Some Remarks on Practical Aspects of Laboratory Testing of Deep Soil Mixing Composites Achieved in Organic Soils

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Abstract. This paper presents the results of laboratory testing of organic soil-cement samples are presented in the paper. The research program continues previously reported the authors’ experiences with cement-fly ash-soil sample testing. Over 100 of compression and a dozen of tension tests have been carried out altogether. Several samples were waiting for failure test for over one year after they were formed. Several factors, like: the large amount of the tested samples, a long observation time, carrying out the tests in complex cycles of loading and the possibility of registering the loads and deformation in the axial and lateral direction – have made it possible to take into consideration numerous interdependencies, three of which have been presented in this work: the increments of compression strength, the stiffness of soil-cement in relation to strength and the tensile strength. Compressive strength, elastic modulus and tensile resistance of cubic samples were examined. Samples were mixed and stored in the laboratory conditions. Further numerical analysis in the Finite Element Method numerical code Z_Soil, were performed on the basis of laboratory test results. Computations prove that cement-based stabilization of organic soil brings serious risks (in terms of material capacity and stiffness) and Deep Soil Mixing technology should not be recommended for achieving it. The numerical analysis presented in the study below includes only one type of organic and sandy soil and several possible geometric combinations. Despite that, it clearly points to the fact that designing the DSM columns in the organic soil may be linked with a considerable risk and the settlement may reach too high values. During in situ mixing, the organic material surrounded by sand layers surely mixes with one another in certain areas. However, it has not been examined and it is difficult to assume such mixing already at the designing stage. In case of designing the DSM columns which goes through a thin layer of organic soil it is recommended to carry out each time the core drilling which checks the degree of material mixing and their strength.

1. Introduction – benefits and limitations of soil mixing
The technologies of deep mixing of the soil with mineral binders are still in the process of development and they find a vast area for implementation in geotechnics. Apart from their traditional usefulness for the reinforcement of the subsoil at the foundations of various structures [1], where the DSM columns may play the role of quasi-piles that transfer the loads to deeper layers of the subsoil, the blocks of reinforced soil may also serve as an extended shallow foundation or even only as “stiff inclusions”, reducing the subsoil compressibility under a shallow foundation.
The leaders in the DSM technologies are the countries that first developed Deep Soil Mixing techniques (Japan, Sweden, USA, and lately also Poland). In the literature across the world there are numerous compilations and rapport (for example [2]) from soil-cement tests, including those based on the organic soil component [3]. Polish experience entail mainly the tests carried out in Gdańsk (in peats) [4] and in Bydgoszcz [in gyttja] [5,6] and recently also at Wrocław University of Science and Technology [7]. In Poland, among the range of the DSM technology applications, most common is the reinforcement of the subsoil under structures (the „wet” DSM). Many experiments and much research of the leaders of that technology were published in Poland and abroad [8-11][10]. The option of the „dry” deep soil mixing („dry” DSM), popular in Scandinavia, is not widely offered on the Polish market. In the „dry” DSM, the obtained values of compression strength ($f_c$) of the columns vary from 1.0 to 6.0 MPa. In “dry” mixing we aim at obtaining a product with, for example, $S_u=90$ kPa (which is a significant quantitative, and even a qualitative, difference). It must be stressed that in “dry” deep soil mixing we use water which is in the ground and it takes part in the process of biding cement and soil. Therefore, the „dry” mixing can also be used, as the last resort, for organic soils, in which the humidity values are very high. In the “wet” technology, cement is applied together with water in the form of a slurry, and many negative experiences prove that this DSM variant is not applicable for homogeneous organic soils. These technologies differ in the way cement (binder) is added to the soil, and the products that result from the mixing are completely different. Basically, the “wet” DSM technologies are used only in the case of small interbeddings of organic components in the soil, which can be mixed with the mineral skeleton. Also in this case it may be very difficult to maintain the homogeneity of the material due to a limited vertical transportation within the column. When we plan a deep soil mixing in such a situation, many questions come to mind, like for example, what can be the maximum thickness of an organic interbedding, what strength can be achieved by way of mixing the soil with the slurry, or what is the durability of the cement-soil based on the organic component, etc.

The laboratory tests aimed at establishing the compression strength and the modulus of volume elasticity, assessed on the basis of the triaxial compression of cubic samples of 15×15×15 cm for differing times since their forming. The subject of tests was also the tensile strength and the samples’ stiffness (measured with the value of the modulus at different stages of loading). It must be stressed that the cement-soil mixed in laboratory conditions is always more homogeneous than the one obtained in the in-situ conditions and sampled with a scoop. A good marker of homogeneity is the samples’ weight – for each batch of mixture, the samples were weighed also before they were bound and the obtained weight differences amounted to a few grams for a 4.5-kilogram sample cube. Also the tests in the dissertation by Zającowski reported in [7], carried out on the samples of cement-soil with the admixtures of fly ashes, confirm unequivocally the variability of parameters of samples taken in situ in comparison with a large homogeneity of the parameters of similar mixtures composed in laboratory room. Altogether, so far 100 compression tests and a dozen or so tension tests have been conducted. The statistically significant number of tested samples, a long-time of monitoring and observation, the complex loading cycles in which the tests were carried out, as well as the possibility of a constant recording of the loading and deformation in the axial and transverse (lateral) directions – are the factors that made it possible to analyze many interdependencies, the three of which are presented in this paper: the increment of compression strength in time, the stiffness of cement-soil in relation to strength, and tensile strength.

2. Parameters of the materials used for the making of sample batches of mixture
Eventually, two types of organic soil and two types of cement (with the parameters specified below) were used for the tests – even though initially the use of three organic soils was planned, i.e. clayey aggregate mud Or 1 and two peats of a different origin, marked Or 2 and Or 3, respectively. Due to the hard-plastic state of the aggregate mud and the difficulty in its mixing, the idea of its use in the experiment was abandoned. It is definitely easier to mix the soil in laboratory conditions (it is possible to crush it into smaller parts) than in situ, so it could be concluded that an attempt at deep soil mixing of this soil in the field would fail. The soil Or 2 was a peat with a volume of 1.2 g/cm³ and the content
of the organic component of about 45%. That peat was taken from the depth of 3.0 – 3.5 m below the ground surface. The Or 3 peat was characterized by a volume density of 1.0 g/cm³ and the content of organic component of circa 40%. It was taken at the depth of 1.5 – 2.0 m below the ground surface. Its humidity measured in situ was 950%!

The hydraulic binder for the making of the cement-soil samples were two types of cement: CEM II B-S 32.5 R – NA and CEM IIIA 32.5 N/LH/HSR/NA. Both cement types are characterized by the strength of 50 MPa achievable after 28 days. The differences are visible in the rate at which the strength increases: the CEM II cement reaches the strength of about 18.5 MPa already after 2 days, whereas the CEM IIIA cement exhibits the strength circa 22.0 MPa only after 7 days [12]. The author’s previous experience from the testing of cement-soil pointed to a comparable usefulness of both cement types for the forming of the DSM material in mineral soils (sands, dusts, clays). The cubic samples of cement-soil, with the dimensions of 15×15×15 cm, were prepared by mixing the organic soil together with the slurry, the density of which equaled 1.5 g/cm³. The cement was applied in doses of pre-defined values so that the final outcome gave 300 or 400 kg/m³ of cement-soil. It must be highlighted that such an amount of cement is relatively large as for deep mixing – most often the amount of cement applied in situ is no more than 300 kg/m³.

3. Laboratory test methodology and results

The tests were conducted for a constant displacement with the velocity of 0.01 mm/s, in controlled temperature of 20°C ± 3°C. The uniaxial compression strength of the cubic cement-soil samples were carried out as outlined in the Code of Practice [13] and method [14], shown in Figure 1, in the PROETI mechanic press, synchronized with a computer recording:

- time elapsed since the beginning of the test,
- axial force loading the sample,
- axial displacement of press piston (reduction of the sample’s length in the axial direction),
- lateral deformation on the horizontal axes of the sample.

![Figure 1. a) Cubic sample of cement-soil in the testing machine. b) Sample after the test.](image)

The data were sent on the ongoing basis to the PC equipped with software for automatic recording of tests. For each series, the tests were conducted, pre-ordering one of the three following loading modes:

- 1/3 of the samples – a standard test with no loading,
- 1/3 of the samples – a test with one loading for $\sigma_z \approx 0.5 \times \sigma_{z,\text{max}}$,
- 1/3 of the samples – a test with two loadings for $\sigma_z \approx 1/3 \times \sigma_{z,\text{max}}$ and $\sigma z \approx 2/3 \times \sigma_{z,\text{max}}$. 
The modulus of volume elasticity was determined as the mean modulus for an approximately rectilinear fragment of a tension-deformation curve by a linear interpolation for various ranges of that curve. The linear interpolation was made by means of the least squares method in Wolfram Mathematica software.

The moduli of volume elasticity were determined depending on the amount of loadings, in the following way:

- no-load test – modulus of volume elasticity \( E_1 \) (Figure 2a),
- one-load test – modulus of volume elasticity \( E_1 \) for a curve before the unloading; the modulus from the loading-unloading curve \( E_{1,odc} \), and the modulus of volume elasticity \( E_2 \) for a curve with a fragment of unloading (Figure 2b),
- two-load test – modulus of volume elasticity \( E_1 \) for the curve before the first unloading, modulus from the first curve of the unloading \( E_{1,odc} \), modulus of volume elasticity \( E_2 \) for the curve between two fragments of unloading, the modulus from the second curve of the unloading \( E_{2,odc} \), and the modulus of volume elasticity \( E_3 \) for the curve from the second unloading (Figure 2c).

**Figure 2.** Marking of the moduli determined for: (a) the no-load variant, (b) the one-load variant and (c) the two-load variant.

The tests for tension strength during the crushing of the samples of cement-soil were carried out in a way similar to compression tests – on cubic samples with the dimensions of 15×15×15 cm. The value of the tensile strength while crushed \( f_t \) was obtained from the formula (1):

\[
f_t = \frac{2 \cdot F_{max}}{\pi \cdot L \cdot d},
\]

where \( F_{max} \) is the maximum loading, \( L \) is the length of the contact line, and \( d \) is the size of the cross-section of the sample.

Over 100 compressions and over a dozen of tension tests have been carried out altogether. Several samples were waiting for failure test one year after they were formed. Several factors, like: the large amount of the tested samples, long observation time, carrying out the tests in complex cycles of loading and the possibility of registering the loads and deformation in the axial and lateral direction – have made it possible to take into consideration numerous interdependencies, three of which have been presented in this work: the increments of compression strength, the stiffness of soil-cement in relation to strength and the tensile strength.
3.1. Increments of compressive strength in time – discussion of results

In accordance with the Code of Practice [13], the following formula may be used in order to estimate strength in time:

\[
f_{cm}(t, s) := f_{cm1} \cdot \exp \left[ s \cdot \left( 1 - \left( \frac{28 \cdot \text{day}}{t} \right)^{0.5} \right) \right],
\]

(2)

Where: \( t \) is time, \( f_{cm1} \) is the strength of concrete after 28 days of its curing, whereas \( s \) is the coefficient dependent on the cement type and the rate of its setting (for concrete 0.20 – 0.38). In this article we used the above dependence (2) to interpolate the strength parameters in time. Parameters \( s \) and \( f_{cm1} \) were assumed as unknown, which were then searched for by means of the least squares method for each particular series of samples. Figure 3 shows value \( f_c \) of particular series for different point in time, supplemented with function (2) obtained from the best matching (marked by the solid line) to laboratory tests. After the analysis of the results presented in Figure 3, the following conclusions can be drawn (which, however, in turn demand further critical analysis):

- in most cases, the level of the obtained strength values does not exceed 500 kPa and is attained as late as after 56 days;
- as a rule, in the period between the 56th to 84th day of cement curing, a drop in strength is observed;
- it is not possible to state which cement type made it possible to obtain higher strength values;
- paradoxically, only in half of the cases did the higher cement content result in higher strength.

Figure 3. The increment of uniaxial compression strength in time for series Or 2 and Or 3 with various cement content
Low strength of soil-cement formed in peats confirms the tests previously conducted by Leśniewska [4]. The long time necessary to attain the maximum strength proves that it is desirable to take into consideration the plan of quality control of the material from which the columns are formed. The test carried out after 28 days may be unreliable (the results may be significantly underrated). The observed decreases in the samples’ strength examined after 3 months are also alarming. They point to the degradation of the cement-organic material even if there are no external corrosive factors. The test samples were stored in a humid environment, humus-acid free, which would be the situation of an actual column formed in hydrated peats.

The lack of correlation between the strength and the cement type and, what is worse, the lack of unequivocal relationship between the strength and the amount of the added slurry content, show that for every series of tests the decisive impact on the results rests within the type of soil used for subsequent concrete mix. Although the test results for specific concrete mix types converge (even for the unfavourable tendency to lose strength over a period of 3 months), there is a significant divergence in the soil composition in samples gathered to one bag from a seemingly homogeneous layer and a similar depth.

Figure 4 shows value $f_c$ of series Or 2 and O3 for different point in time on the background of results published in [11]. These results show $f_c$ increase for DSM columns performed in sandy soils (red dashed line). $f_c$ values achieved in sandy soils are about 10 times bigger than those obtained in organic soils. It seems that the strength increase is similar in organic and sandy soils. However, it must be noticed that there is a decrease of $f_c$ for some part of organic samples.

![Figure 4](image)

**Figure 4.** The increment of uniaxial compression strength in time for series Or 2 and Or 3 compared to results obtained in sands [11]

3.2. *Stiffness vs. strength of soil-cement*

The modulus of volume elasticity ($E$) was determined as a mean from the moduli: $E_1$, $E_2$ and $E_3$. Its value for a given series and particular point in time are presented in Figure 5 in relation to the uniaxial compression strength. As before the results are showed with the background of results obtained in sandy soils presented in [11]. It is visible that this dependence is practically linear and it does not depend on curing time of the test sample, the type of concrete used, and the kind of organic soil applied.
This also means that the possible decrease in strength described in paragraph 3.1. is linked with the simultaneous drop of the test sample’s stiffness. The obtained dependence of compression strength on the mean moduli of volume elasticity may be also approximated by a straight line. This straight line (Figure 5) shows a good matching independent of the kind of organic soil.

Figure 5. The dependence of $E$ modulus on $f_c$ for the soil-cement samples under test compared to results obtained in sands [11].

According to the equation of this straight line, the dependence between the $f_{c,\text{org}}$ vs. $E_{\text{org}}$ will be formulated as follows (3):

$$E_{\text{org}} \approx 120 \cdot f_c [\text{MPa}]$$

It is necessary to remember that this formula is true for the tested group of organic soil and cement types. Generally, it may give the idea of the rank of the values of these moduli in the cases when the organic soil (peat) is mixed with cement. Such value of the modulus of volume elasticity was assumed for the needs of the numerical analysis shown in the next paragraph. It is worth to notice that the relation (3) is significantly different in other type of soils [11]. So in no organic and organic soils not only the level of $f_c$ and $E$ is different but also the relation between them.

3.3. Tensile strength vs. compression strength

Tensile strength was determined for the curing time of 56 days, i.e. for the expected time needed to complete the curing of the ground-cement material. The results obtained in course of testing do not present a considerable amount of trials in the tension test. Despite that, the value of the obtained $f_c/f_t$ relations is consistent with previous experience of the authors in the tests of road stabilization. By way of generalization, it is possible to assume that the tensile strength of soil-cement $f_t$ equals approximately 10% of the compression strength $f_c$. In designing, however, it will be safe and reasonable to assume that the value of $f_t$ is equal to zero.

Of course, it must be also remembered that soil-cement – especially the one obtained in situ – will never be as homogeneous and resistant as the mix prepared in laboratory conditions. Some consolation may be offered by the fact that in the DSM columns with large diameters one can count on the averaging of the values of strength parameters on the possible failure surfaces. In order to confirm that intuition, though, large-scale tests on core samples would be necessary.
4. Numerical model
The data for the numerical model were taken from a typical design of a two-span highway overpass. Taking advantage of the symmetry of the arrangement, a sector of the bridge abutment foundation was modelled together with three DSM columns with the diameter of 1 meter. The foundation (1 meter high and 6 meters wide) was loaded on the surface with an appropriate portion of vertical load acting on the foundation. Several models were made, differing in relation to geotechnical conditions, i.e. the thickness of the weak (organic) soil which ranged between 0-2 meters.

The ceiling of the organic soil always occurs at the depth of 5 m below the bottom of the foundation. Below that level in every case there is a load-carrying layer – sandy soil, in which the last meter of the DSM column is anchored.

Figure 6 shows the model and the DSM columns for the option where the sandy soil thickness (A) is equal to 5.5 m, and the thickness of organic soil (B) equals 0.5 m. In the calculations, the authors applied the HSS (Hardening Soil Small) material model for organic soil, and the linear-elastic model for the foundations and the DSM columns material.

In this example control of stresses in linear-elastic model was neglected which should not be done in a real case. The sand parameters were assumed on the basis of the calculator included in Z_Soil software. These are the typical parameters for compacted medium sands. The organic soil parameters were estimated using the same calculating tool, having at our disposal the results of edometric tests and the tests of the soil physical properties. As far as the DSM columns parameters are concerned, their modulus of volume elasticity was assumed within the thickness of organic soil and thus equal to 24 MPa (120×0.2 MPa), whereas for the column formed in sands – equal to 900 MPa (300×3.0 MPa). As it was assumed that the column was formed by way of the “wet” DSM, the possible “vertical transportation” of the column material was disregarded (as well as the occurrence of the zones with averaged parameters).

5. Numerical FEM computation results
The results of the numerical analysis have been shown in Table 1. The table presents the maximum settlement values at the level of the foundation’s bottom.

![Figure 6](image)

**Figure 6.** 3D view of the whole numerical model and the DSM column separately for the 5.5:0.5 proportion of sand (A) to peat (B).

![Figure 7](image)

**Figure 7.** Picture of DSM columns deformation for model A=5.5, B=0.5.
Table 1. The results of numerical calculations

| A - Sand [m] | B - Peat [m] | S_{\text{max}} [mm] |
|--------------|--------------|---------------------|
| 4.0          | 2.0          | 92                  |
| 5.0          | 1.0          | 49                  |
| 5.5          | 0.5          | 26                  |
| 6.0          | 0.0          | 3                   |

The results of calculations show that the obtained settlement values are practically in each case too high for the designed bridge. The only exception is the system without organic soil and, perhaps, the model where its thickness amounts to 0.5 m. In addition, almost in every case, the locally obtained stress values exceeded the compression strength of the “cement-organics.” Figure 7 includes a scaled image of deformation of these columns under a full load when A=5.5 and B=0.5. It may be noticed that the whole of the DSM columns at the level above the ground is displaced as a rigid body, and the settlement is possible only in the layer of the organic soil.

The numerical analysis presented above includes only one type of organic and sandy soil and several possible geometric combinations. Despite that, it clearly points to the fact that designing the DSM columns in the organic soil may be linked with a considerable risk and the settlement may reach too high values. During in situ mixing, the organic material surrounded by sand layers surely mixes with one another in certain areas. However, it has not been examined and it is difficult to assume such mixing already at the designing stage. In case of designing the DSM columns which go through small lenticels of organic soil it is recommended to carry out each time the core drilling which checks the degree of material mixing and their strength.

6. Summary and conclusions

It is necessary to remember that the DSM technology is recommended mainly for the reduction of settlement in the situations when the load-bearing capacity is ensured. In most cases that is not possible when the subsoil contains organic layers.

Deep soil mixing by means of the wet method (wet DSM) in general is not applicable for organic soils with high organic content. On the basis of the laboratory tests and numerical analysis, the following conclusions have been drawn:

- in most cases of mixing peats with cement content of 300/400 kg/m³, the level of strength did not exceed 500 kPa;
- the relation between $f_c$ and $E$ in case of mixed peats is slightly different than it is for no organic soils with resistance higher than 1 MPa;
- the maximum values of strength obtained after 56 days are not stable in time – after 3 months it was noticed that for some specimen strength and stiffness decreased as compared with the 56-day trial;
- the potential degradation of strength poses a question mark to the credibility of the tentative soil-cement mix, when the long-time of the column’s work is expected under the construction;
- even if the tentative mix of organic soil and cement are made and the moduli of volume elasticity estimated, it may turn out that no rational designing of the DSM columns is possible in such conditions;
- if one takes the risks of mixing the interbeddings of organic soil, the requirements of the final construction acceptance should always be radically made stricter; also, core drilling must be performed in such columns and estimate the potential weakening linked with the corrosion of the column material in time.
It must be noted that the tests were conducted on the grounds with the content of the organic component of 40-45%. The soils with the organic content at the level of a few percent may behave in a different (more advantageous) way. It must be stated that all type of soils can be mixed but the designer must foresee what level of $f_c$ and $E$ will be achieved. The durability of these parameters should be assured. That is why the authors recommend that sample batches of mixture should be made each time an organic ground is being classified, so that it is possible to assess its behaviour in time.

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