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An assessment of how penetration curve adjustment affects the California bearing ratio (CBR)

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Abstract: With the rapidly developing road transport, there is a demand for new roads to be constructed and for the existing ones to be repaired or extended. The base of any road is its foundation, usually made of crushed or uncrushed aggregate. To be used for road foundation purposes, a material needs to meet many requirements, as imposed by relevant standards. One of the basic tests to assess the suitability of an aggregate is to determine its California bearing ratio (CBR).

This paper presents the results of CBR tests for mixed aggregate with the grading of 0–31.5 and 0–63 mm. Detailed assessments were carried out for penetration curves, which in many cases need to be adjusted to meet industry standards. The adjustment of plunger penetration curves in aggregate samples causes CBR to increase compared to the original curves.

Keywords: CBR; pavement structure; subbase; roadbase.

1 Introduction

Road transport is the primary means of transport for both people and cargo, and its importance continues to grow. Compared to rail transport, road transport is developing much faster while serving the core functions of passenger and cargo transport and providing deliveries for the construction, agriculture, commerce and service sectors (Wojewódzka-Król, 2017). In Poland, road transport is constantly developing through the construction of roads, the density of which in 2010 was 87.6 km per 100 km^2, while by 2018 it grew to 97.2 km per 100 km^2 (GUS, 2020).

Hard-surfaced roads, which account for 70% of all public roads in Poland, are designed and constructed specifically for their intended use and location. Materials to be used for road foundation purposes need to meet many requirements, as imposed by relevant standards. Among the physico-mechanical parameters determining the designation of a given material for road construction include water absorption (WA_{24}), frost resistance (F), flakiness index (Fl), sand equivalent (SE), Los Angeles coefficients (LA), resistance to wear micro-Deval (M_{DE}), California bearing ratio (CBR), mass fraction of passing 0.063 mm (*f*) and good particle size distribution of the mixture (Ćwiąkała et al., 2016; Esfahani & Goli, 2018; Hydzik-Wiśniewska, 2020; Hydzik-Wiśniewska & Bednarek, 2020; Pourkhorshidi et al., 2020; Xiao et al., 2012). As the primary element of hard-surfaced road foundation, aggregates are usually sourced from local mines (Kozioł & Kawalec, 2008; Lorek, 2015). The popular belief is that the best material for road foundation is igneous rock aggregate. However, igneous rock aggregates are available only in certain locations, so in situ material or sandstone aggregates are often used instead, in many cases meeting the relevant standard requirements much better (Kozioł et al., 2017; Tarasewicz et al., 2013). In addition, concrete rubble, boiler slag and coal mining waste are increasingly considered as alternative road construction materials (Batog & Hawrysz, 2011; Sas & Głuchowski, 2014; Zawisza & Gruchot, 2017).

This paper presents the results of CBR tests for mixed aggregate with the grading of 0–31.5 mm and 0–63 mm, as found in various regions of southern Poland. Special attention was paid to plunger penetration curves in individual samples. The paper compares and examines the extent to which curve adjustment affects CBR.

2 Pavement structure

A pavement structure is defined as a set of specific layers designed to distribute pressure caused by vehicle wheels across the roadbed and to ensure safe and comfortable driving. The terms ‘pavement structure’ and ‘pavement’ are synonymous here. Pavement rests on the roadbed or on improved subgrade (Judycki et al., 2012).

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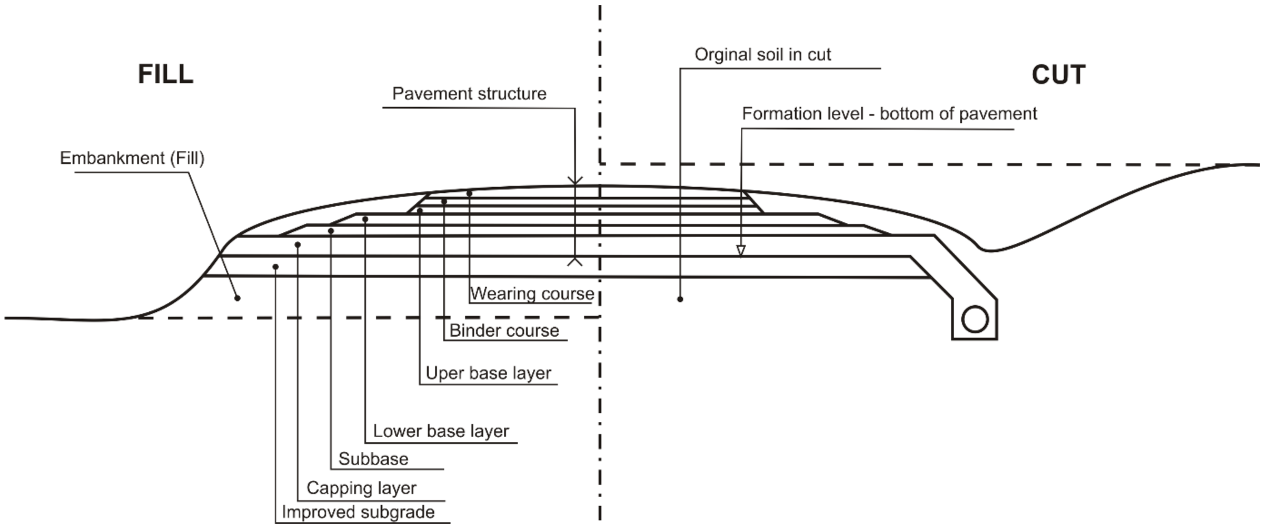

Figure 1: Cross section of pavement structure (Judycki et al., 2012).

Pavement may be constructed as a road embankment, meaning a pavement structure made partially of building land, or as a cutting, meaning pavement situated in a place from which building land was removed (Fig. 1). In either case, the pavement structure is made up of the same layers, and the key difference is the location of the pavement in relation to the ground level. The pavement structure consists of six layers that can be divided into two major groups – top and bottom layers. Top layers include wearing and binder courses and roadbase, which in turn includes upper and lower bases. And bottom layers include subbase and frost-blanket courses. The pavement structure as a whole is embedded in the roadbed, which includes stabilized subgrade and original soil (Judycki et al., 2012).

Wearing and binder courses are made using asphalt mixtures, while roadbase, which may have one or two layers (upper base layer and lower base layer), is constructed on the basis of asphalt concrete, unbound mixture, hydraulically bound material, hydraulically treated soil or cold-recycled mixtures. The subbase, which is to support load transfer by top pavement layers, is constructed using unbound mixtures or hydraulically bound material. Hydraulically treated soil, too, can be used as subbase (Piłat, 2004; Radziszewski et al., 2010). The capping layer, which is to protect the pavement against heaves, can be made from non-expansive soil or lime stabilized soil, or materials used as subbase.

Pavement structures are designed on the basis of the traffic class of the future road. The design traffic class, ranging from KR1 to KR7, is calculated as the total of equivalent standard axle loads of 100 kN per design lane throughout the pavement design life. Depending on the traffic class, specific materials need to be used for individual pavement layers (Chmielewski & Waliszewski, 2016; Mackiewicz & Szydło, 2015).

Unbound mixtures and non-expansive soil are required to have specific properties. One of the criteria used for determining whether a material may be used is its minimum CBR. Table 1 shows minimum CBR values for specific traffic classes and for unbound mixture and non-expansive soil layers.

In other countries, such as Australia, if materials with low CBR (<30%) are used, layer thickness is increased proportionately on the basis of the equivalent standard axle (ESA) load. Figure 2 shows a nomograph used to determine layer thickness for specific traffic loading and CBR values.

3 CBR laboratory tests

Developed in the 1920s at the California Division of Highways by O.J. Porter, the CBR test is currently the most popular method to evaluate base and subbase material quality. Plunger penetration tests on pavement building materials were designed to evaluate the strength of loose bulk and cohesive materials in laboratory and in situ conditions (Ebels et al., 2004; Franco & Lee, 1987; Yoder & Witczak, 1975). CBR provides valuable information about a material's resistance to stress caused by repeated vehicle wheel loads (Turnbull & Ahlvin, 1957). In many countries, CBR is now one of the key tests used for the purposes of pavement design.

Table 1: Minimum CBR values for specific materials by layer and traffic class (Judycki et al., 2012).

Stuff	Layers							
	Roadbase			Subbase		Capping layer		Improved subgrade
	KR1- KR2	KR3-KR4	KR5-KR7	KR1-KR2	KR3-KR7	KR1-KR2	KR3-KR7	KR1-KR7
Unbound mixtures	60	80		The layer does not exist for these traffic categories	60	25	35	20
Non-expansive soil	Does not apply			Does not apply		25	35	20

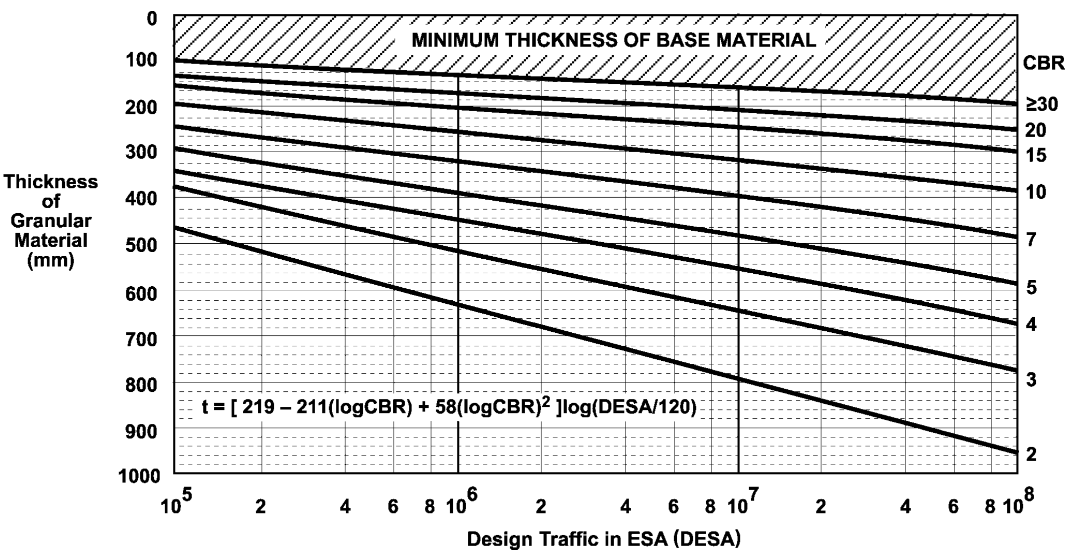

Figure 2: Nomograph for determining the thickness of base and subbase layers (Çelik et al., 2017)

CBR tests are usually carried out in laboratory conditions or, if necessary, in situ. However, the latter is time-consuming and their results are not very useful in practice. Therefore, CBR tests are much more often conducted in laboratories, using special pre-processed samples. The CBR method is the most appropriate and gives the most reliable results for fine-grain soil. But for coarse-grain soil, the reproducibility of its results is poor (Shoop et al., 2008).

Over the hundred years, the CBR method has been in use, there have been many studies to explore both soils and aggregates in order to find correlations between specific properties or parameters and CBR results. For soils, the literature reports findings which show strong correlations between CBR and the plastic limit of soil, maximum bulk density or fine fractions (Bąk & Chmielewski, 2019; Katte et al., 2019; Nguyen & Mohajerani, 2015; Rehman et al., 2015; Talukdar, 2014). But for aggregates, it is difficult to find a parameter that could be considered to significantly affect CBR results (Hydzik-Wiśniewska et al., 2018). And a numerical analysis of CBR tests on the basis of particle flow technique has shown a linear increase in CBR in relation to shear modulus, with no significant impact of Poisson's ratio on CBR (Jiang et al., 2015). Modelling of CBR tests using the finite element method (FEM) has produced results that were so satisfactory that the longitudinal modulus of elasticity (Young's modulus) was declared to correlate with CBR (Hajiannia et al., 2006).

If their *CBR* values are low, aggregate and soil can be used with such admixtures as lime, fly ash and slag. These admixtures act as cementing materials to reinforce the base material. Depending on specific needs and addition types, different amounts of admixtures are possible (Al-Joulani, 2012; Bednarek & Mazurek, 2011; Ratna Prasad & Darga Kumar, 2015).

CBR (1) is expressed as the percentage ratio of the unit load q necessary to drive a plunger in the shape of an elongated cylinder with 20 cm^2 in diameter into a pre-processed sample to a depth of 2.5 and 5.0 mm, at a steady speed of 1.27 mm/min, to the baseline unit load q_p, which is a constant and corresponds to the pressure necessary to drive the same plunter at the same speed to the same depth into normally compacted crushed stone acting as the model material.

$$CBR = \frac{q}{q_p} * 100\% \quad (1)$$

where: q – pressure necessary to drive a plunger with a diameter of 20 cm^2 into a pre-processed sample of the examined aggregate to a depth of 2.5 and 5.0 mm; q_p – baseline pressure, which for (a plunger driven to) a depth of 2.5 mm is 7 MN/m^2, and for a depth of 5.0 mm is 10 MN/m^2.

Since CBR tests are conducted using material samples with optimum moisture content, the content must be determined prior to the tests in line with relevant standards (*PN-EN 13286-2: 2010*). The testing equipment and methodology are selected to match the desired compaction effort. In order to determine optimum moisture content for the mixtures discussed in this paper, we used a 2.2 dm^3 cylinder and a 2.5 kg tamper with a diameter of 50 mm. The material was compacted at a rate of 56 blows per each of its three layers, which corresponds to compaction effort equal to approximately 0.584 MJ/m^3.

CBR tests require about 6 kg of a material taken from the mixture and sifted through a sieve with 20 mm square mesh. Grains larger than 20 mm are replaced with corresponding material with a size between 6.3 and 20 mm (*PN-S 06102: 1997*). Plunger penetration tests are conducted on samples immediately after their compaction and on samples which have been saturated with water for four days. Figure 3 shows a diagram representing a plunger driven into a sample. During penetration, one sensor measures how deep the rod is, and another sensor measures ring deformation, which is then used to calculate the strength. The speed of penetration is 1.27 mm/min, and gauge ring deformation is recorded at depths of 2.5 and 5.0 mm. This data is then used to compute pressure, which serves to determine CBR as a percentage of that pressure, calculated in relation to the baseline pressure.

Test results are presented as a penetration curve, which is a function of depth and pressure or strength. The applicable PN-S-06102 (1997) standard does not take into account the shape of the penetration curve in CBR computation, while the PN-EN 13286-47 (2012) standard requires that CBR be calculated on the basis of plunger depth curve, which needs to be adjusted in some cases. If the initial portion of the curve is concave upwards, no adjustment is necessary and test results are unchanged. But if the initial portion of the curve is concave downwards, then it needs to be adjusted by plotting a tangent at a point in the curve when it is the steepest, i.e., where it bends

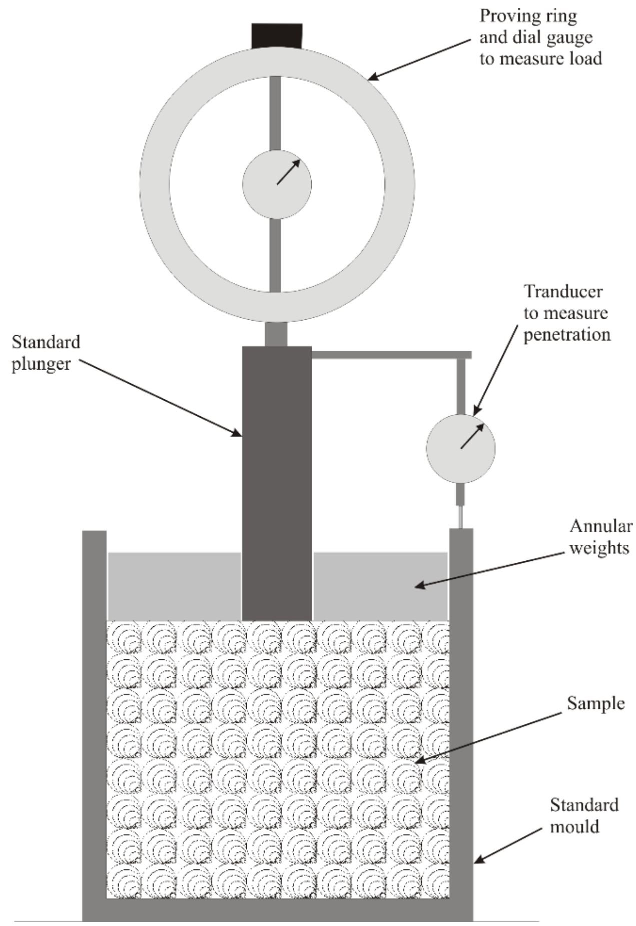

Figure 3: CBR test.

(Fig. 4). Now, the crossing of the tangent and the depth axis is the new starting point of the curve. CBR may not be higher than the penetration depth after an adjustment by more than 7.5 mm. If the adjustment requires a greater depth, the adjustment should be made by no more than 7.5 mm. A curve can be concave if the surface of the sample is uneven and initially the pressure is distributed unevenly across the plunger (*PN-EN 13286-47: 2012*). The American standard defines penetration curve as a stress–strain curve, and it also recommends that the curve be adjusted if there is no proportional resistance to the plunger during the initial stage of penetration (AASHTO T-193, 2007).

Recommendations for penetration curve adjustments appeared in both Polish and foreign standards over a decade ago, but they are not always followed, as can be seen in the diagrams included in some scientific publications (Chebet et al., 2016; Tan et al., 2016). We have found no mentions or analyses in the scientific literature on how such adjustments affect CBR results. There are only some comments on the shape of the obtained penetration curves (Marsh, 1983).

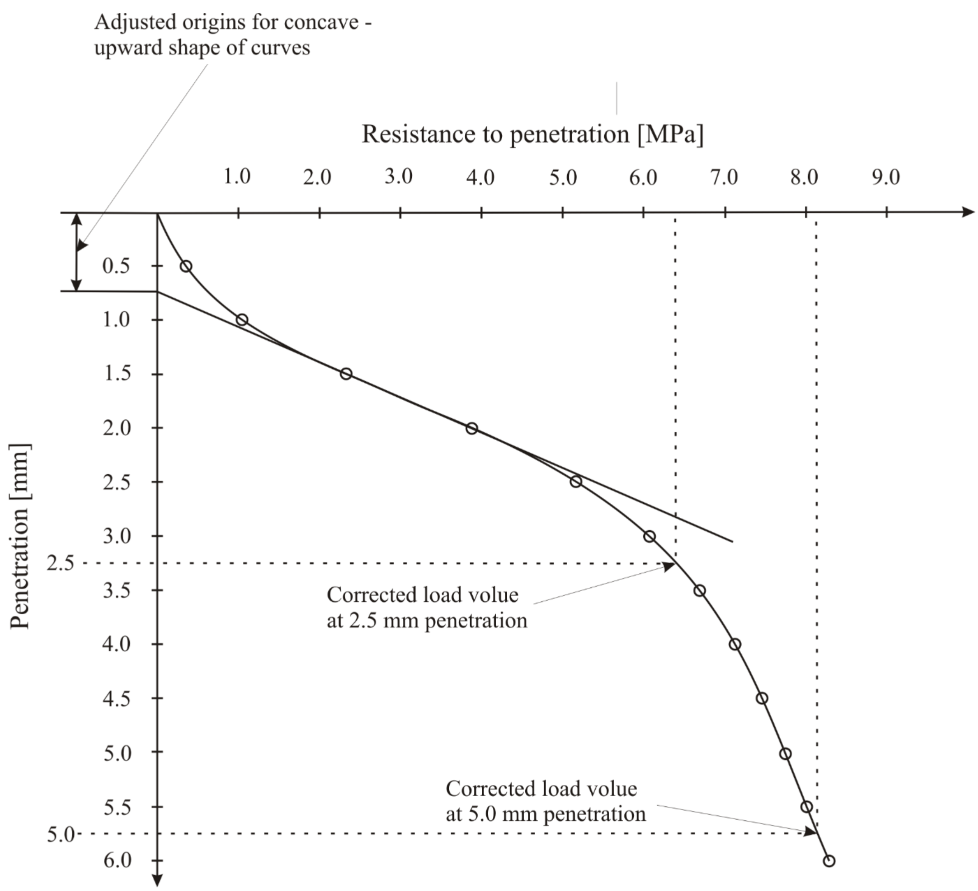

Figure 4: Adjusted penetration curve.

4 Test results

In order to assess the impact of penetration curve adjustment on CBR results, we analysed test results for several types of mixtures with the grading of 0–31.5 and 0–63 mm. Each mixture was tested twice, once immediately after being compacted, and once after the cylinder with the compacted material had been immersed in water for four days. Samples for the study had been obtained from sandstone, dolomite, basalt, granite and magnesite aggregates mined in southern Poland (Fig. 5).

In total, we have examined 48 samples. Table 2 shows the properties of the tested mixtures. Additionally, based on the grain composition (Figs. 6 and 7) of the mixtures, it was calculated uniformity coefficient C_u (2) and coefficient of gradation C_c (3).

$$C_u = {d_{60}}/{d_{10}} \tag{2}$$

$$C_c = {d_{30}^2}/{(d_{10} * d_{60})} \tag{3}$$

where: d_{10} – diameter corresponding to 10% finer in the particle size distribution curve [mm]; d_{30} – diameter corresponding to 30% finer in the particle size distribution curve [mm]; d_{60} – diameter corresponding to 60% finer in the particle size distribution curve [mm].

Figures 8–11 show the results for CBR computations for mixtures with the grading of 0–31.5 mm. The figures include results without penetration curve adjustment and with curve adjustments, whenever necessary. As you can see, penetration curve adjustment causes higher CBR results compared to CBR calculated without any adjustment. This increase ranges from several up to several dozen per cent. The largest increase, by more than 39%, was observed for a basalt mixture and was related to a sample that had been saturated with water for four days (Fig. 10). In a few cases, no adjustment was necessary. The greatest changes were observed after the adjustment of penetration curves for igneous aggregates, such as basalt and granite. Analysed by depth, the results show that penetration curve adjustment significantly affected

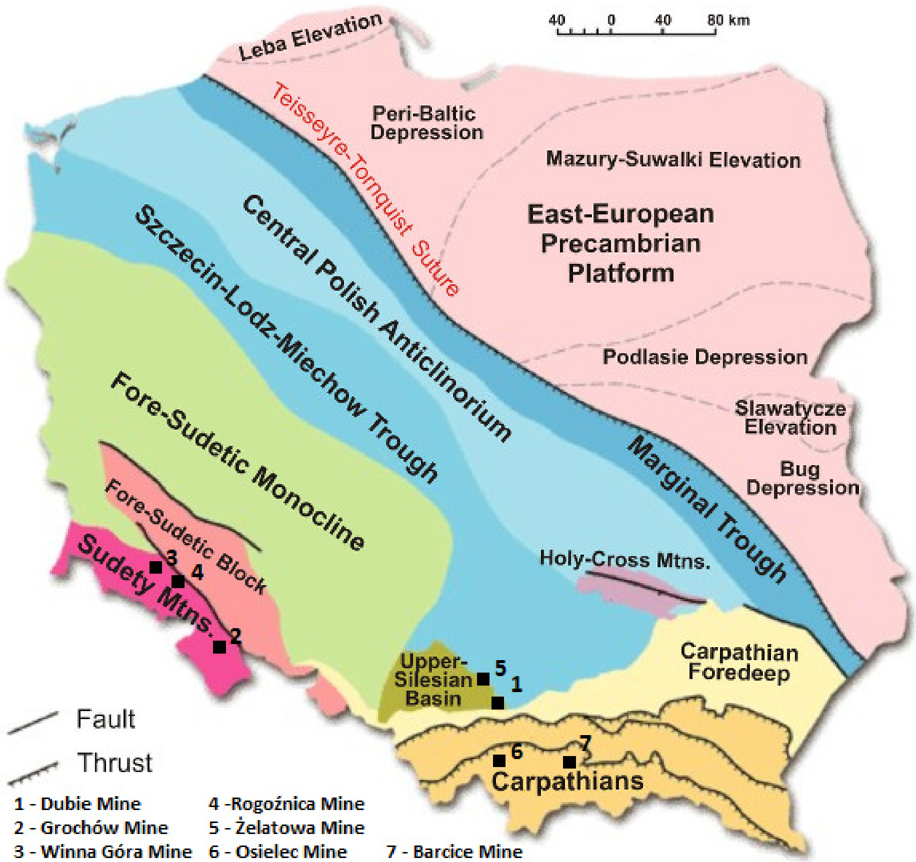

Figure 5: Map of the places where samples were taken for testing (http://redstone-exploration.com/country-profiles/poland/, 2021).

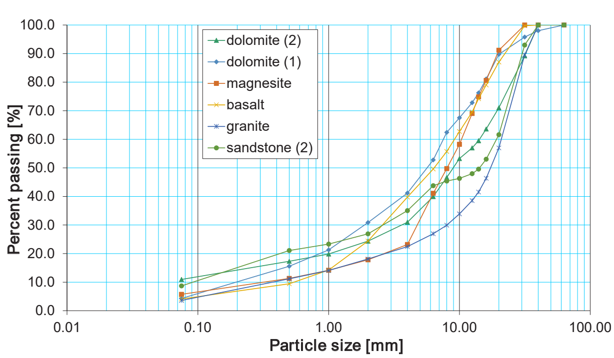

Figure 6: The curves of grain composition of the mixtures of 0–31.5 mm.

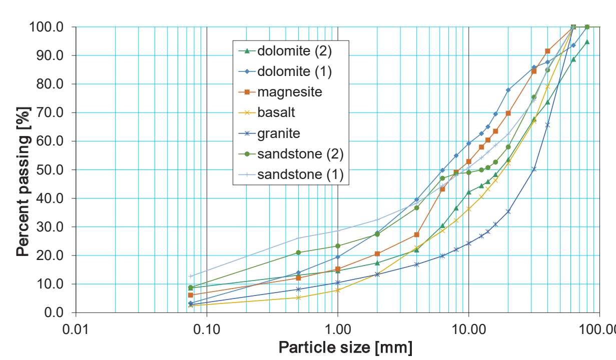

Figure 7: The curves of grain composition of the mixtures of 0–63 mm.

CBR for the depth of 2.5 mm. The average increase in CBR for the depth of 2.5 mm is 10.42%, while that for the depth of 5.0 mm is much lower at 4.64%. There are big differences in CBR values between samples 1 and 2 for the same material. It may be related to the relatively low grain size variation of the mixture. For example, for a mixture of 0–31.5 mm from dolomite (1) for which C_u = 37.5 and C_c = 2.4, the CBR difference between the samples is even 42%. Whereas, for the 0–31.5 mm mixture of dolomite (2), for which C_u = 193.3 and C_c = 12.6, the difference does not reach the value of 5%. Figures 12-15 shows penetration curves and their adjustments for granite aggregate with the grading of 0–31.5 mm.

Table 2: Properties of the tested mixtures.

Mine	Rock type	Mixtures	Mass fraction of passing 0.0063 mm, *f* [%]	Flatness index, *FI* [%]	Shape indicator, *SI* [%]	Sand equivalent, *SE* [%]	Optimum moisture content [%]	Maximum dry density [g/cm³]	Uniformity coefficient, C_u [-]	Coefficient of gradation, C_c [-]
Dubie	Dolomite (1)	0–31.5 mm	4.3	-	19.7	48.7	5.8	2.21	37.5	2.4
		0–63 mm	3.3	-	14.8	47.0	6.2	2.16	44	2.1
Grochów	Magne-site	0–31.5 mm	5.7	14.8	14.5	17.5	6.9	2.02	36.7	4.4
		0–63 mm	6.1	13.0	12.6	18.2	7.3	1.98	56.0	5.5
Winna Góra	Basalt	0–31.5 mm	4.1	49.3	63.7	50.5	6.5	2.23	16.5	1.4
		0–63 mm	2.4	35.1	53.7	50.7	3.9	2.22	17.9	1.4
Rogoźnica	Granite	0–31.5 mm	3.6	15.3	20.5	35.6	3.1	2.08	52.5	7.6
		0–63 mm	2.7	12.9	14.5	33.7	2.1	2.05	42.4	7.4
Żelatowa	Dolomite (2)	0–31.5 mm	10.9	-	31.0	18.0	6.3	2.07	193.3	12.6
		0–63 mm	8.6	-	29.0	16.0	5.6	2.04	178.6	11.0
Osielec	Sands-tone (2)	0–31.5 mm	8.7	35.5	44.2	12.0	4.3	2.05	253.3	2.0
		0–63 mm	4.6	27.2	30.0	12.0	4.7	2.02	280.0	1.4
Barcice	Sands-tone (1)	0–63 mm	12.7	22.5	24.4	13.0	4.5	2.10	269.8	1.6

A similar impact of penetration curve adjustments on CBR results was found for mixtures with the grading of 0–63 mm (Figs. 16–19). The adjustment of the concave portion of the curve resulted in an increase in CBR. For 0–63 mm mixtures, the increase was much higher compared to 0–31.5 mm mixtures. The most significant change, by as much as 87%, was recorded for basalt aggregate and was found in sample 1 penetrated immediately after compaction (Fig. 16). The average increase in CBR for the depth of 2.5 mm is 21.17%, while that for the depth of 5.0 mm is much lower at 8.55%. This average increase for 0–63 mm mixtures is clearly twice as high as that for 0–31.5 mm mixtures. Figures 20–23 show penetration curves for dolomite (1) aggregate with the grading of 0–63 mm and their adjustments in the form of tangents. Large values can be noticed in the case of sample no. 2 after contact with water (Fig. 23); the correction was as much as 1 mm. Shifting the readings by 1 mm meant that for a depth of 5 mm, the corrected reading was 6 mm and it was also the last reading made during the test. This situation shows that running the test to small penetration depths and possibly large adjustments may result in the test being performed poorly and it will not be possible to calculate the CBR value.

On the basis of the obtained values, a statistical analysis was performed with the use of multiple regression. For this purpose, the following factors were taken into account: the resistance index before correction (CBR), the resistance index after correction (CBR’) and sand equivalent (SE). Thanks to this analysis, it is possible

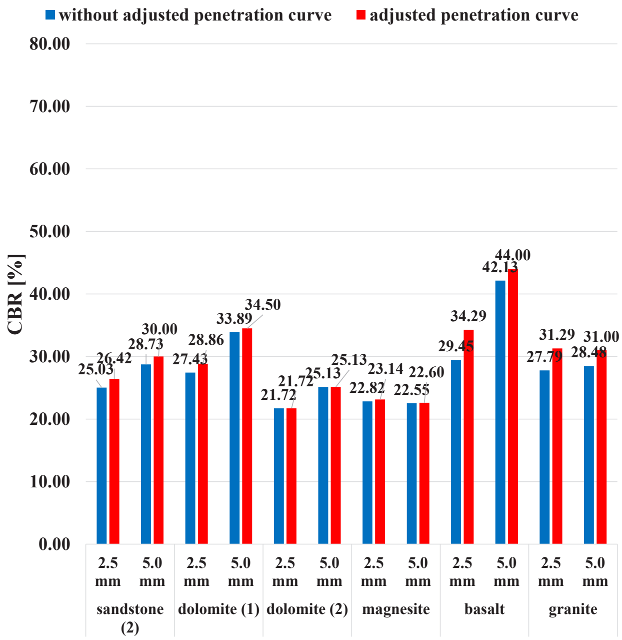

Figure 8: Sample no. 1 of 0–31.5 mm mixture (directly after compaction).

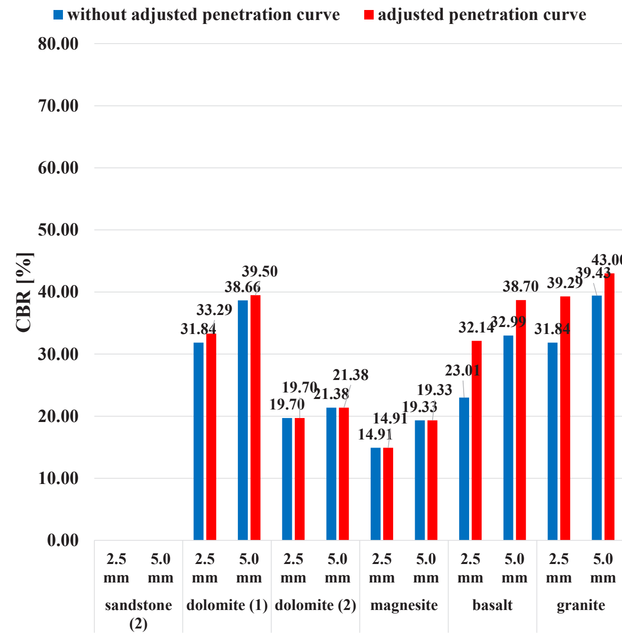

Figure 10: Sample no. 1 of 0–31.5 mm mixture (after four days of soaking in water).

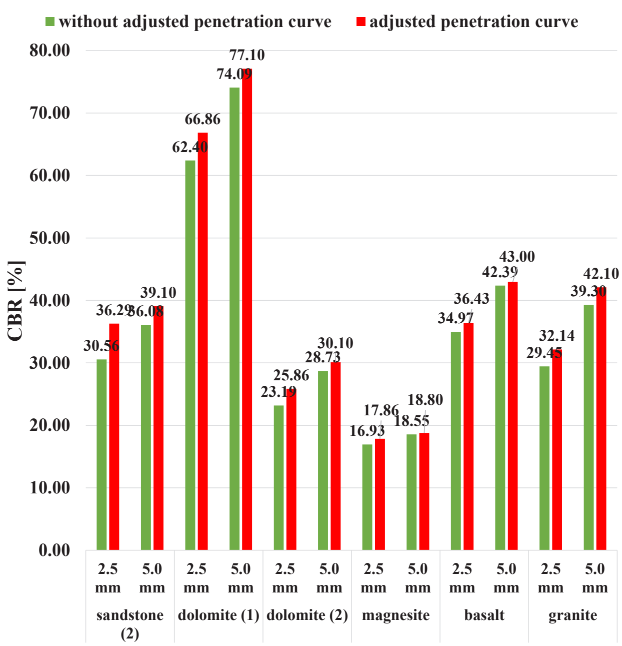

Figure 9: Sample no. 2 of 0–31.5 mm mixture (directly after compaction).

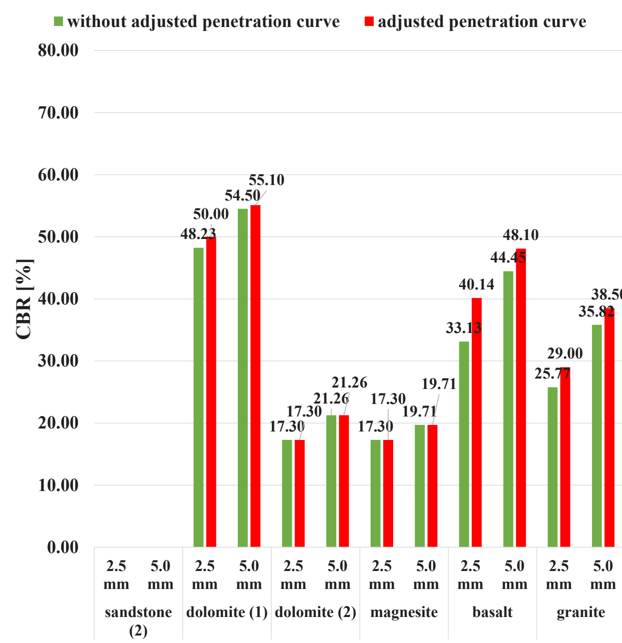

Figure 11: Sample no. 2 of 0–31.5 mm mixture (after four days of soaking in water).

to estimate the value of the bearing capacity index after correction without analysing the penetration curve. Figure 24 shows the results from the obtained statistical model, i.e. the measured and predicted values. On the other hand, Figure 25 shows the set of residuals obtained from multiple regression. It can be noticed that the four values of the residuals obtained from the analysis carried out have a value greater than 10. These points refer to the situations in which the correction of the CBR index caused its greatest increase (41–87%) out of 96 performed corrections.

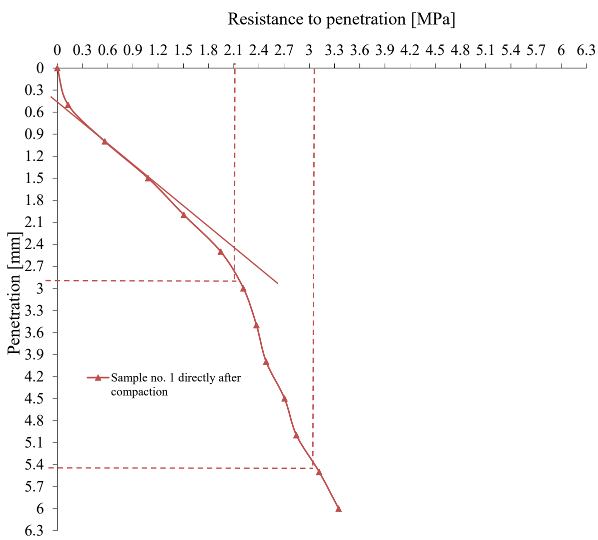

Figure 12: Penetration curves with adjustments for 0–31.5 mm granite aggregate (sample no.1 directly after compaction).

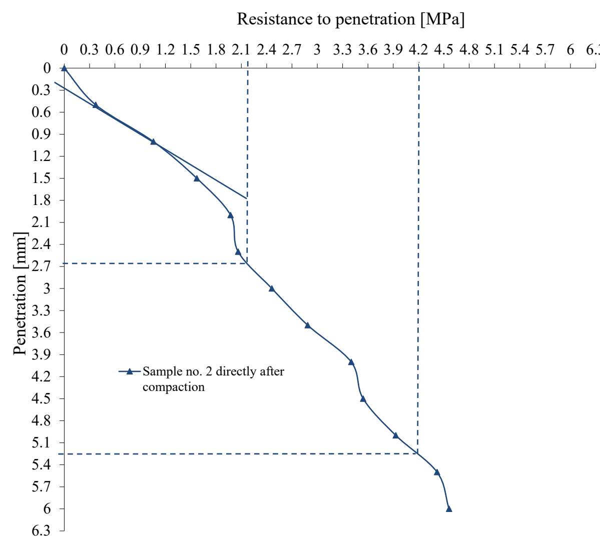

Figure 14: Penetration curves with adjustments for 0–31.5 mm granite aggregate (sample no.2 directly after compaction).

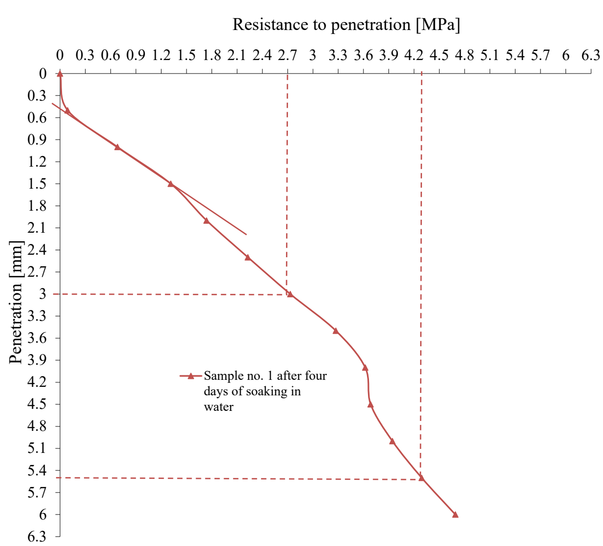

Figure 13: Penetration curves with adjustments for 0–-31.5 mm granite aggregate (sample no.1 after four days of soaking in water).

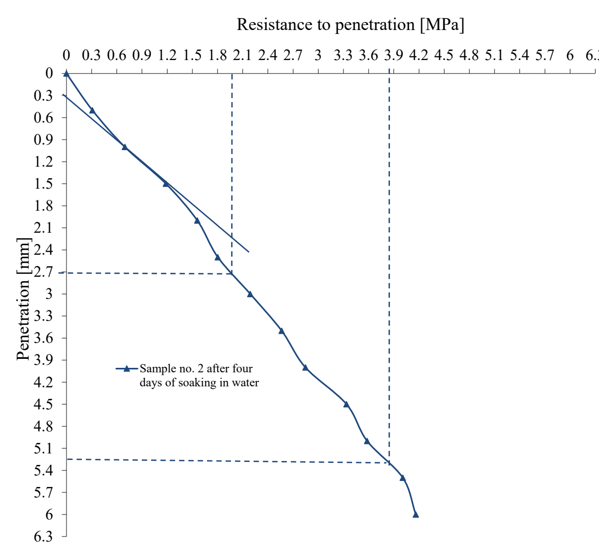

Figure 15: Penetration curves with adjustments for 0–31.5 mm granite aggregate (sample no.2 after four days of soaking in water).

The statistical values of the created model are summarized in Table 3. The values obtained in this way allow to formulate the equation (4), thanks to which it is possible to calculate the load capacity index, including its correction, with a very high accuracy.

$$CBR' = 1.0162 * CBR + 0.1303 * SE - 1.0796 \quad (4)$$

5 Conclusions

The results yielded by the study allow the following conclusions to be drawn:

1. The CBR test is an important study deciding on the use of a given material in road construction, so this test should be carried out with great care. It is important to accurately analyse the penetration curve or have the right shape. A bad concave of the curve can be caused

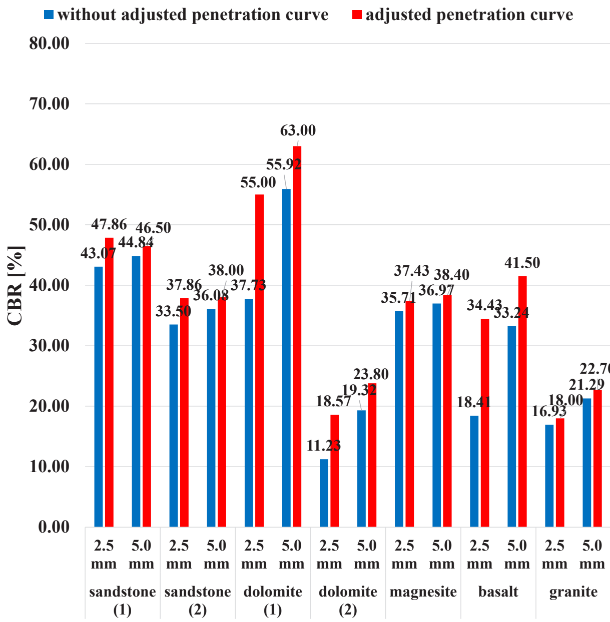

Figure 16: Sample no. 1 of 0–63 mm mixture (directly after compaction).

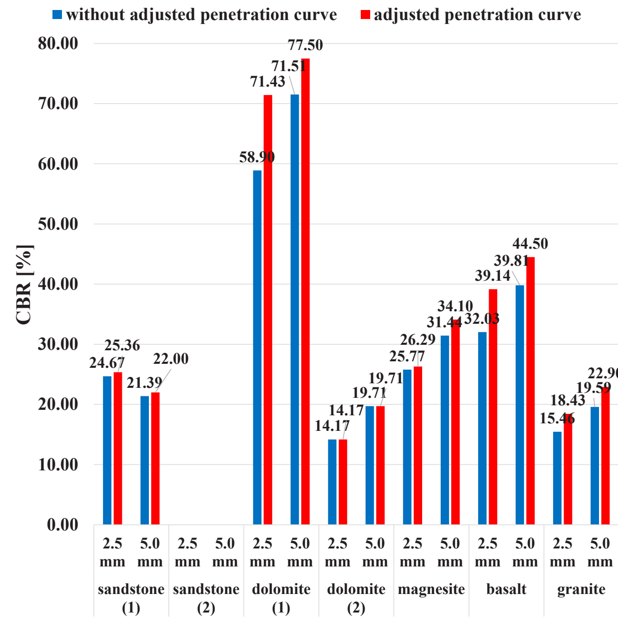

Figure 18: Sample no. 1 of 0–63 mm mixture (after four days of soaking in water).

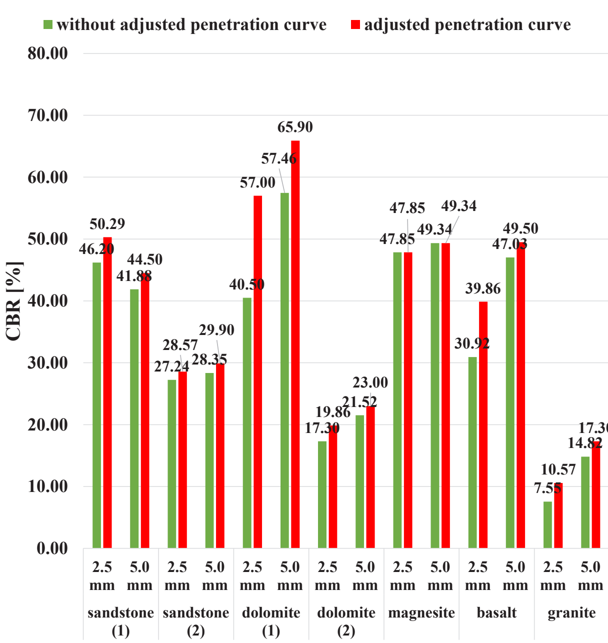

Figure 17: Sample no. 2 of 0–63 mm mixture (directly after compaction).

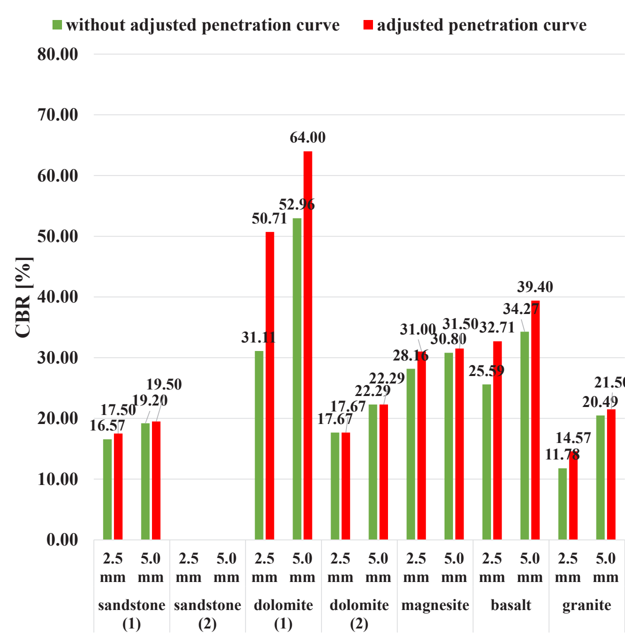

Figure 19: Sample no. 2 of 0–63 mm mixture (after four days of soaking in water).

by poor sample density and uneven surface. However, it is not always possible to obtain a perfectly even sample surface for mixtures with a high grain size.

2. Penetration curve adjustment increases CBR by a few up to several dozen per cent compared to unadjusted results. This average CBR increase is twice as high for 0–63 mm mixtures as that for 0–31.5 mm mixtures.
3. The presented penetration curve for sample no. 2 of the 0–63 mm mixture from dolomite (1) shows that the test should be carried out to a depth much greater

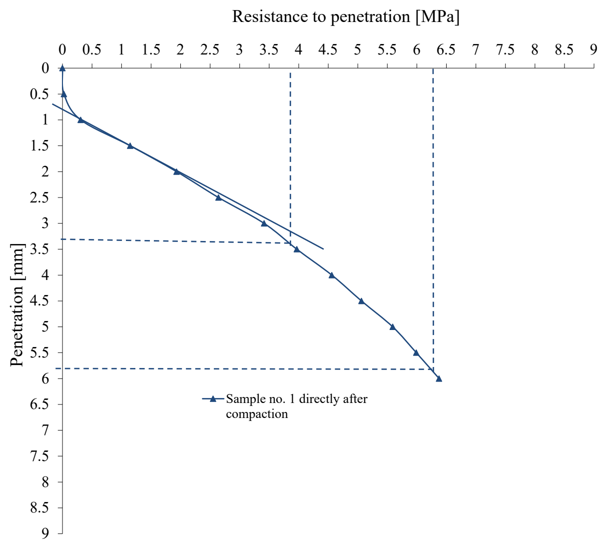

Figure 20: Penetration curves with adjustments for 0–63 mm dolomite (1) aggregate (sample no.1 directly after compaction).

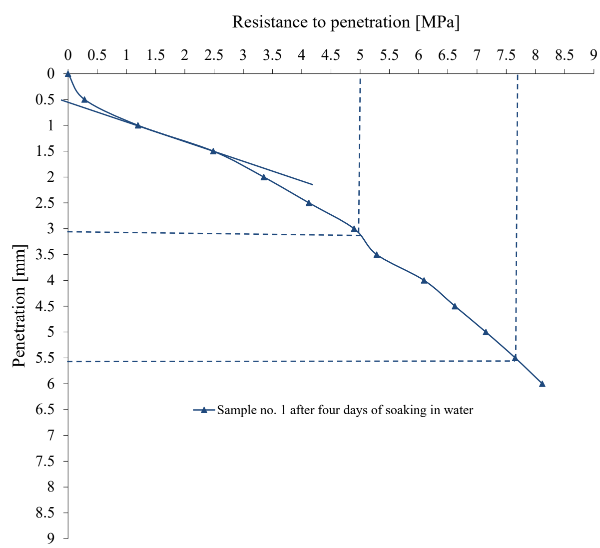

Figure 21: Penetration curves with adjustments for 0–63 mm dolomite (1) aggregate (sample no.1 after four days of soaking in water).

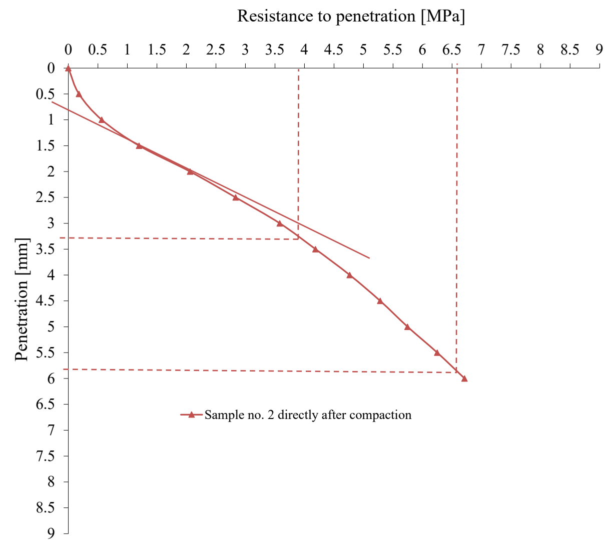

Figure 22: Penetration curves with adjustments for 0–63 mm dolomite (1) aggregate (sa no.2 directly after compaction).

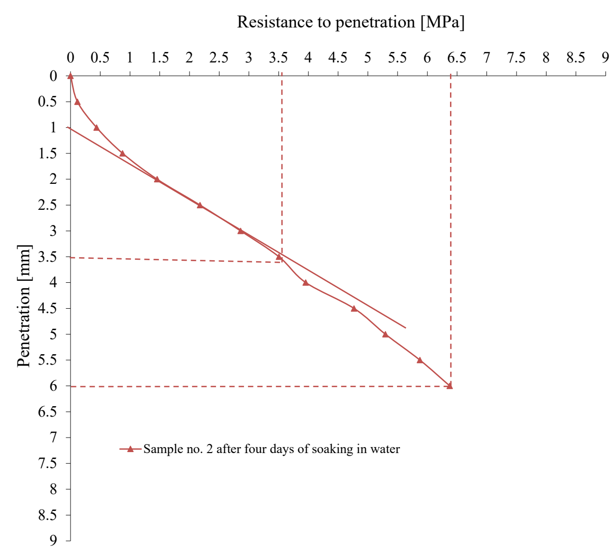

Figure 23: Penetration curves with adjustments for 0–63 mm dolomite (1) aggregate (sample no.2 after four days of soaking in water).

than 6 mm. It is recommended that the mandrel penetrate even to a depth of 10 mm. Too shallow plunger countersink may result in the CBR not being able to be calculated in the event of a high correction. However, it should be remembered that the correction cannot be greater than a depth of 7.5 mm.

4. The largest increase in CBR for 0–31.5 mm mixtures was by 39.70%, while for 0–63 mm mixtures, it was by 87.0%. In both cases, the increase was recorded for basalt aggregate. Basalt aggregate mixtures with grain size 0–31.5 and 0–63 mm were characterized by the highest sand equivalent (respectively, *SE* = 50.5% and *SE* = 50.7%) among the examined aggregates. It can therefore be assumed that a high percentage of sand content in the mixtures will result in lower resistance at the initial stage of penetration of the

Table 3: Regression statistics.

	Coefficients	Standard error	*t* Stat	*p*-value	Lower 95%	Upper 95%	*R* square
Intersection	-1.07964	0.9721	-1.11063	0.269592	-3.01004	0.850757	0.9471
CBR	1.01623	0.029785	34.1188	2.22E-54	0.957083	1.075377	
SE	0.130287	0.025822	5.045615	2.23E-06	0.07901	0.181564	

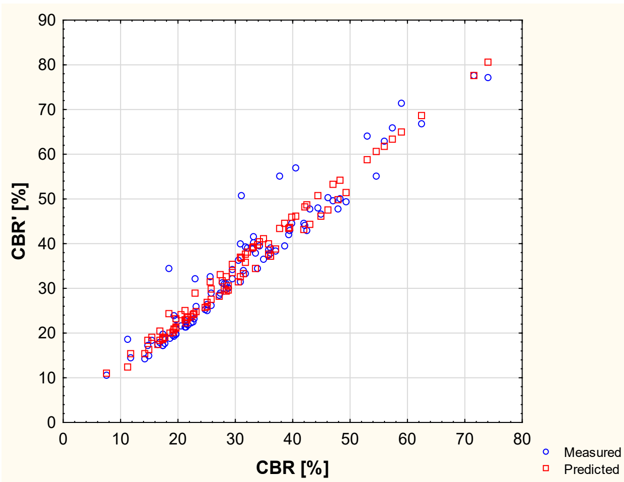

Figure 24: Regression analysis results.

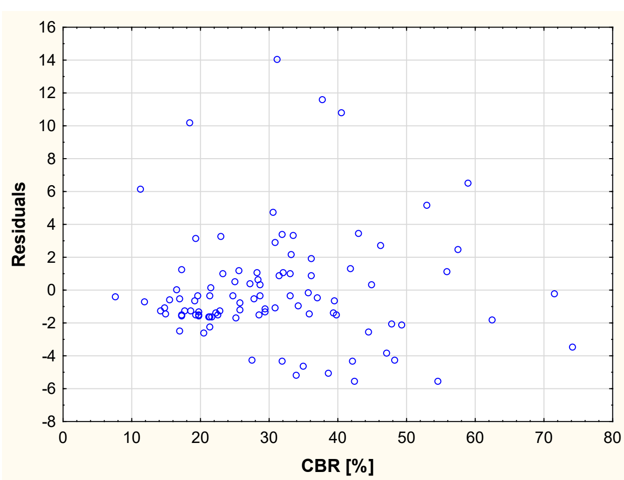

Figure 25: Residuals from analysis results.

mandrel and therefore a greater correction of the CBR index.

5. In a few cases, a large difference in CBR values (even as much as 42%) is noticeable between samples from the same material. This situation may be related to the relatively low grain uniformity of the mixture. For well-grained mixtures, i.e. with a high value of the C_u index and when C_c = 1–3, the differences in the obtained CBR indexes will be low. It is related to the distribution and arrangement of grains during the compaction of the material. If the aggregate mixture has different grain sizes, its correct compaction will not cause differences in the obtained results.
6. The average increase in CBR for a penetration depth of 2.5 mm for 0–31.5 mm mixtures was by 10.42%, and for 0–63 mm mixtures it was by 21.17%. And for a penetration depth of 5.0 mm, the average increase in CBR for 0–31.5 mm mixtures was by 4.65%, while for 0–63 mm mixtures it was by 8.55%.
7. In a few cases, penetration curve adjustment improved CBR so much as to allow the mixture to be used for a higher traffic class or pavement (base or subbase) layer. A case in point is a mixture of 0–63 mm dolomite aggregate.
8. The developed multiple regression model allows for the calculation of the CBR' index taking into account its correction without analysing the penetration curve. It turned out that for this purpose, the *CBR* index without correction and the sand equivalent *SE* are needed. The obtained statistical values allow to conclude that the proposed equation will make it possible to calculate the index with a high probability of accuracy.

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