the newly implemented devices and result interpretation under local soil conditions. Works on LFWDs are carried out in following directions: comparative studies upon parameters of various types of LFWDs, as well as comparative studies of soil tests results using LFWDs and other well-recognized for *in situ* tests. Those studies are performed to find correlations between parameters achieved by means of various types of LFWDs and geotechnical parameters needed in engineering practice.

The results achieved using particular LFWDs differ (Brandl *et al.* 2003; Vennapusa, White 2009). Differences between data are due to differences in generated impact forces and construction of vibration amortizing device which leads to non-uniform vibration frequency and non-uniform impulse duration. Thus, there is a need to standardize instruments or to calibrate particular LFWDs under local soil conditions and to work out the directives for result interpretation for every type of LFWD separately.

3. Deformation modulus of a soil

Parameters of LFWD ZFG are presented in Table 1.

In tests by means of LFWD ZFG, the impulse is transferred through the tension plate onto a surface of tested soil layer and invokes the deformation beneath the plate. Electronic deflection-data device registers values of deformation modulus of the layer E_{vd} (MPa), calculated according to Eq (1). Assuming as constants $k = 2$ and $\nu = 0.5$ the following formula was achieved:

$$E_{vd} = 1.5\frac{r\sigma_D}{s} = \frac{22.5}{s}, \quad (2)$$

where r = 150 mm – radius of pressing loading plate; σ_D = 0.1 MPa – amplitude of dynamic tension beneath pressing loading plate; s – mean surface deflection of pressing loading plate calculated from 3 testing impacts results made after 3 preliminary tests, mm. In addition, $\frac{s}{V}$ ratio (in ms) is recorded, where V – mean deflection velocity calculated for 3 testing impacts.

In German recommendations related to the control of earthworks the use of LFWD is permitted in road construction to test the compaction of embankments and backfills (alternatively or additionally to static plate loading) according to Table 2. At the same time the calibration works still continue.

Weingart (1993) reported that $\frac{s}{V}$ ratio brought additional information on a soil compaction quality. In the case of non-cohesive soils with flat grain-size distribution curve, high value of $E_{vd} > 50$ MPa and low value of $\frac{s}{V} < 3.5$ ms are achieved, while non-cohesive soils with steep grain-size distribution curve and poorly compact are characterized by low $E_{vd} < 25$ MPa and high $\frac{s}{V} > 3.5$ ms values.

4. Experimental procedure

Tests were carried out for non-cohesive soils in north-eastern Poland. The deformation modulus was determined applying LFWD ZFG 02. The degree of compaction was calculated according to the formula:

$$I_s = \frac{\rho_d}{\rho_{d\max}}, \quad (3)$$

where $\rho_{d\max}$ – the max dry density using standard Proctor's method, g/cm^3; ρ_d – the dry density in embankment or subsoil, g/cm^3 according the Eq (4):

$$\rho_d = \frac{100\rho}{100+w}, \quad (4)$$

where ρ – the soil bulk density, g/cm^3; w – the soil water content, %.

Table 1. Parameters of LFWD ZFG

Model	Plate diameter, mm	Weight, kg		Height of the weight fall, mm	Force, kN	Pressure under plate, MPa	Impulse duration, ms
		falling weight	total				
ZFG 01 up to ZFG 05	300	10	30	approx 700	7.07	0.1	18 ± 2

Table 2. List of approximate limit values of E_{vd} according to German specifications

Specification		Embankments and grounds	E_{vd}, MPa	I_s*	E_{v2}**, MPa
ZTVA-StB 97 1997	backfill of cross-cuts and trenches in road construction		60	–	120
			50	–	100
			40	–	80
			25	–	45
ZTVE-StB 94 1994	ground works in road engineering	gravel (GW, GI)***	≥ 55	≥ 1.00	≥ 100
			≥ 45	≥ 0.98	≥ 80
			≥ 40	≥ 0.97	≥ 70
		gravel and sand (GE, SE, SW, SI)***	≥ 40	≥ 1.00	≥ 80
			≥ 35	≥ 0.98	≥ 70
			≥ 32	≥ 0.97	≥ 60
		mixed and fine-granulated material (*gemischt- und feinkornige Boden*)****	≥ 25	≥ 1.00	≥ 45
			≥ 15	≥ 0.97	≥ 30
			≥ 10	≥ 0.95	≥ 20
Directives of Zorn – producer of deflectometer *ZFG* (Brandl *et al.* 2003)	embankments	in general	> 25	–	> 45
		mixed, fine-granulated, stone material (*feinkornig, gemischt-kornig, Felsschuttung*)***	> 25	> 1.03	> 45
	embankments made of granulated material	gravel (GW – *weitgestuft*)***	> 55	> 1.00	> 120
		gravel (GI – *intermittierend*)***	> 45	> 1.00	> 100
			> 40	> 0.97	> 80
		gravel (GE – *enggestuft*)***	> 40	> 1.00	> 80
		sand (SE, SW, SI)***	> 30	> 0.97	> 60
			> 25	> 0.95	> 45

Remarks: * in TP BF-StB Teil B 8.3 (2003) the degree of compaction is designated as D_{Pr};
** German designation of reload deformation modulus from a static Plate Loading Test (300 mm plate diameter): E_{v2};
*** German soil designations: GE, GI, GW, GU, SI, SW (according to *DIN 18 196* (1988)).

4.1. *In situ* tests

In situ tests were conducted for embankments or non-destroyed soils in subsoils. The soil was tested most often on the terrain surface level. In part of measurement points, soil was excavated and every 0.3 m layers were tested. Deformation modulus E_{vd} was determined at every layer. The bulk density of soil was determined using sand volumeter at a distance about 0.3 m from deflectometer trace and soil samples were taken for laboratory Proctor's tests, moisture content, and granulation were determined by means of sieve analysis. Following types of soil were tested: silty sand (1), fine sand (2), and sandy-gravel mix (4). The grain-size distribution of studied soils is presented in Fig. 1. Coefficients of graining uniformity C_U were as follows: 2.03–4.75; 1.30–2.30; 3.91–18.3. Max dry densities of soils tested by means of the standard Proctor method ρ_{dmax} amounted to: 1.607–1.735; 1.595–1.743; 1.920–2.193 g/cm^3, at optimum moisture contents w_{opt}: 14.2–16.4; 11.8–17.7; 6.7–11.1%. Moisture content of tested soils ranged from 1.6% to 10.5%.

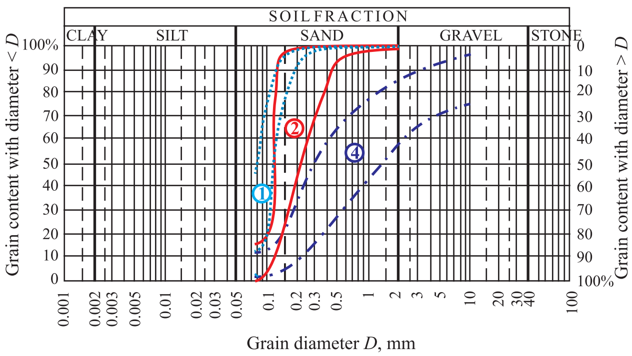

Fig. 1. Grain-size distribution curves from sieve analysis for soils tested *in situ*

4.2. Laboratory tests

Laboratory tests were carried out in Geotechnical laboratory in Bialystok University of Technology, in cylindrical calibration chamber (diameter 0.75 m and height 1.1 m). The soil layer of 1.0 m thickness was put in form of one of 3 soil types: fine sand (2), medium sand (3) and sandy-gravel mix (4) (Grzesiuk 2006). The soils were compacted in layers of about 0.3 m thickness using surface vibrator. The grain-size distribution curves for studied soil types are presented in Fig. 2. Coefficients of graining uniformity C_U were as follows: 1.57; 2.50; 14.82. The max dry densities tested by means of the standard Proctor method ρ_{dmax} amounted to: 1.689; 1.747; 1.959 g/cm^3, at optimum water contents w_{opt} of: 11.7; 9.7; 8.1%. Moisture contents of soils w ranged from 3.5% to 14.8%. The deformation modulus and bulk density of soils were tested for layers of 0.2–0.3 m thickness by removing subsequent layers. The bulk density of soil was determined using sand volumeter, while water content – by drying method.

4.3. Statistical analysis of results

The tests were provided with a set of 210 cases: *in situ* tests – 120 cases, including: for silty sand – 10, for fine sand – 62, for sandy-gravel mix – 48, whereas laboratory tests – 90 cases, including: for fine sand – 35, for medium sand – 25, and for sandy-gravel mix – 30.

Statistical analysis of results aimed at working out the linear regression model for the dependence of deformation modulus on the degree of compaction. The linear regression model is of the form: $y = a + bx \pm \varepsilon$, where ε is standard error of estimate.

Dependence $E_{vd} = f(I_s)$ for all soil types is expressed in Fig. 3 by Eq (3), dependence for *in situ* studies – by Eq (1), and laboratory tests – by Eq (2). The better soil compaction, i.e. higher degree of compaction, the higher value of modulus E_{vd}. It can be seen that standard error of estimate for Eq (1) in Fig. 3 is twice as high as for Eq (2). Eq (1) is also characterized by lower the Pearson correlation coefficient R. This coefficient allows analyzing the relationships between the variables. The more correlation coefficient close to value 1, the stronger linear correlation is between tested variables.

These difference results much greater scatter within *in situ* than laboratory results which are apparent in Figs 4 and 5. Fig. 4 presents dependence $E_{vd} = f(I_s)$ for particular soil type groups studied *in situ*. It can be seen that greater result scattering appeared in *in situ* tests: standard error of estimate is relatively high amounting to 9.38–11.95 MPa at moderately high correlation coefficients 0.91–0.52. It proves non-uniformity of *in situ* studied soil types in reference to their grain distribution and compaction. The greater value of correlation coefficient was stated for relationship (1) for silty sand, the lowest one – for relationship (4) for sandy-gravel mix. As an example, for sandy-gravel mix at $I_s = 1$, Eq (4) in Fig. 4 leads to $E_{vd} = 41.05 \pm 11.95$ MPa, which gives the relative error of result about 29%. Fig. 5 presents dependence $E_{vd} = f(I_s)$ for soils tested in laboratory. Laboratory tests for 3 soil types produced less result scatter: standard error of estimate is 3.91–5.59 MPa at correlation coefficient of 0.72–0.82. For relationship (2) for fine sand the lower correlation coefficient was obtained

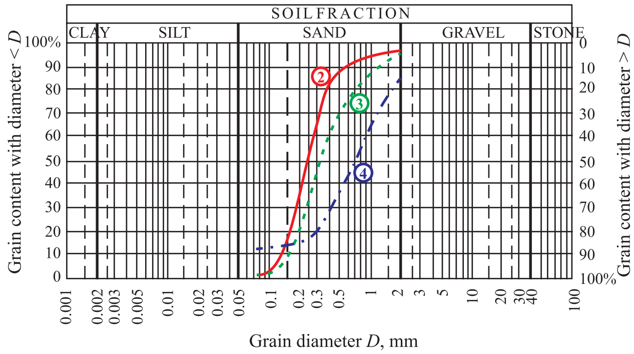

Fig. 2. Grain-size distribution curves from sieve analysis for soils tested in the laboratory

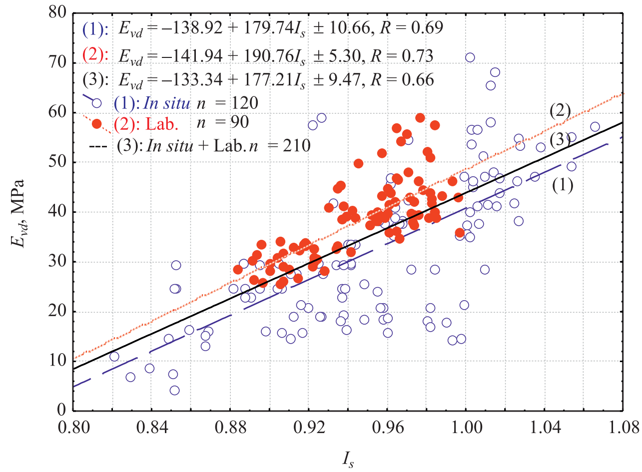

Fig. 3. Dependence $E_{vd} = f(I_s)$ for various tests: (1) – *in situ* tests; (2) – laboratory tests; (3) – data from *in situ* tests and laboratory tests altogether

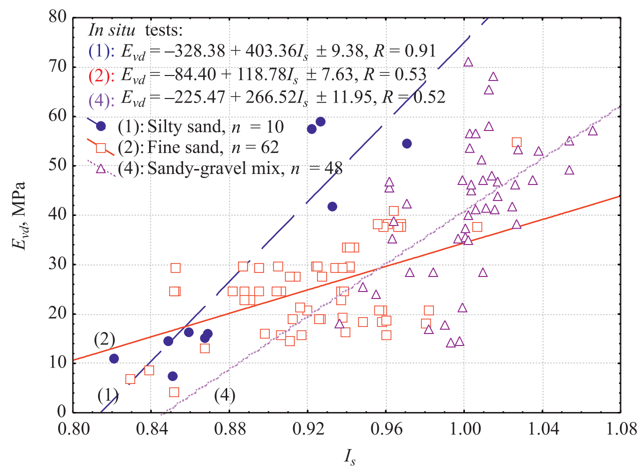

Fig. 4. Dependence $E_{vd} = f(I_s)$ for data from *in situ* tests: (1) – silty sand; (2) – fine sand; (4) – sandy-gravel mix

than for relationships (3) or (4) for medium sand or sandy-gravel mix. As an example, for fine sand at $I_s = 1$, Eq (2) in Fig. 5 leads to $E_{vd} = 49.02 \pm 5.59$ MPa which results in a relative error of about 11%.

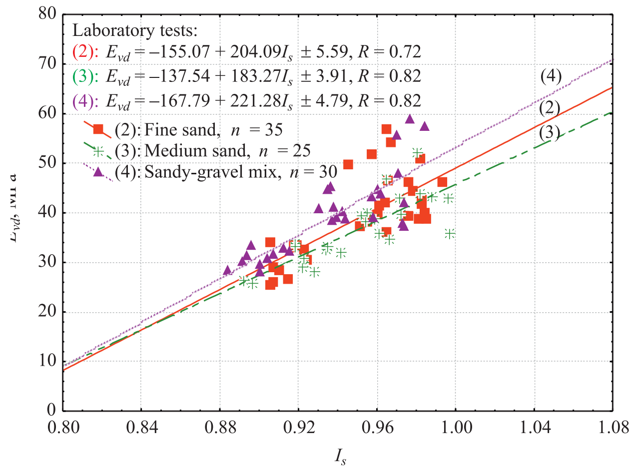

Fig. 5. Dependence $E_{vd} = f(I_s)$ for data from laboratory tests: (2) – fine sand; (3) – medium sand; (4) – sandy-gravel mix

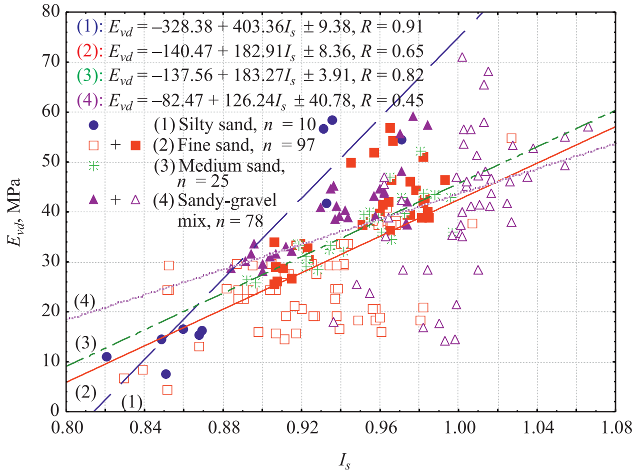

Fig. 6. Dependence $E_{vd} = f(I_s)$ for different soil types: (1) – silty sand; (2) – fine sand; (3) – medium sand; (4) – sandy-gravel mix

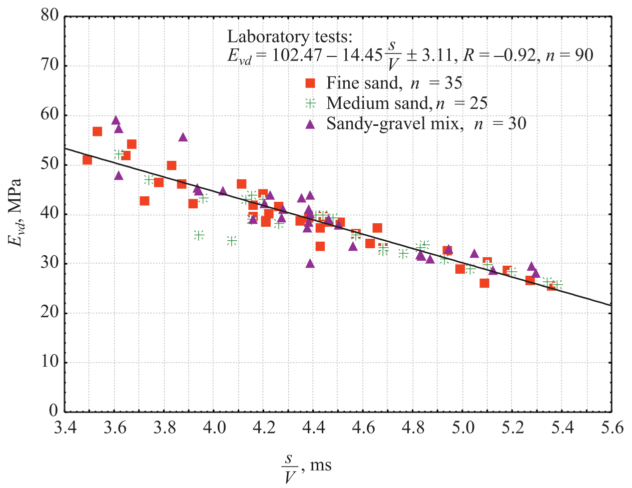

Fig. 7. Dependence $E_{vd} = f\left(\frac{s}{V}\right)$ for laboratory tests results

Achieved results suggest that prior to building-in a particular soil type into an embankment the calibration studies should be performed and the limit values of E_{vd} should be determined depending on required I_s value. Therefore, the precision of soil compaction control could be enhanced. Fig. 6 presents achieved dependencies for studied soil groups for all results. It can be notice that equation graphs (2)–(4) for fine sand or medium sand or sandy-gravel mix are similar to each other. Equation graph (1) for silty sand is different from other tested soils.

Laboratory results helped in defining the dependence $E_{vd} = f\left(\frac{s}{V}\right)$ that is shown in Fig. 7. The apparent negative linear correlation of high value of correlation coefficient between E_{vd} and $\frac{s}{V}$ is seen. Value of modulus E_{vd} proportionally decreases along with $\frac{s}{V}$ ratio increase. It can be stated that $\frac{s}{V}$ ratio may be (apart E_{vd}) addition parameter of soil compaction tested by means of LFWD. For tested soils the value $E_{vd} > 50$ MPa at $\frac{s}{V} < 3.6$ ms was achieved which confirmed observations made by Weingart (1993).

5. Conclusions

Tests using LFWD is relatively new, modern and quick method for compaction control of embankments. Different types of deflectometers arise and works upon this method development still continue.

Greater result scatter was achieved in *in situ* tests than laboratory tests which can prove larger uncertainty of *in situ* measurements resulting from non-uniformity of tested soils in reference to their graining and compaction. As an example, for sandy-gravel mix at $I_s = 1$, $E_D = 41.05 \pm 11.95$ MPa which results in a relative error of results of about 29%. Laboratory tests for 3 soil types produced less result scatter: standard error of estimate is 3.91–5.59 MPa at correlation coefficient of 0.72–0.82. As an example, for fine sand at $I_s = 1$, $E_{vd} = 49.02 \pm 5.59$ MPa, which results in relative error about 11%.

In order to obtain enhanced precision of embankment compaction control, calibration studies should be performed and the limit values of E_{vd} should be determined depending on required I_s value before the soil is built in embankment.

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